Operation of Decentralised Wastewater Treatment Systems (DEWATS) under tropical field conditions

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Declarations

I, Nicolas Simeon Reynaud, herewith confirm the conformity of this copy with the original dissertation titled

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I further declare that unless indicated, this thesis is my own work and that it has not been submitted, in whole or in part, for a degree at another University or Institution.

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Signature

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Abstract

Decentralised Wastewater Treatment Systems (DEWATS) such as disseminated by the Bremen Overseas Research and Development Association (BORDA) are increasingly being recognized by decision makers across the world as an option for service delivery in densely populated low-income areas. However, little practical experience has been gathered methodologically on basic engineering and performance aspects surrounding these systems.

This thesis investigates full-scale anaerobic reactors of communal DEWATS implemented in tropical regions in order to consolidate the basis of future design and support monitoring, operation and maintenance procedures. Special focus is laid on the operation of the Anaerobic Baffled Reactor (ABR) as the core technology of DEWATS.

Field research has been conducted for over four years at numerous communal systems in Indonesia, India and South Africa in order to (i) verify the generally used parameter values for DEWATS design and operation, (ii) identify factors limiting the treatment efficiency of existing systems in the field and (iii) investigate the performance of DEWATS and DEWATS treatment steps (especially ABRs) under tropical field conditions in terms of effluent concentration, Chemical Oxygen Demand (COD) removal, sludge stabilisation and sludge activity.

Field data on average per capita wastewater production in DEWATS implementation areas, long term fluctuations and peak-flow values are presented. General per capita organic load and per capita nutrient load, per capita biogas production in digesters and per capita sludge accumulation in ABRs are estimated.

Based on available data and field observations, treatment limiting factors are hypothesised to be rain-water intrusion, general under-loading, organic under-loading and elevated raw-water salinity in coastal areas.

Effluent measurements performed at one hundred nine systems in Indonesia indicated guaranteed maximum concentrations of 200 mg COD_t l^{-1} for anaerobic DEWATS treatment effluent if the treated wastewater is non-saline.

ABR COD reduction of four case studies was poor in three cases and fair in one case. Sludge accumulation rates indicated good sludge stabilisation and sludge activity in all four systems. Anaerobic Filters (AF) contributed in all three case studies, in which they were part of the plant design, significantly to COD reduction. Nutrient effluent concentrations were comparably high. Large fractions of effluent organics were found to be biodegradable.

It is hypothesised that system treatment would improve significantly if maximum hydraulic load was lower, general organic load was higher and therefore both close to design estimation. It is thus proposed to control the amount of storm water intruding the systems, increase feed concentration through partial grey-water exclusion and reduce the nutrient load in system effluent through partial urine diversion. It is further proposed to reduce the HRT of the settler below 10 h in order to increase the organic load to the ABR.

It is further hypothesised that systems could be operated at higher hydraulic dry weather load than currently assumed since active anaerobic digestion appears to be capable of establishing itself under extreme hydraulic pressure. This may lead to a considerable reduction of building costs.

Anaerobic digestion modelling with the existing ADM-3P model confirmed that observed sludge accumulation rates indicate active hydrolytic systems. The model could however not be used to produce soluble COD effluent concentration benchmarks due to its sensitivity to methanogenic rate constants. The general view held for anaerobic reactors treating wastewater with high solid content is that hydrolysis is the rate-limiting degradation step. It is hypothesised that this does not apply for solid accumulating systems such as the ABR.

Hypotheses resulting from the thesis

- The average per capita BOD load in DEWATS implementation areas is **20 to 40 g BOD_5 cap^{-1} d^{-1}**.
- The average per capita wastewater production in DEWATS implementation areas in Java/Indonesia is **80 l cap^{-1} d^{-1}** and does not depend on average household income.
- The average per capita wastewater production in water stressed and poor DEWATS implementation areas in Bangalore/India is **30 l cap^{-1} d^{-1}**.
- The average peak-flow factor of wastewater production in DEWATS implementation areas in Java/Indonesia, Bangalore/India and Durban/South Africa is **1.9**.
- The average per capita nutrient production across investigated DEWATS implementation areas is **5.6 g NH_4-N cap^{-1} d^{-1} and 0.8 g PO_4-P cap^{-1} d^{-1}**.
- Approximate average nutrient concentrations of DEWATS anaerobic treatment step effluent is **70 mg NH_4-N l^{-1} and 10 mg PO_4-P l^{-1}**.
- The average per capita biogas production of communal wastewater fed biogas-digesters is approximately **20 l cap^{-1} d^{-1}**. Whether the wastewater is pure black-water or mixed black- and grey-water does not noticeably affect the per capita biogas production.
- The optimal design HRT for biogas digesters fed with communal wastewater is **2.5 d**.
- The per capita sludge accumulation rate in ABRs operated in Indonesia and India is approximately **3 to 8 l cap^{-1} y^{-1}**. The HRT and type of pre-treatment (whether it is a settler or biogas digester) do not noticeably affect the sludge accumulation.
- Factors limiting DEWATS treatment are rain-water intrusion, general under-loading, organic under-loading and elevated raw-water salinity in coastal areas.
- The currently guaranteed maximum effluent concentration for anaerobic DEWATS treatment, provided the treated wastewater is non-saline, is **200 mg COD_t l^{-1}**.
- The time of day at which DEWATS effluent samples are drawn does not significantly influence the COD measurement outcome.
- COD reduction through ABRs may currently be considered non-optimal in numerous systems.
- Large COD fractions of anaerobic DEWATS effluents are biodegradable.
- Sludge-stabilisation and sludge-digestion inside ABRs may currently be considered good.
- AFs contribute significantly to COD reduction in numerous systems.
- ABR and AF treatment would improve significantly if maximum hydraulic load was lower, general organic load was higher and therefore both loads close to design estimation.
- ABRs could be operated at higher hydraulic dry weather load than currently assumed which would lead to a considerable reduction of building costs.
- Solid retention of ABRs should be improved by designing the last reactor chamber larger and by connecting a lamella clarifier before its effluent.
- Anaerobic digestion modelling with the existing ADM-3P model and its current calibration confirmed that observed sludge accumulation rates indicate active hydrolytic systems.
- Anaerobic digestion modelling with the existing ADM-3P model and its current calibration cannot be used to produce soluble COD effluent concentration benchmarks.
- Hydrolysis is not the rate-limiting degradation step in solid accumulating systems such as the ABR.

Kurzfassung

Die durch die Bremen Overseas Research and Development Association (BORDA) verbreiteten Decentralised Wastewater Treatment Systems (DEWATS) werden international von Entscheidungsträgern zunehmend als Möglichkeit angesehen, kommunale Abwasserreinigung in dichtbesiedelten, einkommensschwachen Gegenden zu ermöglichen. Allerdings wurden bislang wenig praktische Erfahrungen methodisch über grundlegende Aspekte der Anlagendimensionierung und Anlagenleistungsfähigkeit aufgenommen.

Im Rahmen dieser Dissertation wurden anaerobe Reaktoren kommunaler DEWATS unter tropischen Feldbedingungen untersucht, um eine Datengrundlage für zukünftige Dimensionierung, Wartung und Betrieb, als auch Monitoring der Anlagen zu schaffen. Schwerpunkt wurde dabei auf den Anaerobic Baffled Reactor (ABR) als Kerntechnologie von DEWATS gelegt.

Felduntersuchungen wurden in der Zeit von mehr als vier Jahren an zahlreichen kommunalen DEWATS in Indonesien, Indien und Südafrika durchgeführt, um (i) die gängig gewählten Parameterwerte für Anlagen-Dimensionierung und -Betrieb zu überprüfen, (ii) leistungslimitierende Faktoren im Feldbetrieb zu identifizieren und um (iii) die Leistungsfähigkeit von DEWATS und DEWATS-Reinigungsstufen (insbesondere des ABRs) unter tropischen Feldbedingungen bezüglich Abflusskonzentrationen, Reduzierung des Chemischen Sauerstoffbedarfs (CSB), Schlammstabilisierung und Schlammaktivität zu untersuchen.

Basierend auf den Untersuchungsergebnissen, wurden durchschnittliche Einwohnergleichwerte, Langzeitvariationen und Faktoren für Zuflussspitzen für kommunale Abwasserproduktion in DEWATS-Zielbevölkerungsgruppen präsentiert. Ferner werden allgemeine Pro-Kopf-CSB-Frachten, -Ammoniumfrachten und -Phosphorfrachten, die Pro-Kopf-Biogasproduktion in kommunalen Biogasanlagen sowie die Pro-Kopf-Schlammakkumulation in ABRs abgeschätzt.

Auf Felduntersuchungen basierend, wurden Fremdwassereinfluss, generelle Unterbelastung, organische Unterbelastung und erhöhte Frischwassersalinität in Küstengebieten als leistungslimitierende Faktoren im Feldbetrieb identifiziert.

An 109 indonesischen Anlagen durchgeführte Abflusskonzentrationsmessungen ließen auf eine garantierte Abflusskonzentration der anaeroben Reaktoren von 200 mg CSB l^{-1} schließen, wenn der negative Einfluss von erhöhter Frischwassersalinität ausgeschlossen werden kann.

Der CSB-Abbau durch ABRs in vier detailliert untersuchten DEWATS war gering in drei Fällen und befriedigend in einem Fall. Anaerobe Filter (AF) trugen in den drei Fällen, in denen sie Teil der Anlagenkonfigurationen waren, signifikant zur CSB-Reduzierung bei. Ammonium- und Phoshorkonzentrationen in allen Reaktorabläufen waren vergleichsweise hoch. Ein großer Anteil des CSBs in Reaktorabläufen war biologisch abbaubar.

Es wird die Hypothese aufgestellt, dass sich die Leistungsfähigkeiten der Anlagen signifikant verbessern würden, wären die Anlagenbelastungen den Auslegungswerten ähnlicher, d.h., wären die maximalen hydraulischen Belastungen geringer und die organischen Belastungen höher. Es wird deshalb geraten, den Fremdwasserzufluss zu minimieren, die Anlagenzulaufkonzentration durch partielle

Grauwasserversickerung zu erhöhen und die Ammonium- und Phoshorkonzentration im Zulauf durch partiellen Urinabschlag zu verringern. Es wird außerdem vorgeschlagen, die hydraulische Aufenthaltszeit in Absetzbecken (settlers) auf zehn Stunden zu begrenzen, um so die organische Belastung der ABRs zu erhöhen.

Ferner wird die Hypothese aufgestellt, dass Anlagen unter höherer Trockenwetterbelastung als bislang angenommen betrieben werden können, da aktiver anaerober Abbau auch unter extremen hydraulischen Belastungen möglich erscheint. Dies könnte zu einer signifikanten Senkung der Baukosten führen.

Die Modellierung anaerober Abbauprozesse mit dem existierenden ADM-3P-Modell bestätigten, dass im Feld beobachtete Schlammakkumulationsraten auf eine aktive Hydrolyse schließen lassen. Das Modell konnte jedoch nicht genutzt werden, um Bezugswerte für den gelösten CSB im Ablauf der Anlagen zu erhalten, da es eine vergleichsweise hohe Sensitivität in Bezug auf die Raten für Methanogenese aufwies. Die allgemein anerkannte Sichtweise ist, dass die Hydrolyse den geschwindigkeitsbestimmenden Abbauschritt bei der anaeroben Behandlung feststoffreicher Abwässer darstellt. Es wird die Hypothese aufgestellt, dass dieses nicht für feststoffakkumulierende Systeme, wie den ABR, zutrifft.

Thesen

- Die durchschnittliche Pro-Kopf-BSB-Last in DEWATS-Einsatzgebieten beträgt **20 bis 40 g BOD_5 cap^{-1} d^{-1}**.
- Der durchschnittliche Pro-Kopf-Abwasseranfall in DEWATS-Einsatzgebieten in Java/Indonesien beträgt **80 l cap^{-1} d^{-1}** und wird nicht signifikant vom Durchschnittseinkommen beeinflusst.
- Der durchschnittliche Pro-Kopf-Abwasseranfall in ariden, sehr einkommensschwachen DEWATS-Einsatzgebieten in Bangalore/Indien ist beträgt **30 l cap^{-1} d^{-1}**.
- Der durchschnittliche Faktor für Zuflussspitzen im Abwasseranfall in DEWATS- Einsatzgebieten in Java/Indonesien, Bangalore/Indien und Durban/Südafrika beträgt **1.9**.
- Die durchschnittliche Pro-Kopf-Ammonium- und Pro-Kopf-Phosphor-Last in DEWATS-Einsatzgebieten in Java/Indonesien, Bangalore/Indien und Durban/Südafrika beträgt **5.6 g NH_4-N cap^{-1} d^{-1} und 0.8 g PO_4-P cap^{-1} d^{-1}**.
- Die durchschnittliche Ammonium- und Phosphor-Ablaufkonzentration von anaeroben DEWATS-Reaktoren beträgt etwa **70 mg NH_4-N l^{-1} und 10 mg PO_4-P l^{-1}**.
- Die durchschnittliche Pro-Kopf-Biogasproduktion von mit kommunalen Abwässern beschickten Biogas-Anlagen beträgt etwa **20 l cap^{-1} d^{-1}**. Die prozentuale Zulaufzusammensetzung aus Schwarz- und Grau-Wasser beeinflusst diesen Wert nicht signifikant.
- Die optimale hydraulische Aufenthaltszeit für mit kommunalen Abwässern beschickten Biogas-Anlagen beträgt etwa **2.5 d**.
- Der durchschnittliche Pro–Kopf-Schlammanfall in ABRs in Indonesien und Indien beträgt etwa **3 bis 8 l cap^{-1} y^{-1}**. Die hydraulische Aufenthaltszeit und die Art der Vorklärung (Absetzbecken oder Biogas-Anlage) beeinflussen diese Werte nicht signifikant.
- Leistungslimitierende Faktoren im Feldbetrieb sind Fremdwassereinfluss, generelle Unterbelastung, organische Unterbelastung und erhöhte Frischwassersalinität in Küstengebieten.
- Die garantierte Abflusskonzentration von anaeroben DEWATS-Reaktoren beträgt **200 mg CSB l^{-1}**, sofern der negative Einfluss von erhöhter Frischwassersalinität ausgeschlossen werden kann.
- Die Uhrzeit an der DEWATS Abflussproben genommen werden, beeinflusst die gemessenen CSB Konzentrationen nicht signifikant.
- Der CSB-Abbau durch ABRs zahlreicher DEWATS ist momentan nicht optimal.
- Der Abfluss anaerober DEWATS-Reaktoren beinhaltet hohe Anteile an biologisch abbaubarem CSB.
- Schlammstabilisierung und Schlammabbau innerhalb der ABRs können momentan als gut angesehen werden.
- AF-Reaktoren tragen in zahlreichen Systemen signifikant zur CSB-Reduzierung bei.
- Die Leistungsfähigkeit von ABRs und AFs würde sich signifikant verbessern, wenn ihre maximale hydraulische Last geringer und die allgemeine organische Last höher wäre und beide Lasten somit der Anlagenauslegung entsprächen.
- ABRs könnten mit höherer hydraulischer Trockenwetterlast betrieben werden als derzeit angenommen, was zu einer signifikanten Reduzierung der Baukosten führen würde.

- Der Feststoffrückhalt durch ABRs sollte durch eine Vergrößerung der letzten Kammer und durch den Einsatz eines Feststoff-Filters an dessen Ablauf verbessert werden.
- Die Modellierung anaerober Abbauprozesse mit dem existierenden ADM-3P-Modell einschließlich seiner Kalibrierung bestätigten, dass im Feld beobachtete Schlammakkumulationsraten auf eine aktive Hydrolyse schließen lassen.
- Die Modellierung anaerober Abbauprozesse mit dem existierenden ADM-3P-Modell einschließlich seiner Kalibrierung kann nicht genutzt werden, um Bezugswerte für den gelösten CSB im Ablauf der Anlagen zu erhalten.
- Die Hydrolyse ist nicht der geschwindigkeitsbestimmende Abbauschritt für feststoffakkumulierende Systeme wie den ABR.

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LIST OF ABBREVIATIONS

ABR	Anaerobic Baffled Reactor
AD	Anaerobic digestion
AF	Anaerobic Filter
BGD	Biogas digester
bio.	billion
BOD_5	Biochemical Oxygen Demand
BORDA	Bremen Overseas Research and Development Organisation
CBO	Community Based Organisation
COD	Chemical Oxygen Demand
COD_{in}	feed COD concentration
COD_p	particulate COD
COD_s	soluble COD
COD_t	total COD
conc.	concentration
CSC	Community Sanitation Centre
CSTR	Completely Stirred Tank Reactor
DALY	Disability-Adjusted Life Year
DEWATS	Decentralised Wastewater Treatment System
EC	Electric Conductivity
Exp. ch.	Expansion chamber (part of a biogas digester)
GDP	Gross Domestic Product
GoI	Government of Indonesia
IWA	International Water Association
JMP	Joined Monitoring Program
M	Mean
MDG	Millennium Development Goals
mio.	million
MO	Micro-organisms
NGO	Non-governmental organisation
O&M	Operation and maintenance
per cap	Per capita
PGF	Planted Gravel Filter
pretr.	pre-treatment
Q	Volumetric flow-rate
rem.	Removal
RSD	Relative Standard Deviation
SD	Standard Deviation
SEM	Scanning Electron Micrographs
S/I	Substrate to inoculum ratio
SBS	School Based Sanitation
SMA	Specific Methanogenic Activity
SMA_{max}	Maximum Specific Methanogenic Activity
SME	Small and Medium Enterprise

SMP	Soluble Microbial Product
sol.	Soluble
SOP	Standard Operational Procedure
SP	Sampling Point
SRT	Sludge Retention Time
SS	Settable Solids
SSS	Small Sewer System
synth.	Synthetic
t	Time
T	Temperature
TS	Total Solids
UA	Uncertainty Analysis
UASB	Up-flow Anaerobic Sludge Blanket (reactor)
UN	United Nations
UNDP	United Nations Development Program
UNICEF	United Nations Children’s Fund
USD	US Dollar
VFA	Volatile Fatty Acids
VIP	Ventilated Improved Pit-latrine
Vol.	Volume
VS	Volatile Solids
VSS	Volatile Settable Solids
WAS	Waste Activated Sludge
WHO	World Health Organisation
WSP	Water and Sanitation Program
ww	Wastewater
WWTP	Wastewater Treatment Plant

LIST OF SYMBOLS

°C	Degree Celsius
cap	Capita
cm	Centimetre
d	Day
g	Gram
h	Hour
kg	Kilogram
km	Kilometre
l	Litre
m	Metre
mg	Milligram
min	Minute
mio.	Million
ml	Millilitre
mm	Millimetre
mS	Milli-Siemens
P	Number of connected people
Q_d	Daily volumetric flow-rate
Q_p	Per capita wastewater production
t_Q	Time of most wastewater flow per day
$v_{up,max}$	Maximum ABR up-flow velocity on one day
$v_{up,mean}$	Average ABR up-flow velocity on one day
y	Year
σ_m	Standard error of mean

1. INTRODUCTION

1.1. The global sanitation crisis

Improper sanitation directly affects public health and is one of the main factors holding back human development. Diarrhoeal disease alone is estimated to account for 4.1% of the total DALY[1] global burden of disease while mostly affecting children in developing countries (WHO, 2014a). It is responsible for 1.8 million deaths every year (WHO, 2014a). The World Health Organization (WHO) further estimates that 88% of that burden is directly attributable to unsafe water supply, poor sanitation and lack of hygiene. Improved sanitation alone would reduce these numbers by one third (WHO, 2014a). Also, poor sanitation, including hygiene, causes at least 180 million disease episodes annually (WSP, 2008). The link between sanitation and other aspects of development has been recognized by the United Nation's Millennium Project Taskforce on Water and Sanitation:

"..increasing access to domestic water supply and sanitation services and improving water resources management are catalytic entry points for efforts to help developing countries fight poverty and hunger, safeguard human health, reduce child mortality, promote gender equality, and manage and protect natural resources. In addition, sufficient water for washing and safe, private sanitation facilities are central to the basic right of every human being for personal dignity and self-respect." (Lenton et al., 2005)

However, up until now the world lives through a sanitation crisis. The Joined Monitoring Program (JMP) from the WHO and the United Nations Children's Fund (UNICEF) publishes annually estimates on the world's sanitation coverage. The latest JMP report from 2013 states that global sanitation coverage in 2011 was 64% with 2.5 billion people not using improved sanitation facilities (national values for selected states are shown in Table 1).

For this estimate the JMP defines "improved sanitation" as being a ventilated improved pit (VIP) latrine, a pit latrine with slab, a composting toilet and flush or pour-flush to either piped sewer system, septic tank or pit latrine. This methodology does not consider whether the wastewater discharged to piped sewer is treated before being released to the environment.

Mara (2003) reports that more than 50% of the world's oceans, rivers and lakes are polluted due to untreated wastewater. South-East Asian countries, for example, are known to have *"'very severe water pollution' for faecal (thermotolerant) coliforms, biochemical oxygen demand (BOD_5) and lead, and 'severe water pollution' for suspended solids"* (UN, 2000). Especially in developing countries the largest sources of water body pollution have been found to be communal rather than industrial (WSP, 2013).

[1] The disability-adjusted life year (DALY) represents a measure of overall disease burden. It is expressed as the number of years lost due to ill-health, disability or early death.

Critics therefore argue that the discharging of untreated communal wastewater does not hygienically separate humans from human excreta and therefore question the validity of JMPs definition of “improved sanitation”. Baum et al. (2013) readopted the methodology followed by the JMP, however only classifying discharge to sewer as “improved sanitation” if it included treatment before release to the environment. By doing so the JMP-estimation for global sanitation coverage in 2010 would have to be dropped from 62% to only 40%. This implies that there were 4.1 billion people across the world that did not have access to improved sanitation facilities in 2010.

Over the last years, the World Bank’s Water and Sanitation Program (WSP) published a number of reports which attempt to quantify the economic losses attributed to lack of sanitation. Table 1 summarizes the results for a number of selected countries and puts them into relation to the countries respective Gross Domestic Product (GDP). The losses consider effects on health (health care costs, income loss due to sickness, premature mortality), water (increased costs of drinking water treatment, reduction of fish-production), environment (loss of productive land), time to access unimproved sanitation and tourism (WSP, 2008). The reports further demonstrate that, by far, costs are highest through health and water and that the financial losses affect mostly poor households and children (WSP, 2008).

The WSP concludes from one study in Central Java (WSP, 2013) that investments into wastewater treatment are outweighed by factor 2.3 by the financial benefits they bring. The WHO even states that “*every USD invested in sanitation translates into an average return of 9 USD”* (WHO, 2007).

Table 1: Improved sanitation coverage and annual loss through inadequate sanitation for selected countries

Country	Indonesia	Philippines	Vietnam	Cambodia	Lao	India	Zambia	Tanzania
Improved sanitation coverage*	59%	74%	75%	33%	62%	35%	42%	12%
Annual loss through inadequate sanitation (USD)#	6.3 bio.	1.4 bio.	780 mio.	450 mio.	193 mio.	53.8 bio.	194 mio.	206 mio.
Annual loss as fraction of country GDP#	2.3%	1.5%	1.3%	7.2%	5.6%	6.4%	1.3%	1.0%

* WHO/UNICEF 2013, # WSP, 2008; WSP, 2011; WSP, 2012; WSP, 2012

The global situation has urged governments and multilateral institutions to raise sanitation higher up on the agenda. For instance, under Target 7c of the Millennium Development Goals (MDG) governments pledge to halve the proportion of people without sustainable access to safe drinking water and basic sanitation by 2015 compared to 1990. More recently in November 2013, the United Nations (UN) General Assembly Third Committee adopted a resolution on “the human right to safe drinking water and sanitation”. Herein all UN member states recognize that the rights to water and sanitation are part of the International Covenant on Economic, Social and Cultural Rights (ICESCR), Convention on the Rights of the Child (CSC) and the Universal Declaration of Human Rights (UDHR). The Committee has therefore confirmed that these rights are legally binding upon states.

But the task of meeting these promises is huge. If the current trend continues, the JMP forecasts that the MDG sanitation target will be missed by a total of half a billion people (WHO/UNICEF, 2013). Adopting the above mentioned definition of “improved sanitation” proposed by Baum et al. (2013), this value rises to 1.9 billion people.

The political will is getting stronger in many countries, of which some are putting considerable efforts into taking on the challenges at hand.

For example, the Government of Indonesia (GoI), in its long term development plan (RPJPN), has set the ambitious target of full access to sanitation by 2019 (SMEC, 2013). From 2010 to 2014 GoI is implementing a program to accelerate sanitation development (PPSP) planning to reach 70 mio. Indonesian citizens without previous access to improved sanitation. The program foresees to reach 5% of this population with centralised and 95% of this population with decentralised solutions (GoI, 2014).

The success of these efforts will certainly also depend on whether the local existing technical and conceptual challenges of wastewater treatment can be overcome. For instance, the implementation in the past decades of large-scale centralised sewer and treatment systems following the western model has shown that this approach does often not meet the requirements posed by the reality in developing and emerging countries. Too high running costs, unstable electric power supply and lack of skilled personnel often lead to under-loaded, badly functioning or downright broken-down systems (Eales, 2008; Kamal et al., 2008; USAID, 2006). Also, the implementation of low-lying canalisation networks into existing urban centres with high population densities such as can be found all over in developing and emerging countries represents a huge financial and logistical hurdle.

Other widely implemented sanitation options are onsite solutions such as different types of latrines (including pit-latrines and household septic tanks (WSP, 2009)). Household latrines, although being heavily relied on, are often not appropriately built and lead, especially in densely populated areas, to widespread ground-water contamination. A correctly built septic tank needs to be regularly emptied and includes a soak-away which requires a certain soil-permeability, minimum ground-water table depth and a minimum distance of about 30 m to the next water source (WHO, 2014b). These requirements can often not be met.

Dry sanitation has many advantages, especially for water-scarce countries, such as low fresh water wastage, low investment and maintenance costs and the possibility of nutrient reuse in agriculture. Simple technical solutions have also been found for regions where populations practice wet anal hygiene. Dry sanitation is however not universally accepted, with flushing toilets often being considered to be superior and a social stigma associated with dry technologies (Duncker and Matsebe, 2008).

Therefore, there is urgent need for the further development of locally adapted concepts for situations in which conventional approaches fail.

1.2. The BORDA-Decentralised Wastewater Treatment Systems (DEWATS) approach

The International Water Association (IWA) has defined decentralised wastewater management as the opposite of centralised wastewater management. Centralised wastewater management is characterised by one wastewater treatment plant (WWTP) for the largest possible confined catchment area in a region. Decentralisation therefore means the break-up of the catchment area into smaller areas (IWA, 2014). Following this definition, the smallest possible decentralised system is an on-site facility. Decentralised Wastewater Treatment Systems (DEWATS) can in principle vary in size and can include any available technology from simple passive anaerobic systems to technically highly complex solutions.

In this thesis the term “DEWATS” refers specifically to passive anaerobic treatment systems going beyond single household on-site facilities, such as implemented by the German non-profit organisation Bremen Overseas Research and Development Association (BORDA).

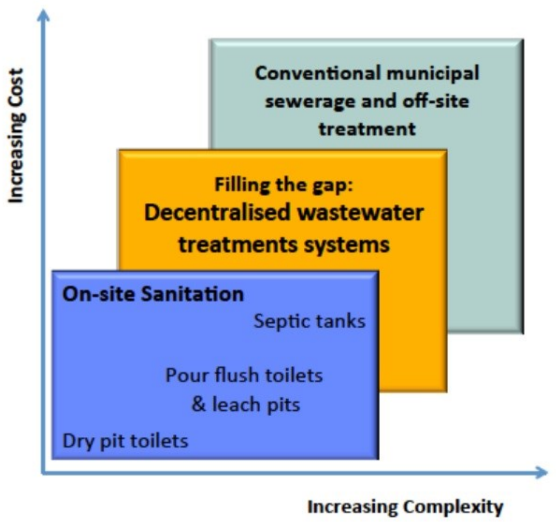

Figure 1: BORDA-DEWATS fill the technology gap between on-site sanitation and centralised treatment (Eales et al., 2013)

For over a decade, BORDA and its local partner non-governmental organisation (NGO) network have been playing a pioneering role in developing and implementing decentralised waterborne sanitation solutions for low income communities in densely populated urban areas. BORDA’s DEWATS approach aims at filling the technology gap existing between on-site sanitation technology and large waterborne centralised systems (see Figure 1). BORDA’s concept of DEWATS is based on the understanding that only a system requiring a minimum of maintenance can be sustainable in the long run. DEWATS technology therefore requires no electricity and pumps and does not contain movable parts. It can be built out of freely available material and with local man-power. Until 2013 BORDA’s partner network has implemented over 1,500 systems world-wide reaching approximately 300,000 people in South Asia, South-East Asia and Southern Africa (personal communication, BORDA). BORDA’s DEWATS approach for communal wastewater treatment is in general “demand-responsive” and “community-based” which means that systems will only be implemented in communities which actively request them and which are willing to do the required maintenance work.

Communal DEWATS are either connected to households by a “Shallow Sewer System” (SSS) or to “Community Sanitation Centres” (CSC) with communal toilets, showers and at times laundry areas. CSCs are also implemented in boarding schools and then termed as “School Based Sanitation” (SBS) systems. “Mixed systems” are SSS additionally connected to a CSC.[2]

Over the last years BORDA-type DEWATS are increasingly being recognized by public authorities as well as bi-lateral and multi-lateral donors as a viable option in the provision of sanitation services to under-privileged population sectors.

The Government of Indonesia for instance regards community-managed DEWATS as being its best available improved sanitation option in selected urban areas until full centralised sewage network and

[2] DEWATS are further being implemented as wastewater treatment for small and medium enterprises (SME) such as hospitals, slaughterhouses and tofu-producers. These DEWATS types, however, are not further discussed in this thesis.

treatment are possible. Part of its current development program foresees to connect 5% of the urban population, or six million people, to DEWATS facilities until end 2014 (Eales et al., 2013).

1.3. Objectives of the thesis

The dimensioning of the plants follows a freely available design-procedure developed in 1998 (Sasse, 1998) based on literature and knowledge available at that time.

Since then, little practical experience has been gathered methodologically on the basic engineering and performance aspects surrounding DEWATS. This is due to the relatively recent history of DEWATS implementation, the limited number of researchers active in the field and the existing challenges encountered during field investigations in project areas. Some of the most pressing engineering questions result from the fact that:

- Little knowledge and field-data are available on key design parameters (such as per capita loads) for low income, urban communities targeted by DEWATS.
- Although known to be very robust, the system's tolerance towards the considerable variations of operational factors it is exposed to in the field is largely unknown.
- The relation between system loading and treatment efficiency remains uncertain. This relation, however, directly influences the dimensioning and therefore the building costs.

The general motivation behind this thesis is the improvement and consolidation of system design by addressing these urgent technical issues. By doing this, this thesis is anticipated to contribute to the understanding of DEWATS field operation and thereby to support the maturation of DEWATS towards an established and well understood sanitation option as part of the solution to the global wastewater crisis.

The thesis focuses on the anaerobic treatment steps of BORDA-type communal DEWATS implemented in tropical regions. Special focus is laid on the operation of the Anaerobic Baffled Reactor (ABR) as the core technology of DEWATS.

The main research questions treated in this thesis are:

- Are the usually estimated parameter values for DEWATS design correct?
- What are the factors limiting the treatment of existing systems?
- How do DEWATS and DEWATS treatment steps (especially ABRs) perform under tropical field conditions in terms of effluent concentration, Chemical Oxygen Demand (COD) removal, sludge stabilisation and sludge activity?
- Can dynamic anaerobic modelling be used to help interpret existing ABR field data by providing benchmark value ranges for the operational parameters "sludge build-up" and "effluent COD_s concentration"?
- What are the implications of these findings on future design?
- What conclusions can be drawn concerning future treatment monitoring methods?
- What are suggested future field investigations?

1.4. Project time line

The investigations presented in this thesis were performed from 2009 to 2013 in Indonesia, India and South Africa. The available data was produced by a large number of people during this period (see Table 2). Project-teams were regularly trained on-site by the author who coordinated the research activities and provided backstopping via the internet over the entire period. A set of standard operational procedures for both field investigations and laboratory investigations was elaborated and continuously updated.

Table 3 indicates which staff members were involved in research coordination, field work or laboratory investigations over the course of the years.

Table 2: BORDA staff involved in research activities

India		Indonesia		Staff involved in “Screening study” (Chapter 4)
Name	**Acronym**	**Name**	**Acronym**	**Name**
Alexandro Miller	AM	Adita Yuniati Puspitasari	AYP	Adita Yuniati Puspitasari
Anne Bugey	AB	Agus Suroyo	AS	Anang Bagus Setiawan
Eva Mary	EM	Anggi Putu Wiratma	APW	Franziska Kny
Jan Knappe	JK	Aris Tri Susilo	ATS	Gressiadi Muslim Muttaqin
Kantaraj Antoni	KA	Arnoldy Satya Bhakti Toad	ASBT	Hendro Saputro
Lorenz Streckmann	LS	Esti Sri Hardianti	ESH	Ilona Lender
Nicolas Reynaud	NR	Ferika Rahiem	FR	Maren Heuvels
Parashivamurti	PSM	Gerrit Meyer-Rieke	GM	Michael Seibold
Rajesh Pai	RP	Hendro Saputro	HS	Muhammad Zamroni
Rajesh Shenoy	RS	Ikatri Wulandari	IW	Nicolas Reynaud (Project leader)
Rohini Pradeep	RPr	Ilona Lender	IL	Noka Destalina
Sachin M.H.	SM	Janeska Mahardika	JM	Rosa Bennemann
Saijyoti Vulimiri	SJV	Karin Borkmann	KB	Septa Nugroho
Santosh Ramaiah	SR	Maike Gärtner	MG	Tri Wahyudi Purnomo (Project leader)
Susmita Sinha	SS	Michael Seibold	MS	
Timmesha R.	TR	Nicolas Reynaud	NR	
Venkatesh	V	Nur Ida Fitrianto	NIF	
		Rosa Bennemann	RB	
		Sri Peni Wijayanti	SPW	
		Timbul Santosa	TS	

Table 3: Staff responsibilities and research contributions over the years (acronyms as defined in Table 2)

Interregional research coordination: NR	India				Indonesia			
	2010	**2011**	**2012**	**2013**	**2010**	**2011**	**2012**	**2013**
Main regional research coordination	RP/ RPr	RP/ RPr	SS/ RPr	SS/ RPr	FR/ SPW	AYP	AYP	AYP
Research coordination	AM	AB		JK	KB	RB	MS	MG
Field work	AM	AB	EM	EM	ESH	ASBT	AYP	AS
	KA	NR	KA	JK	GM	AYP	HS	ATS
	NR	RPr	LS	NR	HS	GM	IW	AYP
	RPr	SM	NR	RPr	JM	HS	JM	HS
	TR	TR	RPr	RS	KB	IL	MS	IW
	V	V	SR	SJV	NR	IW	NIF	MG
			SM	V		JM	NR	NIF
			SJV			NR	TS	NR
			TR			RB		
			V					
Laboratory work	AM	AB	EM	EM	IW	IW	IW	IW
	RPr	PSM	LS	JK		NIF	NIF	MG
		RPr	RPr	RPr		RB		NIF
		SM	SM	RS				NR
			SJV	SJV				

Figure 2 and Figure 3 present the dates on which monitoring activities (sludge activity measurements, gas production measurements, field sampling including temperature, alkalinity, pH, conductivity, turbidity, COD and BOD_5 investigations, sludge height measurements and flow measurements) were performed at DEWATS in Bangalore/India and at DEWATS in Yogyakarta/Indonesia.

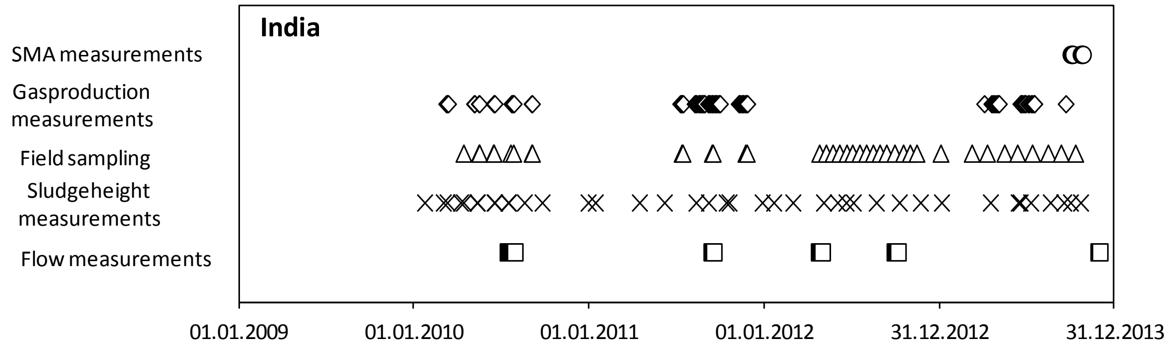

Figure 2: Monitoring activities performed in India, SMA = Specific Methanogenic Activity

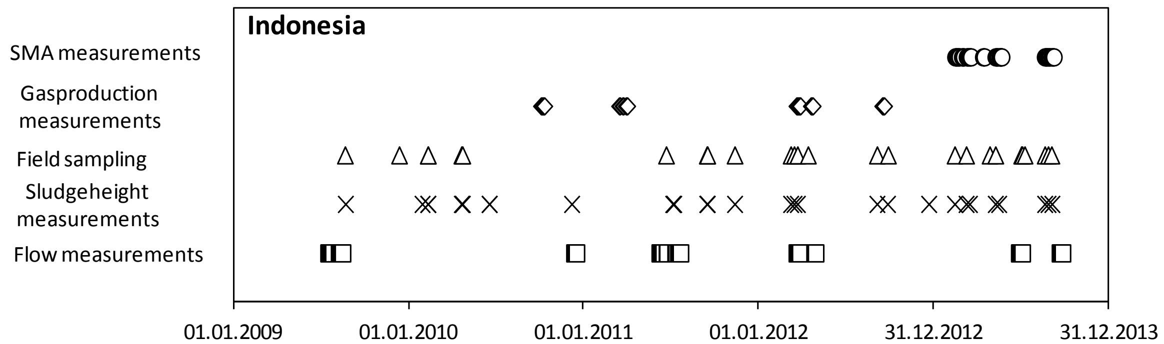

Figure 3: Monitoring activities performed in Indonesia

Field investigations in NewlandsMashu/Durban, South Africa, presented in this thesis are effluent COD concentration measurements and flow measurements performed from 24.02.2012 to 15.08.2012. Field investigators were Lars Schöbitz, Nicolas Reynaud, Phatang Sananikone and Dr. Sudhir Pillay. Laboratory measurements were carried out by Nicolas Reynaud and Dr. Sudhir Pillay.

1.5. Factors surrounding research in developing countries

Field research in the project regions was handicapped by a number of factors and it is important to interpret the available data in the light of the circumstances in which it was produced. These difficulties are due to the very nature of DEWATS (being comparably small these systems are exposed to large wastewater fluctuations) and to the field reality in tropical and developing countries. Difficulties arose due to:

- high fluctuation of feed quality and quantity due to small number of connected households
- tropical rains affecting sampling and system treatment
- wide geographical spread of systems
- general logistics and transportation to remote sites
- high staff turn-over
- inaccessibility of the reactor chambers due to blocked man-holes
- limited amount of hardware and chemicals for analytical investigations
- analytical uncertainties due to low quality standards in most commercial laboratories
- general lack of reliable data
- difficulty to conduct flow measurements
- intermittent availability of electric power
- partly incomplete design documentation of facilities
- partly unknown history of plant operation and performance
- partly surprising changes of treatment-affecting factors (such as loading, breakages, discharge of toxic chemicals to the systems)

1.6. Organisation of the thesis

This thesis contains four chapters (Chapters 4, 5, 6 and 7) which in themselves can be read as separate studies. Each contains its own data presentation, discussion and conclusion. This was done because of the varying characteristics of the datasets on which the chapters are based and the different aspects of DEWATS operation treated by the chapters. A number of results overlap thematically and each chapter contributes to answering the overall research-questions. The last chapter therefore re-evaluates and summarizes all outcomes in the light of the main research-questions.

Chapter 1 introduces the global sanitation challenges and the role DEWATS may play in these. Objectives of the study are presented.

Chapter 2 compiles literature on DEWATS treatment modules, anaerobic digestion and communal wastewater characteristics in developing countries.

Chapter 3 describes the investigation methods and data interpretation methods used for this thesis.

Chapter 4 compiles and discusses available field-data on ***general*** design relevant and operation relevant parameters. These parameters are: per capita wastewater production of communities connected to DEWATS and hydraulic peak flow factors, DEWATS effluent characteristics and their variation over time, biogas-production of biogas-digesters (BGD) and sludge build-up rates in ABRs.

Chapter 5 presents and discusses field-data gathered at 108 DEWATS during a once-off monitoring campaign performed across the islands of Sumatra, Java and Bali from September to November 2011. This chapter presents an overview of how DEWATS perform ***broadly***. It discusses available information on factors potentially affecting system performance, attempts to relate system loading to effluent quality and provides a broad view on which effluent concentrations can be expected from anaerobic DEWATS reactors under current operation conditions.

Chapter 6 presents and discusses more ***in-depth*** performance data from four case studies gathered over four years. This section particularly focuses on the effect of system loading on reactor operation in terms of COD removal, sludge stabilisation and sludge activity and extrapolates the implications of these findings on future reactor design and operation. The presented investigations focus on the DEWATS module ABR but also consider the DEWATS pre-treatment modules (biogas digester (BGD) and settler) and the Anaerobic Filter (AF).

Chapter 7 presents the use of a dynamic anaerobic digestion model to support the interpretation of the in-depth field data discussed in Chapter 6. The latter was handicapped by the lack of treatment performance data of other full-scale ABRs operating under similar field conditions (notably sludge accumulation rates and effluent soluble COD (COD_s) concentrations). The presented modelling exercises were therefore driven by the necessity to obtain benchmark value estimations for the operational parameters *sludge build-up* and *effluent COD_s concentration.* Field measurement results are compared to these benchmark values in order to assess the activity of anaerobic digestion in the systems. The chapter further discusses model predictions on treatment efficiency increase depending on the Organic Loading Rate (OLR). It also summarizes potential further applications of the model concerning ABR design and operation. Finally, future investigation needs arising from the model exercise outcomes are outlined.

Chapter 8 summarizes the results from Chapters 4 to 7 based on the main research questions listed in Section 1.3.

1.7. Publications resulting from this study

Table 4 compiles the publications resulting from this study in which the author was involved. Electronic copies of these publications can be accessed as explained in Appendix A6.

Table 4: Publications resulting from this study

Reference	Publication type	Conference	Role of N. Reynaud	Presented at conference by
Reynaud et al. (2009)	Conference paper	IWA, Kathmandu	First author	Nicolas Reynaud
Reynaud et al. (2010b)	Conference paper	WISA, Durban	First author	Dr. Sudhir Pillay
Reynaud et al. (2010a)	Conference poster	IWA, Surabaya	First author	Nicolas Reynaud
Bugey et al. (2011)	Conference paper	IWA, Manila	Second author	Susmita Sinha
Miller (2011)	M.Sc. Thesis		Mentoring	
Reynaud and Buckley (2011)	Conference paper	IWA YWPC, Pretoria	First author	Nicolas Reynaud
Reynaud et al. (2011)	Conference paper	IWMC, Perth	First author	Nicolas Reynaud
Pillay et al. (2012)	Conference paper	WISA, Cape Town	Second author	Dr. Sudhir Pillay
	Conference poster	WISA, Cape Town	First author	Nicolas Reynaud
Pradeep et al. (2012)	Conference paper	IWA, Nagpur	Second author	Rohini Pradeep
Reynaud et al. (2012a)	Conference paper	IWA, Nagpur	First author	Nicolas Reynaud
Reynaud et al. (2012b)	Conference paper	IWA, Nagpur	First author	Nicolas Reynaud
Pillay et al. (2014)	WRC-Report		Second author	

2. LITERATURE REVIEW

2.1. Common BORDA-DEWATS treatment modules

2.1.1. Combination of modules

The typical DEWATS setup is modular and consists at least of a primary treatment unit, which can be a biogas digester or settler, and a secondary anaerobic treatment unit, generally an anaerobic baffled reactor (ABR) combined with an anaerobic filter (AF). Tertiary treatment is included in some systems in the form of a planted gravel filter (PGF). In some cases post-treatment occurs in an aerobic polishing pond. The exact combination and seizing of modules varies between systems and is adapted to cater to the individual situations and the requirements of the respective communities.

2.1.2. DEWATS primary treatment

2.1.2.1. Biogas digester (BGD)

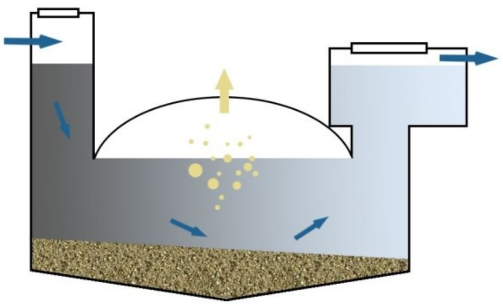

Figure 4: Cross section of a typical BORDA fixed dome biogas digester (courtesy of BORDA)

BORDA DEWATS biogas digesters are fixed dome digesters without external mixing and are designed for hydraulic retention times (HRT) of 24 h to 48 h. Depending on the implementation, they are fed with raw non-screened wastewater including grey- and black-water or purely black-water.

Little literature could be found on the treatment efficiency of biogas digesters treating communal wastewater. The focus of most papers lies on biogas-production and co-digestion of wastewater and manure or organic household waste.

Hamad et al. (1981) and Polprasert et al. (1986) for example reported organic matter to methane conversion of 35% to 50% at HRTs of 38 d to 95 d. Mang and Li (2010) mentioned BOD_5 reduction of 25% to 60% in digesters treating black-water with HRTs of at least 20 d.

Since biogas digesters are used as the primary treatment step within the BORDA design and are dimensioned with a similar HRT as settlers (see following paragraph), similar treatment efficiencies are assumed in the following.

2.1.2.2. Settler/septic tank

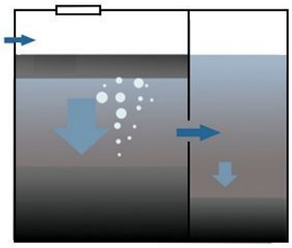

Figure 5: Cross section of a typical setter or septic tank (courtesy of BORDA)

The second technical option used for pre-treatment in the BORDA DEWATS design is the settler or septic tank. When representing the only treatment step it is designed with an hydraulic retention time (HRT) of approximately 24 h (Sasse, 1998). When representing primary treatment further followed by secondary treatment it should be designed with significantly lower HRT of approximately 2 h (Sasse, 1998).

Foxon (2009) concluded in her review on septic tanks that their treatment efficiency is generally 30% to 50% BOD_5 reduction at 48 h HRT treating domestic wastewater. Bench-scale investigations by Nguyen et al. (2007) confirmed this.

Koottatep et al. (2004) observed 71% COD removal at 48 h HRT during their investigations.

2.1.3. DEWATS secondary treatment

2.1.3.1. Anaerobic Baffled Reactor (ABR)

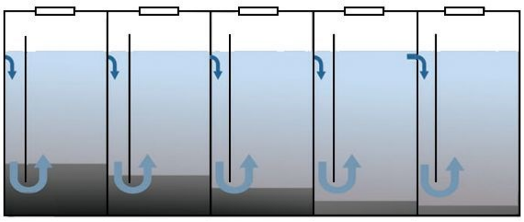

Figure 6: Cross section of a five chamber ABR (courtesy of BORDA)

The anaerobic baffled reactor (ABR) design with alternating standing and hanging baffles forces the wastewater to flow repeatedly through settled sludge, thereby increasing the contact between organic pollutants and biomass. It is often referred to as the core treatment step of DEWATS. Further details are discussed under Section 2.3.

2.1.3.2. Anaerobic Filter (AF)

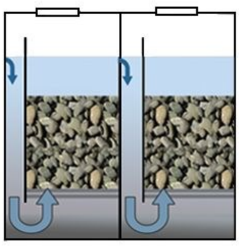

Figure 7: Cross section of a two chamber AF (courtesy of BORDA)

Anaerobic filters (AF) are fixed-bed reactors, designed to receive wastewater with low concentrations of settleable solids and designed to biodegrade non-settleable and dissolved organics. The wastewater flows through the filter voids, resulting in close contact between the biomass fixed on the filter-material (rocks, gravel) and the suspended and dissolved substrate. Further details are discussed in Section 2.4.

2.1.4. Planted Gravel Filters (PGF)

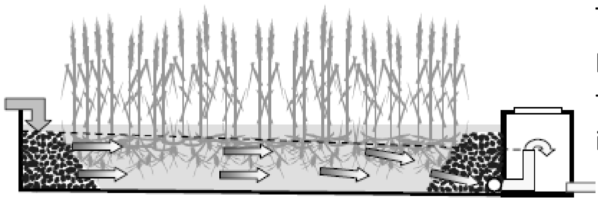

Figure 8: Cross section of a PGF (courtesy of BORDA)

The planted gravel filter (PGF) further reduces pathogens, organic pollutants and nutrients from the secondary treatment effluent. This technology is not further discussed in this thesis.

2.2. Anaerobic digestion

Anaerobic digestion (AD) is one of the main treatment mechanisms in all DEWATS modules discussed in this thesis. During AD organic matter is converted to CO_2 and CH_4 in a series of interrelated biochemical processes. About 5% of the COD decrease manifests as biomass COD (Tchobanoglous et al., 2003).

Anaerobic digestion is generally described as four major interrelated sub-processes: hydrolysis, acidogenesis, acetogenesis and methanogenesis (see Figure 9). Each of these processes is mediated by different microbial groups of which the characteristics and favourable living conditions vary.

During ***hydrolysis***, complex organic polymers, such as carbohydrates, proteins and lipids are broken down by hydrolytic micro-organisms (MO) to simple sugars, amino acids and long chain fatty acids. ***Acidogenesis*** refers to the fermentation of these simple sugars and amino acids to simple organic acids. The *acetogenic* MOs further degrade the simple organic acids to acetic acid during the so called ***acetogenesis***. This fermentation step has little effect on the pH. During the last step, the **methanogenesis**, methane is either produced by the slow-growing *hydrogenotrophic methanogens* which use hydrogen and carbon dioxide as substrate, or by a group of archea called *acetoclastic methanogens* which converts acetic acid under strictly anaerobic conditions to methane. This last MO group accounts for up to 70% of the methane production (Seghezzo, 2004) and for most of the conversion of COD.

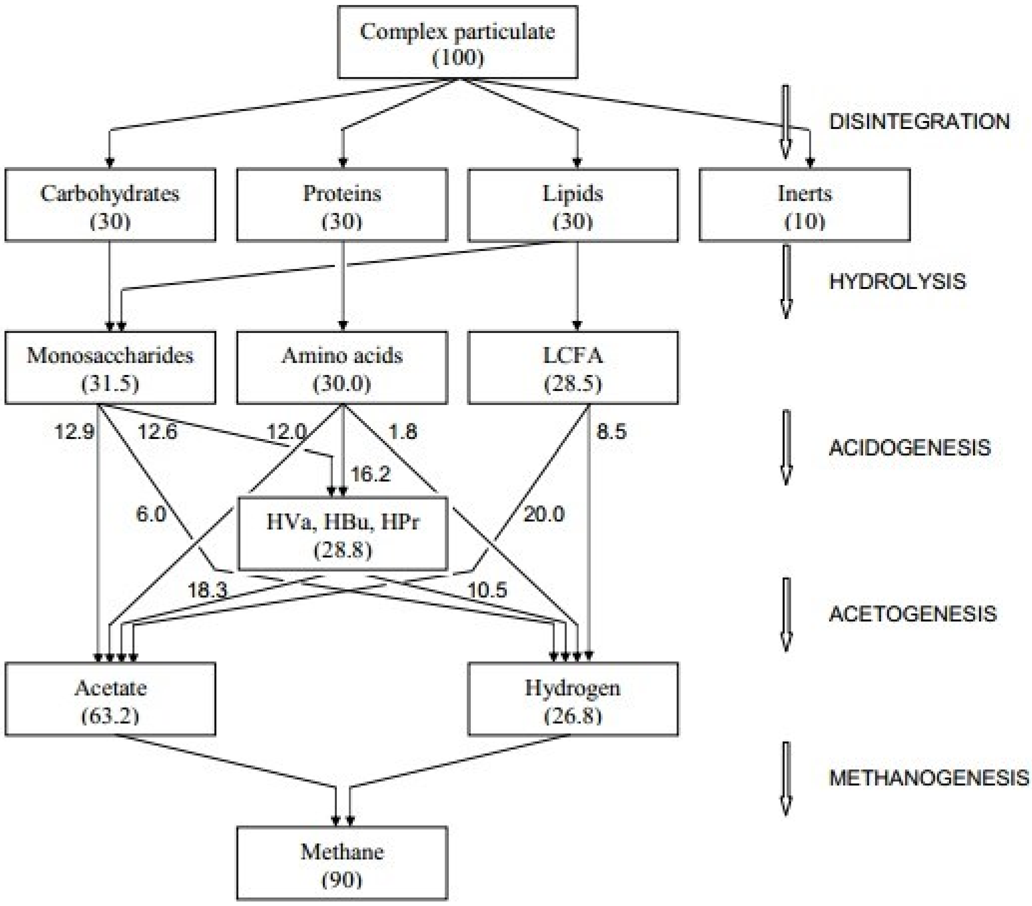

Figure 9: Anaerobic biological degradation, adapted by Foxon (2009) from Batstone et al. (2002). Figures in brackets indicate COD fractions

The first 2 processes produce acid whereas methanogenesis consumes it and generates alkalinity. *Methanogens* are particularly pH-sensitive resulting in methanogenesis being inhibited at a pH below 6.5 if too much acid is generated during the former sub-processes. This inhibition would cause a further drop of pH and therefore a complete souring of the system, as *methanogens* represent the only acid-consuming MO group. Good buffering and a high enough level of alkalinity are therefore important to prevent this precarious balance of acid production and acid consumption from tipping towards complete inhibition of the anaerobic digestion.

2.3. The ABR treating communal wastewater under mesophilic conditions

2.3.1. Introduction

The Anaerobic Baffled Reactor (ABR), or Baffled Septic Tank, was developed by McCarty and co-workers at Stanford University in the early 1980s (Bachmann et al., 1985). It was then implemented widely in China before knowledge about its effectiveness spread further. The ABR has been described as a series of up-flow anaerobic sludge blanket reactors (UASBs) reducing TS and organics in the wastewater.

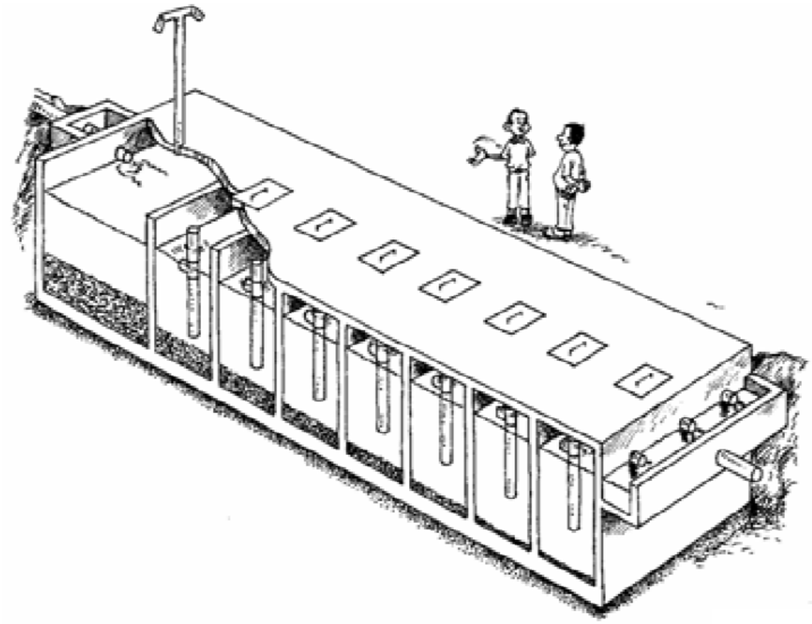

Figure 10: Cross section of an ABR design with six chambers including a two chamber settler (courtesy of BORDA)

Two mechanisms are responsible for the treatment properties of ABRs: anaerobic digestion and solid retention (Foxon, 2009). Anaerobic digestion happens through the contact of organic pollutants in the wastewater and the biomass of the sludge, suspended or settled inside the reactors. Solid retention takes place through the settling of solids inside the up-flow area of the reactors. The rate limiting step of anaerobic digestion of wastewater with high solids content, such as communal wastewater, is generally regarded to be the hydrolysis (Sotemann et al., 2005).

A detailed review of the ABR was published in the late 1990s (Barber and Stuckey, 1999). Table 5 lists the advantages of the technology.

Table 5: Advantages of the ABR adapted from Barber and Stuckey (1999)

Advantages
Construction
Simple design
Neither moving parts, nor pumping or electricity are required
No mechanical mixing
Inexpensive to construct
High void volume
Reduced clogging
Reduced sludge bed expansion
Low capital and operating costs
Biomass
No requirement for biomass with unusual settling properties
Low sludge generation
High solids retention times
Retention of biomass without fixed media or a solid-settling chamber
No special gas or sludge separation required
Operation
Low HRT
Intermittent operation possible
Extremely stable to hydraulic shock loads
Protection from toxic materials in influent
Long operation times without sludge wasting
High stability to organic shocks
Can treat a large range of wastewater concentrations

Research on laboratory or pilot scale ABR treating communal type wastewater has since been reported from England, South Africa, Germany, India, Nepal, Vietnam, Thailand and China.

A very large number of implementations exist in China. About 120,000 decentralised systems financed through the Chinese Rural Energy Office and including ABR technology have been recorded until 2003 by the Biogas Institute of the Ministry of Agriculture (BIOMA) (Panzerbieter et al., 2005). The real number of implementations however is certainly larger but no statistics for small decentralised systems in China exist to date.

In Tenjo, Columbia (population < 2,500 inhabitants) an ABR system consisting of two reactors with five chambers treats a combined stream of commercial dairy waste and communal wastewater (Orozco, 1997).

"Rotaria Energie und Umwelt-technik GmbH" have implemented approximately 40 communal wastewater treatment systems in South America in which the ABR functions as a pre-treatment step which is followed by a planted gravel filter (personal communication Rotaria). Other companies also implement ABRs in Brazil. Two firms "AquaVerde" and "Conviotec" are currently using ABR technology in Germany. Engineers at "AquaVerde" base their designs on the same procedure as BORDA.

2.3.2. Factors influencing the communal ABR performance in warm climates

Studies have noted that in the case of anaerobic treatment of low strength wastewater the reactor setup needs a high solid retention time and the required reactor volume is determined by the hydraulic rather than the organic load (Lettinga and Pol, 1991). Bischofsberger et al. (2005) however also mentioned that the feed concentration in itself is an important factor to be considered: although anaerobic technology can be used for a wide range of organic loads it is more efficient at high loads and COD feed concentrations should be at least 400 mg l^{-1}. Another important factor is the up-flow velocity due to its direct influence on solid retention (Foxon, 2009; Sasse, 1998).

Foxon (2009) also mentioned the influence of raw-water alkalinity on the system pH and therefore on the establishment of a stable anaerobic microbiological population.

Domestic wastewater flows are inherently highly variable both in terms of quantity and quality (Friedler and Butler, 1996) and increasingly so for smaller systems.

Anaerobic reactors are affected by such variations but the effect depends on the type, magnitude, duration and frequency of variations. The response of the system could be the accumulation of VFA, drop of pH and alkalinity, sludge washout, change in biogas production and composition and decrease in performance (Leitao et al., 2006b). It is therefore important to understand the effect of average hydraulic and organic load as well as peak loads on the treatment performance of anaerobic reactors.

2.3.3. ABR design tool

Sasse (1998) contains an open source ABR design tool which is used for all BORDA ABR designs. It predicts the ABR treatment efficiency and effluent COD and BOD_5-concentration depending on a number of functions. These account for the influence of the following parameters: feed concentration, organic loading rate, hydraulic retention time, number of chambers and temperature. Each function specifies for each parameter-value a certain factor. The treatment efficiency is then calculated by multiplying all five factors. The tool further specifies that the maximum up-flow velocity on one day $v_{up,max}$ should be kept below a maximum value. The design tool input parameters are: per capita wastewater production (Q_p), per capita BOD_5 load, number of connected people (P), number of up-flow chambers and time of most wastewater flow during an average day (t_Q). ABRs are generally designed with four to six chambers. The "time of most wastewater flow" is generally set between 8 h and 12 h. The peak up-flow velocity is calculated with the following equation with A_{ABR} representing the area of one ABR chamber:

$$v_{up,max} = \frac{P*Q_p}{t_Q*A_{ABR}} \qquad \textit{Equation 1}$$

Literature generally mentions the average up-flow velocity ($v_{up,mean}$) which is a special case of Equation 1 where flow is constant flow over the day and t_Q therefore equals 24.

Sasse (1998) mentions that the functions on which the design calculations are based were derived from scientific publications, handbooks and personal experience. However no references are cited. The author also cautions that this body of data and information, although representing the best knowledge at the time, is rather weak. He therefore suggests that users modify the functions when more experience and knowledge is available.

BORDA published a new version of the book in 2009 (Gutterer et al., 2009) but the initial design calculations by Sasse (1998) have until now remained for the very most part unchanged[3].

2.3.4. Literature on ABR treatment: review objectives

The literature is reviewed with the objective of compiling and integrating existing knowledge on ABR performance under mesophilic conditions (20°C to 32°C) with low strength wastewater feed. This is

[3] Some minor changes have been applied in Gutterer et al. (2009) to the functions predicting reactor treatment, however without significantly affecting the calculation-outcome. The main change concerns the proposed design value for maximum up-flow velocity inside the ABR which was lowered from 2 m h^{-1} to 1 m h^{-1}.

done with a focus on the effect of the main design parameters hydraulic load, organic load and ABR compartment number on the treatment processes COD-retention and COD-digestion. Indicators for COD retention are the effluent COD fractions and reports on sludge washout. Indicators for digestion are the specific biogas-production, sludge activity and specific sludge built-up rate. The outcomes are compared to the existing BORDA ABR design based on Sasse (1998). Knowledge gaps are identified and a basis of comparison for BORDA DEWATS ABR field investigation data is created.

The literature review is therefore done in order to answer the following questions:

From previous literature...

- ...how do ABR systems generally perform?
- ...what is the influence of organic loading rates on the ABR treatment processes?
- ...what is the influence of hydraulic loading rates on the ABR treatment processes?
- ...what is the influence of shock loads on the ABR treatment processes?
- ...what is the role of the ABR compartmentalisation?

Very little information is available on full or pilot-scale ABR implementations and most studies are based on laboratory-scale research.

2.3.5. Investigations on laboratory-scale ABRs

2.3.5.1. Literature selection

Literature selection criteria for this review chapter are:

- Study on laboratory-scale ABR technology
- Low COD concentration feed wastewater (150 mg l^{-1} to 2,000 mg l^{-1})
- Mesophilic conditions (20°C to 32°C)

During the last ten years a large body of literature on this topic has been produced in China. Most papers however are not available in English. This chapter includes information on eleven translated Chinese papers[4]. Their relevance to the topic was identified either through their English abstract or title. A more thorough review of Chinese papers was not possible due to cost and time constraints but might yield further helpful information in future.

2.3.5.2. Available literature and general performance of ABRs

Table 6 summarizes the performance data presented in the reviewed literature which covers a large range of organic and hydraulic loading rates. Reported treatment efficiencies were generally between 70% and 90% COD_t removal.

Available information on the effect of treatment influencing parameters on COD removal and the processes digestion and retention will be presented in the following sections.

[4] Translations were done by an external consultant for BORDA and can be accessed as explained in Annex A6.

2.3.5.3. Methodological limitations of published research and processing of relevant literature

A number of methodological limitations within the published studies became apparent during the literature review and have to be considered in the following sections.

In most studies ABRs were seeded with highly active sludge from high rate reactors and the tests were often run directly after. It is questionable how representative these studies are for normal ABR operation since they inherently assume that highly active MO populations establish inside an ABR fed with low concentrated wastewater. Sludge characteristics are bound to change through adaptation to their new environment (Krishna and Kumar, 2008; Xin et al., 2005). Krishna and Kumar (2008) mentioned a 250 d period after start-up in order to attain constant soluble COD effluent concentrations. COD_p concentration had reached a constant value after 100 d but constant biogas-production was only attained after approximately 200 d. Bodkhe (2009) mentioned a period of 90 d to reach stable treatment, however without seeding.

The effect of the main three treatment-influencing parameters HLR, v_{up} and organic loading rate (OLR) were generally coupled in the reviewed studies. The reason is that mostly the feed flows were increased while feed concentrations remained constant.

In research focusing on different loading rates, changes of loading rates should only be initiated after stabilisation of effluent concentrations and treatment efficiency. Bodkhe (2009) reported a period of more than two weeks after loading change for stable conditions to establish. This was confirmed by Intrachandra (2000) (however through tests run with soluble wastewater). In the reviewed works constant operating conditions were maintained between only a few and up to 50 d. Especially Chinese authors often reported very short constant operation periods of less than 15 d without providing proof that the effluent concentration had reached a constant level. The conclusions of such investigations have to be used with caution.

Many authors used synthetic wastewater in order to maintain constant feed characteristics and because communal wastewater is difficult to procure in sufficient quantities over longer testing periods. In some cases the synthetic wastewater was complex, containing solids, in other cases purely soluble. The use of purely soluble organic feed does of course not take into account the various influences of particulate wastewater components on the treatment of communal wastewater. Some authors used sewage and keep the feed COD_t concentration constant by dosing soluble substrate such as glucose. In these publications the amount of substrate added is not specified but it is assumed that this type of feed had a solid content not comparable to communal wastewater.

Studies done with complex wastewater are therefore prioritized in this review in order to draw conclusions on communal ABR application. In case they provide too little or no information on a certain topic, publications based on soluble wastewater are used and declared as such.

Table 6: Performance data on bench-scale ABRs treating low strength ww under mesophilic conditions, contains calculated results, data at times derived from graphs

Substrate	Volume[§] l	Chambers	COD in mg l^{-1}	HRT h	v_{up} m h^{-1}	OLR kgCODm^{-3}d^{-1}	COD out mg l^{-1}	COD removal %	Reference
Synth. grey-water	8	6	438 - 492	48 - 84	n.a.	0.1 - 0.3	109 - 143	71 - 75	Witthauer and Stuckey (1982)
n.a.	350	2°	750	15	n.a.	1.2	170	75	Vincenzi (1989) quoted by Garuti
Domestic & industrial ww	350	2°	590	41490	n.a.	2.2	260	56	Garuti et al. (1992)
Slaughterhouse ww	5.2	4	730	3 - 27	n.a.	0.7 - 2.1	80 - 130	75 - 89	Polprasert et al. (1992)
Synth. sol. ww	16.2	5	1200	24	n.a.	1.2	80	93	Dai et al. (2000)
Synth. sol. ww	4.1	3	500 - 1000	6 - 24	0.04 - 0.16	0.5 - 4.0	35 - 80	90 - 95	Intrachandra (2000)
Semi-skimmed milk	10	8	500	1 - 40	0.03 - 1.03	0.3 - 9	25 - 300	40 - 95	Langenhoff et al. (2000)
Colloidal dog food and rice	10	8	500	6 - 40	0.03 - 0.2	0.3 - 2	50 - 100	80 - 90	Langenhoff et al. (2000)
Synth. sol. ww	15	3	300 - 400	12 - 24	n.a.	0.4 - 0.7	32 - 45	87 - 91	Manariotis and Grigoropoulos (2002)
Black-water	n.a.	5*	500	84	n.a.	n.a.	180	64	Nguyen et al. (2003)
Domestic ww + black-water	40	4^	1970	24 - 48	0.03 - 0.07	1 - 2	236 - 315	84 - 88	Koottatep et al. (2004)
Domestic & synth. sol. ww	15	6	150 - 850	3 - 12	n.a.	0.3 - 6.8	56 - 89	50 - 93	Shen et al. (2004)
Synth. sol. ww	9.9	4	470 - 514	4 - 10	n.a.	1.1 - 2.9	93 - 144	72 - 80	Xin et al. (2005)
Domestic & synth. sol. ww	15	6	500	3 - 12	n.a.	1 - 4	50 - 70	85 - 90	Chen and Shen (2006)
Synth. sol. ww	9.9	4	500	4 - 7	n.a.	1.7 - 3	110 - 170	66 - 78	Hu et al. (2006b)
Synth. sol. ww	9.9	4	500	7	n.a.	1.7	95	81	Hu et al. (2006a)
Pretreated restaurant ww	31.7	6	900 - 1500	8 - 12	n.a.	1.8 - 3	80 - 320	78 - 91	Hu and Yin (2007)
Synth. sol. ww	9.6	4	500	7	n.a.	1.7	100	80	Liu et al. (2007)
Diluted black-water	283	6[$]	500	12 - 72	0.04 - 0.25	0.2 - 1	120 - 210	58 - 76	Nguyen et al. (2007)
Domestic ww	17	6[#]	305	18 - 48	0.05 - 0.13	0.2 - 0.4	87 - 95	69 - 79	Feng et al. (2008a)
Synth. complex ww	10	8	500	6 - 20	0.1 - 0.3	0.6 - 2	38 - 57	89 - 93	Krishna and Kumar (2008)
Slaughterhouse ww	15	4	2750	15 - 28	n.a.	2.4 - 4.4	165 - 330	88 - 94	Nie et al. (2008)
Domestic & synth. sol. ww	300	6	300	12 - 24	n.a.	0.3 - 0.6	100	67	Yang et al. (2008)
Domestic ww	32	9	400	3 - 144	0.02 - 0.9	0.1 - 3.2	25 - 140	65 - 94	Bodkhe (2009)
Synth. sol. ww	10	8	500	8 - 10	0.2 - 0.25	1.2 - 1.5	47 - 50	90	Krishna and Kumar (2009)
Domestic ww	15	5	680	8 - 24	0.05 - 0.16	0.7 - 2.0	105 - 227	67 - 85	Nasr et al. (2009)
Synth. sol. ww	92	4^	500 - 1400	48	0.05	0.3 - 0.7	40 - 65	92 - 96	Sarathai (2010)
Domestic ww	92	4^	1400	48	0.05	0.7	30	98	Sarathai (2010)
Synth. sol. ww	9	4	1530	2 - 20	n.a.	2 - 18	119	92	Yuan et al. (2012)
Synth. sol. ww	15	5	1700	24	0.08	1.7	160	91	Peng et al. (2013)

[§] total active ABR volume; °ANANOX process with 3rd chamber anoxic chamber; * 2 settlers. 2 ABRs. 1 ABR with carrier material; ^ 1 settler, 3 ABRs; [$] Reactors are up-flow cylinders; [#] ABR with bamboo carrier material; ww = wastewater; sol. = soluble; synth.= synthetic

2.3.5.4. Influence of organic load on treatment processes

This section summarizes the published observations on the effect of the OLR variation (at constant HRT) on the ABR treatment processes. Information on this topic was found only within studies on soluble synthetic wastewater.

Sarathai (2010) varied the COD feed concentration from 480 mg l^{-1} to 1,400 mg l^{-1} while maintaining a constant HRT of 48 h for periods of 30 d per loading rate. The observed treatment efficiencies during the different loading rates were very similar (94% to 95%).

Results published in Shen et al. (2004) confirm the above: COD feed concentrations of 550 mg l^{-1} and 850 mg l^{-1} led to COD reductions of 90% and 95% respectively. Lower COD feed concentrations of 150 mg l^{-1} and 350 mgl^{-1} however induced a drop of COD removal to 50% and 80% respectively.

Intrachandra (2000) performed a number of acetogenic and methanogenic activity tests on sludges exposed to different OLRs. Unable to quantify this, the researcher observed that increased OLR led to a shift of sludge activity to the rear compartments. The system seemed to have no difficulties in adapting to double OLR.

Nie et al. (2008) reported that doubling the OLR while keeping the HRT constant lead to a slight treatment efficiency decrease in the first chamber. Overall efficiency however remained constant.

2.3.5.5. Influence of hydraulic loading on general COD reduction

This section summarizes the available information on the effect of hydraulic loading on COD reduction in ABR technology. No literature was found on the effect of hydraulic load decoupled from the OLR. All available data is from studies with complex feed where the COD concentration is kept constant but the feed flow is increased. A change in HRT therefore always represents a change in OLR. The literature presented in the previous section however indicates that OLR variations within ranges typical for communal wastewater and with COD feed concentrations of at least 500 mg l^{-1} have rather negligible effects on the treatment. In this and the following two sections, observed changes in treatment are therefore linked to variations in hydraulic loading rate rather than to variations in organic loading rate.

Figures 3 and 4 show the total COD removal observed in literature in relation to HRT and average up-flow velocity. The dotted line represents the predicted Sasse design COD removal when varying the number of connected people but keeping the ABR size constant. It was computed by using the following standard input parameter values: per capita wastewater production (Q_p): 100 l cap^{-1} d^{-1}, per capita BOD_5 load: 60 g cap^{-1} d^{-1}, number of connected people (P): 200, time of most wastewater flow (t_Q): 10 h, temperature: 28°C and number of ABR chambers: five. COD feed concentration after pre-treatment and therefore at ABR COD feed is 800 mg l^{-1}. The typical BORDA ABR design value pointed out on the dotted line represents the design at unchanged typical load (P = 200 connected users).

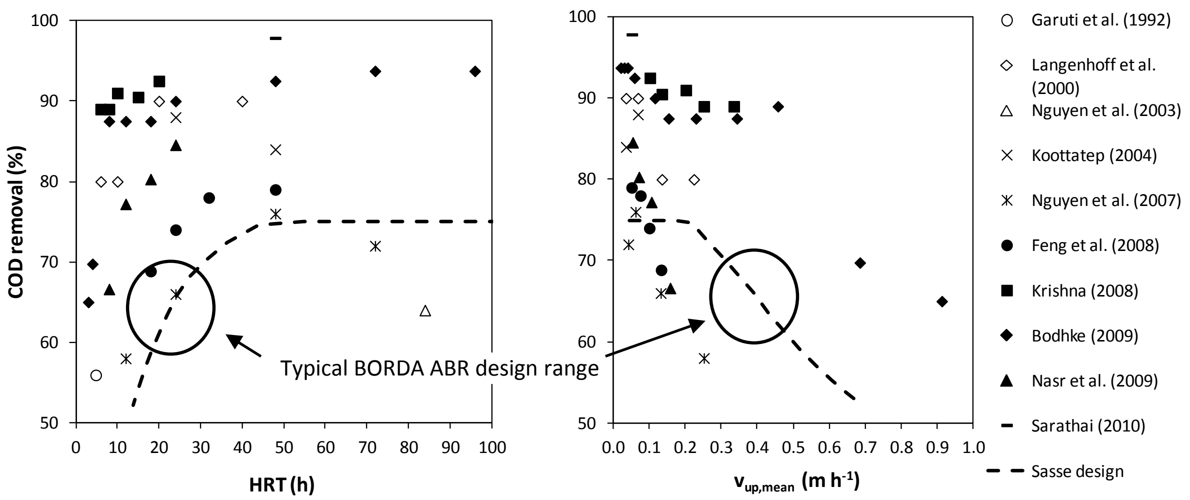

Figure 11: Performance efficiency against various hydraulic retention times

Figure 12: Performance efficiency against various average up-flow velocities (v_{up})

There is a wide variation of reported COD removal rates which is most likely partly due to experimental differences.

Experimental differences that probably have little effect on the treatment efficiency are temperature variations between 25°C and 35°C and the amount of initial inoculums: most studies were done at constant 35°C. Intrachandra (2000) observed that ABR sludge adapts to a temperature drop from 35°C to 25°C in a way that COD removal is not affected. Langenhoff et al. (2000) reported that two parallel laboratory-scale ABRs with different amounts of inoculum perform similarly.

Table 7 lists potentially COD removal influencing factors such as inoculum-type, start-up period and period of constant loading. As explained above, long start-up and long constant loading periods such as described in Krishna and Kumar (2008), Bodkhe (2009) and Nasr et al. (2009) are important for representative results. However there are differences in the outcomes of the two studies: Krishna and Kumar (2008) and Bodkhe (2009) reported by far the best performances, especially at higher loading rates with $v_{up,mean}$ above 0.1 m h^{-1}. This is remarkable since Bodkhe (2009) ran his experiments without seeding the reactor. Nasr et al. (2009) however found considerably lower COD removal at similar loading rates.

BORDA DEWATS ABRs are designed with reactor chamber effluent pipes 200 mm below water level. Most published investigations were performed on ABRs with simple overflow weirs between reactor chambers which certainly reduces the scum retention of the system. As explained above, pH and alkalinity are important process parameters. Several publications however include no information on such.

Also, none of the existing publications takes the diurnal fluctuations of communal wastewater production into account since all systems were loaded with constant feed flow. The low loading of a full-scale reactor during the night may however affect the treatment characteristics of that reactor (Lettinga et al., 1993).

Other influencing factors are the reactor geometry (ratio of chamber length to chamber height) and feed composition.

Table 7: Experimental differences potentially influencing COD removal

Reference	Inoculum	Start-up period d	Constant loading period d	Chamber effl. below waterlevel	pH in	pH out	Alkalinity in $mgCaCO_3l^{-1}$
Langenhoff et al. (2000)b	digester sludge	14	14 - 55	yes	n.a.	n.a.	n.a.
Nguyen et al (2003)	n.a.	n.a.	n.a.	no	n.a.	n.a.	n.a.
Koottatep (2004)	n.a.	n.a.	n.a.	no	7.4 – 8.5	n.a.	240 - 450
Nguyen (2007)	digester sludge	n.a.	n.a.	no	n.a.	n.a.	n.a.
Feng (2008)	digester sludge	38	21	no	7.2	7.4	400
Krishna (2008)	UASB sludge	285	60 - 90	no	7.8	7.1	n.a.
Bodhke (2009)	none	90	> 14	n.a.	7.5 – 8.2	7	230 - 300
Nasr (2009)	anaerobic sludge	100	100	no	6.7 – 7.1	7.3	n.a.
Sarathai (2010)	ABR sludge	90	30	no	6 - 7	7	600 - 900

While the above listed differences between the studies reduce the degree with which comparisons can be made, conclusions on trends are still possible:

Several studies describe high COD removal rates of about 90% (Krishna and Kumar, 2008; Langenhoff et al., 2000; Nasr et al., 2009; Sarathai et al., 2010) and are therefore above the Sasse ABR design prediction of 68%.

Almost all authors reported an increase of COD removal with an increase of HRT until 48 h and the tendency to asymptote at higher HRTs (Bodkhe, 2009; Feng et al., 2008a; Krishna and Kumar, 2008; Langenhoff et al., 2000; Nasr et al., 2009; Nguyen et al., 2007) in accordance with the Sasse design. The decrease observed by Nguyen et al. (2007) at HRT above 50 h may be attributed to mass transfer limitations because of insufficient mixing and therefore reduced substrate-MO contact or too little OLR.

Significant decrease at low HRTs was observed by Bodkhe (2009) below 6 h and by Nasr et al. (2009) below 12 h.

Most observed $v_{up,mean}$ were below 0.5 m h^{-1} (see Figure 4) with often considerably better treatment performances than predicted by the Sasse design.

Langenhoff et al. (2000) reported that each feed flow-rate increase initially led to an effluent COD increase and reduced biogas production. Both however regained a constant level over time. Intrachandra (2000) and Bodkhe (2009) indicated that ABR treatment performance takes about two weeks to become constant after a load change. Feng et al. (2008a) reported that after HRT change effluent COD_t concentration increases during 10 d before re-attaining the initial value. Krishna and Kumar (2008) observed an increase of COD_s concentration in the effluent during 10 d after each HRT change after which the values dropped back to the initial level.

2.3.5.6. Influence of hydraulic loading on reduction of COD fractions

This section summarizes the published observations on the influence of hydraulic loading on the ABR reduction of COD fractions. All authors except one (Krishna and Kumar, 2008) observed effluent solid increase with increased hydraulic load although of different degrees. COD_s concentration increase in the effluent with increased load was mentioned by Langenhoff et al. (2000), Krishna and Kumar (2008) and Nasr et al. (2009). COD_s concentration increase is generally reported to be less than COD_p concentration increase.

Langenhoff et al. (2000) described their reactors as being slightly influenced by washout with $v_{up,mean}$ of 0.03 to 0.2 m h^{-1}. ABR effluent VFA concentrations were about 5 mg l^{-1} COD equivalents under stable operation. Reduction of HRT generally led to an increase of SMP[5] in the effluent. This observation was confirmed by Feng et al. (2008b).

Nguyen et al. (2007) reported an increase of effluent TSS at loading rates resulting in $v_{up,mean}$ of more than 0.06 m h^{-1}.

During experiments undertaken by Feng et al. (2008a), particulate COD reduction declined with increasing hydraulic load ($v_{up,mean}$ of 0.05 m h^{-1} to 0.13 m h^{-1}). The COD data was supported by increasing Settleable Solids (SS) effluent concentrations. Soluble COD reduction was largely constant with acetic acid having been by far the largest effluent soluble COD fraction.

Krishna and Kumar (2008) ran their reactors with $v_{up,mean}$ of 0.1 m h^{-1} to 0.33 m h^{-1} and reported relatively constant particulate COD effluent values at all HRTs. Effluent COD was mainly soluble which slightly increased with increasing load.

Bodkhe (2009) observed a slight effluent VSS increase with increasing load. Overall VSS retention however was always higher than 85%. Bodkhe (2009) observed no sludge washout at any of the investigated loading rates of which the highest resulted in a $v_{up,mean}$ of 0.9 m h^{-1}.

Nasr et al. (2009) reported a decrease in treatment efficiency when the HRT was reduced from 24 h to 12 h which was mainly due to increased effluent solids content. Effluent TS increase was observed at 12 h and 8 h HRT which corresponded to $v_{up,mean}$ of 0.1 m h^{-1} and 0.15 m h^{-1}. An increase of effluent soluble COD and VFA was also observed with increasing load however to a lesser extent.

2.3.5.7. Influence of hydraulic loading on digestion

This section summarizes the published observations on the effect of the hydraulic loading on the digestion indicators biogas-production and sludge activity.

Krishna and Kumar (2008) reported a reduction of biogas production per kg COD removed when HRT was reduced from 20 h to 6 h ($v_{up,mean}$ increased from 0.1 m h^{-1} to 0.33 m h^{-1}). 44% to 56% of the incoming COD was converted to methane of which approximately one quarter was dissolved and 10% retained as sludge. The COD mass balance however could not be closed and significant amounts of COD were not accounted for.

Bodkhe (2009) on the other hand reported that the best observed biogas yield of 0.34 m³ CH_4 kg COD^{-1} and biogas CH_4 content was reached at an HRT of 6 h ($v_{up,mean}$ of 0.45 m h^{-1}). Reduction of specific biogas yield was however observed when the load was further increased and $v_{up,mean}$ reached 0.7 m h^{-1}. Bodkhe (2009) concluded that approximately 50% to 60% of the incoming COD had been converted to methane. Different biogas-generation rates were observed during stable HRT and were traced back to the delayed degradation of accumulated solids.

[5] Soluble microbial products (SMP) are defined as non- or slowly-biodegradable “compounds of microbial origin which result from substrate metabolism and biomass decay“. SMPs can considerably affect the ABR effluent COD concentration since they „...have been found to account for the majority of soluble organic material in the effluent from biological treatment processes“ (Langenhoff et al. 2000). SMPs are analytically determined by subtracting the VFA- from the soluble COD-concentration.

Nasr et al. (2009) stated that biogas production increased with increased loading (24 h to 8 h HRT), however without converting this information to a specific biogas yield and without presenting data or specifying the number of measurements on which the published averages are based. Nasr et al. (2009) reported further that methanogenic activity increased from the front to rear chamber starting with 0.05 g COD g VS^{-1} d^{-1} in the first chamber. The sludge activity increased with increasing load in all chambers. Here too, detailed data was not presented in the paper. The publication contains a solid mass balance based on TS measurements. The difference between observed TS increase and calculated TS increase was interpreted as digestion, concluding that sludge-accumulation and sludge–digestion was proportional to loading rate. Most accumulation was observed in the first compartments.

2.3.5.8. Influence of organic shock loads on treatment processes

This section summarizes the published observations on the effect of organic shock loads on the ABR treatment processes.

Two studies with complex wastewater supply information on this topic:

Krishna and Kumar (2008) tested the effect of organic shock loads on the treatment efficiency of an ABR running at a constant HRT of 8 h. The feed concentration was doubled and tripled for a duration of 4 h and 8 h respectively. Noticeable COD increase only occurred in the first three to four chambers with the first two chambers being most strongly affected. The effluent concentrations after the 4^{th} chamber were constant. All concentrations returned to their initial levels 6 h after shock load.

Bodkhe (2009) mentioned constant effluent concentrations although it was evident that feed concentrations fluctuated strongly.

Investigations with soluble wastewater carried out by Chinese research teams (Hu et al., 2006a; Liu et al., 2007) confirm the above: OLR peaks had almost no effect on effluent concentrations and primarily affected the first few chambers where VFA increased and pH decreased. Operating situations always regained normality within a few hours after the peak.

The results on soluble feed reported by Intrachandra (2000) show that initial concentrations were regained approximately 10 d after organic peaks led to OLR of 1 to 4 kg COD m^{-3} d^{-1}. Increase in CH_4 production, VFA, hydrogen and acidity were observed in the first two chambers. However after 48 h of high organic load, the acid components decreased in the first compartment. This was interpreted as sludge adaptation to the new loading.

2.3.5.9. Influence of hydraulic shock loads on ABR treatment processes

This section summarizes the published observations on the effect of hydraulic shock loads on the ABR treatment processes.

Two studies with complex wastewater provide information on this topic:

Krishna (2007) increased the system load over a period of 3.5 h to 1 h HRT or a $v_{up,mean}$ of 2 m h^{-1} while keeping the feed concentration constant. This represented a six-fold loading increase in comparison to normal operation at 6 h HRT. The shock-load resulted in a 450% effluent COD concentration increase due to increased VFA and more so COD_p concentration. The author mentioned that less than 10% of the sludge initially contained inside the reactor was washed out during the shock-load due to increased up-flow-velocity and biogas production. The reactor reached its former treatment efficiency and effluent quality in less than 10 h. The author pointed out that more research should be done on this topic.

Sarathai (2010) investigated the effect of peak-flows on ABR treatment efficiency by simulating them for 1 h duration twice a day. The peak flow factors (PFF)[6] were 2, 4 and 6 leading to $v_{up,mean}$ values of about 0.1 m h^{-1} to 0.3 m h^{-1}. This flow regime had very little effect on the treatment efficiency: only a 5% difference to constant feed flow was measured.

Three studies with soluble feed substrate also supply information on this topic:

Langenhoff et al. (2000) observed that reducing the HRT to 1.3 h (v_{up} of 1 m h^{-1}) over 48 h resulted in very little solids washout (less than 0.05 g VSS d^{-1}).

Intrachandra (2000) however halved the HRT of a bench-scale reactor to 3 h over 10 d ($v_{up,mean}$ of 0.3 m h^{-1}) and observed a strong decrease of treatment efficiency in all chambers as well as biomass washout. The latter was attributed to higher liquid velocity and gas production. No adaptation to such load was observed over time and good treatment performance was resumed only when the load was reduced to its previous value. The acetogenic activity and sludge particle size were reduced noticeably 30 d after the high load experiment. It is interesting to note here that the system had no difficulties adapting to a 100% increase of only the OLR.

Krishna and Kumar (2009) report sludge wash-out during their laboratory-scale ABR investigation at an HRT of 6 h ($v_{up,mean}$ of 0.33 m h^{-1}).

2.3.5.10. Role of the ABR compartmentalisation

This section summarizes the published observations on the role of ABR compartments on the reactor performance.

Koottatep et al. (2004) mentioned the importance of compartments in absorbing strong feed fluctuations.

All studies agree that by far most COD removal takes place in the first two to three chambers. Increased loading can lead to a shift of COD reduction to the rear compartments (Krishna and Kumar, 2008; Nasr et al., 2009; Nguyen et al., 2007). This was confirmed by researchers running ABRs with soluble wastewater (Dai et al., 2000; Hu et al., 2006a; Intrachandra, 2000; Nie et al., 2008; Peng et al., 2013; Shen et al., 2004; Yuan et al., 2012). All noted that the rear compartments (after the third) do not play much of a role in terms of COD reduction.

Throughout all the studies, the highest VFA concentrations and lowest pH values were found in the first two compartments, increasing towards the rear of the reactor. The authors generally interpreted this as methanogenic and acidogenic species having comparatively high biochemical activities in the first compartments.

Krishna and Kumar (2008) observed a predominance of hydrolyzing and acid producing MO in the first compartment and large numbers of what was thought to be *Methanosaeta* in the second and third

[6] The peak flow factor (PFF) is defined as the ratio of peak to average hourly flow.

compartments, decreasing towards the rear of the reactor. Investigations were done using scanning electron micrographs (SEM).

Nasr et al. (2009) reported that the specific methanogenic activity (SMA) of the sludge increased from the first (0.05 g COD g^{-1} VS d^{-1}) to the last chambers with the highest biogas-production at the rear of the reactor. Bodkhe (2009) however measured most biogas production in the front and decreasing down the reactor.

Peng et al. (2013) investigated the spatial distribution of microbial communities in ABR chambers treating soluble wastewater. They observed the highest relative number of methanogens in the rear compartments. Microbial community analyses were carried out through fluorescent in situ hybridization and Denaturing Gradient Gel Electrophoresis (DGGE).

By far the most COD was however removed by the front compartments. This indicated that in absolute numbers most methanogens might very well have been in the front compartments.

2.3.6. Investigations on pilot- or full-scale ABRs

2.3.6.1. Literature selection

Literature selection criteria for this review section are:

- Studies on pilot or full-scale ABR technology
- Communal wastewater
- Mesophilic conditions

A number of publications and reports on full-scale ABR systems operating in colder climates was found (ENPHO, 2011; Falkenberg, 2012; Müller, 2009; Orozco, 1997; Pravinjith, 2010; Schalk et al., 2014; Singh et al., 2009) but are not further discussed below since they provide no information about mesophilic operating conditions.

2.3.6.2. Review

This section summarizes the published research outcomes and treatment data of pilot or full-scale ABR systems treating communal wastewater under mesophilic temperature ranges.

Nguyen et al. (2007) presented limited information on the treatment efficiency of an ABR designed for the treatment of wastewater from 20 households and livestock breeding in Vietnam. The design includes one settling chamber and four ABR chambers. Feed COD concentrations were very high averaging around 2,500 mg l^{-1}. The average treatment efficiency was 88% COD reduction. However no information is provided about the actual plant loading or the number of measurements the average values are based on. The authors reported that the treatment efficiency decreased after 2 y and that the sedimentation chamber should therefore be emptied biannually.

Rochmadi et al. (2010) reported average values of Indonesian communal BORDA DEWATS ABR COD feed and effluent concentrations of 346 mg COD l^{-1} and 61 mg COD l^{-1} respectively, probably measured on the islands of Java and Bali. These values appear to be averages of measurements performed at several plants. The methodology section of this paper however is incomplete. Information is neither given about the exact number, loading or plant-setups nor on the number and standard deviation of the measurements.

Kerstens et al. (2012) presented information gathered during their visits to eight Indonesian BORDA DEWATS with ABR technology. The setups of five systems included a settler as primary treatment followed by an ABR and AF of varying chamber-numbers fed by household wastewater. Two systems treated a mixture of household and public toilet wastewater. Their pre-treatment consisted of a biogas-digester followed by ABR and AF. The authors mention effluent concentrations of about 100 mg COD l^{-1} to 150 mg COD l^{-1} for all systems. Values are based on two sampling visits per plant and it was mentioned that they might have been influenced by storm water since samples were taken during the rainy season. Information on the plant loading was given as an estimated number of users.

The only available exhaustive set of information gathered during a pilot or full-scale study which goes beyond the sole description of COD removal and investigates the removal process against system load is given in Foxon (2009). The investigations were done in Durban/South Africa on a pilot ABR with eight chambers and 3 m³ reactor volume. The reactor was pump-fed with a constant flow of screened communal wastewater.

The author concluded that the ABR functions as a solid retention device in which the solids are reduced by anaerobic digestion. The main factor controlling solids washout and therefore reactor performance is the up-flow velocity. It was deduced that the up-flow velocity and to a lesser degree the HRT are the critical design parameters.

At high up-flow velocities a stable anaerobic population failed to establish itself which led to increased solid accumulation rates:

While running the system at a HRT of 22 h ($v_{up,mean}$ of 0.5 m h^{-1}), mass balance calculations indicated that 30% of the feed COD were removed by AD while 0.43 kg dry solids accumulated per kg applied COD. Reduced loading (HRT: 42 h, $v_{up,mean}$: 0.3 m h^{-1}) however, led to improved performance: about 60% of feed COD were digested to biogas and 0.11 kg dry solids accumulated per kg applied COD.

Sludge-granulation was observed and microscopy investigations showed increased microbial diversity and number of methanogens. Acetoclastic morphotypes were observed in the rear compartments. Higher free and saline ammonia increase over the treatment further indicated improvement of the anaerobic digestion. Approximately 80% COD removal was observed but it was noted that low initial wastewater alkalinity (about 200 mg $CaCO_3$ l^{-1}) leading to a low pH (<6.5) generally inhibited the anaerobic processes.

The author concluded that for low alkalinity feed concentrations $v_{up,mean}$ should be 0.3 m h^{-1} with an HRT of about 20 h. Higher average up-flow velocities may be possible if the pH could be kept higher.

The author noted that higher peak velocities might not be problematic since in previous research short peaks of high flow led to less solids washout than longer peaks of lower flow (Garuti et al., 2004). Little washout could be tolerated as long as the prevailing $v_{up,mean}$ allowed establishment of granules. From the time that fast settling granules had been established, flows could be increased.

Most treatment was observed in the first compartments. The pH value was lower in the first compartment than in the feed and gradually increased towards the rear of the reactor. It was concluded that hydrolytic and acidogenic processes occurred in all chambers.

The ABR reduced the variation of feed concentration fluctuations.

Illegal dumping of high concentrated wastewater at the head of works led in one occasion to souring and reactor failure of the pilot ABR. The ABR however had recovered completely 10 d after this incident.

2.3.7. Summary of ABR findings

2.3.7.1. General ABR performance

High COD removals larger than 90% were observed in several laboratory studies run under ideal conditions. The little data available on full-scale implementations indicates that COD removal of about 80% is attainable under field condition which is higher than predicted by the BORDA ABR design (approximately 70%).

2.3.7.2. Influence of organic loading rates on the ABR treatment processes

No publications could be found on the effect of hydraulic load decoupled from the OLR. Most authors describe experiments in which the feed flow is varied while the feed concentration is kept constant. Literature however indicates that the thus induced OLR variations have a negligible effect on the treatment compared to HRT.

Several laboratory studies using a feed containing only solubilised organics reported that the OLR varying within the range typical for communal wastewater at constant HRT does not influence the ABR treatment significantly as long as feed concentrations are not too low (Intrachandra, 2000; Nie et al., 2008; Sarathai, 2010; Shen et al., 2004). One publication with incomplete information on research methodology (Shen et al., 2004) states that reduction of treatment efficiency was observed with synthetic COD feed concentrations below 350 mg l^{-1}. This is in line with the literature value for the lowest desirable anaerobic reactor COD feed concentration of 400 mg l^{-1} (Bischofsberger et al., 2005). The comparability of these investigations to real scale situations is somewhat lessened due to the soluble nature of the feed. The investigations however confirm the hypothesis stated by Foxon (2009) who concluded after a comprehensive pilot study that the up-flow velocity and (to a lesser degree) the HRT are the critical design parameters for ABR, not the OLR.

2.3.7.3. Influence of hydraulic loading rates on the ABR treatment processes

Treatment efficiency perturbations after hydraulic load changes lasted generally 10 d to 14 d during which reduced biogas production and higher effluent COD concentrations were observed (Bodkhe, 2009; Feng et al., 2008b; Koottatep et al., 2004; Krishna and Kumar, 2008; Langenhoff et al., 2000; Nasr et al., 2009; Nguyen et al., 2003; Nguyen et al., 2007; Sarathai, 2010).

HRTs of about 48 h yielded generally the best COD reduction in laboratory studies which is in accordance with the predictions of the BORDA DEWATS design. Several authors also reported similar reductions at significantly lower HRTs (Bodkhe, 2009; Koottatep et al., 2004; Krishna and Kumar, 2008; Langenhoff et al., 2000; Nasr et al., 2009). Average up-flow velocities in these studies were generally between 0.05 m h^{-1} and 0.5 m h^{-1} and are therefore also comparable to BORDA DEWATS design. Increase in hydraulic loading led in most cases to effluent solid increase (Bodkhe, 2009; Feng et al., 2008a; Koottatep et al., 2004; Langenhoff et al., 2000; Nasr et al., 2009; Nguyen et al., 2003; Nguyen et al., 2007; Sarathai, 2010). The increase in effluent solid concentration was generally more important than the increase in effluent soluble COD concentration although there are variations amongst the studies. Increase in effluent soluble COD at higher hydraulic loading was often due to rising VFA concentrations. One author noted that increased hydraulic loading led to a rise of SMP concentration in the effluent.

The literature is scarce and not unanimous concerning the effect of general system loading on digestion: Krishna and Kumar (2008) report a decrease of the specific methane production when HRT was decreased from 20 to 6 h whereas Bodkhe (2009) stated that highest specific methane production was observed at a 6 h HRT. The authors however mention difficulties closing COD mass balances with their data. Nasr et al. (2009) reported that the SMA and sludge digestion increased with increased load (24 to 8 h HRT) but did not present detailed data to support this statement.

The outcomes of a comprehensive pilot-study (Foxon, 2009) with constant hydraulic loading periods (excluding the typical flow fluctuations which full-scale plants are intrinsically exposed to) indicate that the ABR functions as a solid retention device in which the solids are reduced by anaerobic digestion. The treatment was negatively affected by low alkalinity and pH under which conditions 20 h HRT and an $v_{up,mean}$ of 0.3 m h^{-1} were proposed as design values. Higher loading led to reduced digestion of the retained sludge. It was hypothesised that a system pH above 6.5 would have allowed higher hydraulic loading.

No data on full-scale plants has been published to date about the relationship between loading (hydraulic or organic) and COD reduction.

2.3.7.4. Influence of shock loads on the ABR treatment processes

All investigations undertaken with complex and soluble wastewater confirm that OLR peaks (up to triple normal load) have almost no effect on effluent concentrations and primarily affect the first few chambers where COD concentrations may increase. Operating situations always regained normality only a few hours after the peak.

ABR systems seem to be much more sensitive to hydraulic peak loads but little research has been published on this topic. Sludge washout depends on numerous factors such as the strength and the duration of peak flows, reactor geometry, sludge settling characteristics and amount of sludge present in the reactor. Comparison of results is difficult since not all information on these factors is available for the existing studies. Langenhoff et al. (2000) for instance stated that very little sludge washout out was observed with 1 m h^{-1} up-flow velocity. Others however reported washout at considerably lower $v_{up,mean}$ (Intrachandra, 2000; Krishna and Kumar, 2009).

The system generally recovers from hydraulic peaks after a few days.

2.3.7.5. Role of the ABR compartmentalisation

The compartmentalisation of the ABR was a strongly stabilizing factor in all laboratory-scale studies and the pilot-plant investigation. All authors agree that organic feed fluctuations are evened out across the reactor and that most COD reduction is observed in the first three compartments. The COD reduction may shift to the rear compartments under higher loading rates while the reactor effluent concentration remains constant.

VFA concentrations are always highest and pH lowest in the front compartments with a respective gradual decrease and increase toward the rear of the reactor.

The separation of different microbiological trophic groups such as reported for high load applications was only observed partially with low concentration feed.

2.3.8. Comparing ABR findings to a similar treatment process: the UASB

ABRs have been described as several UASB reactors in series. A fundamental difference however has to be noted: excess sludge in UASBs has to be discharged regularly (up to several times a week) (Lettinga et al., 1993) whereas ABRs are being operated over several years without sludge discharge.

UASB reactors treating domestic wastewater under tropical conditions have been thoroughly investigated under both laboratory and full-scale conditions for many years by research teams from the Wageningen Agricultural University (Lettinga et al., 1993). The above summarized findings on the ABR are compared to the existing knowledge on UASB technology with a focus on the effect of hydraulic and organic loading rate on reactor performance. All publications mentioned below deal with single compartment UASB reactors.

Lettinga et al. (1993) summarized data gathered over several years on pilot and full-scale plants treating communal wastewater. General treatment capacities of 55% to 75% COD reduction were observed at average HRTs of 5 to 6 h. This is comparable to published research results on the first chamber of ABR reactors. Up-flow velocities were however very high with an average value of 4 m h^{-1} and diurnal peaks of 8 m h^{-1} during 2 h since the reactors were exposed to natural flow fluctuations. Interestingly, treatment was found to be better with higher loading during the day and lower loading during the night than with constant loading over 24 h. The plants were fed with wastewaters of varying COD concentrations (200 mg l^{-1} to 900 mg l^{-1}). No difference in treatment performance is reported. Roughly 40% of feed TSS is converted to excess sludge and 25% to CH_4 whereas about 30% leaves the reactor with the effluent. The much lower reported sludge accumulation rates in ABRs indicate superior sludge stabilisation compared to the UASB.

Leitao et al. (2006b) presented pilot scale investigations on the relationship between hydraulic and organic loading and UASB treatment efficiency. Decreasing feed concentration and HRT both led to decreased treatment. They hypothesised that wastewater COD concentrations below 300 mg l^{-1} leads to mass transfer limitation. Similar conclusions are drawn on ABRs and anaerobic reactors in general. Investigations on the effect of hydraulic loading were carried out with wastewater COD concentrations of approximately 800 mg l^{-1}. HRTs below 4 h (v_{up} above 1 m h^{-1}) were too short and best treatment efficiencies were about 60% COD reduction with an HRT of 6 h and a v_{up} of 0.6 m h^{-1} which is a higher optimal load than found for ABRs by Foxon (2009). pH and alkalinity in the UASB however were higher and therefore more conducive to anaerobic digestion. The authors noted that effluent COD concentrations were strongly influenced by feed fluctuations which compares to observations made on the first ABR reactor chamber.

The effect of shock loads on UASB performance is presented by Leitao et al. (2006a). Shock loads of 6 h duration and five and three times the steady state load value for organic and hydraulic load respectively were applied. Maximum $v_{up,mean}$ of 2 m h^{-1} were reached. It was generally concluded that the system was robust with regards to pH stability and recovery time. Shock loads however led to strong sludge washout and the reactor was not able to attenuate the imposed feed COD fluctuations. The ABR, because of its compartmentalisation, appears to be more resilient especially towards organic fluctuations.

2.4. The AF treating communal wastewater under mesophilic conditions

Anaerobic Filters were first described in 1968 (Young, 1991) and have subsequently, similarly to the ABR, mainly been used for the treatment of high-strength industrial wastewaters. More recent investigations have also documented the AF's ability to efficiently treat low-strength communal wastewater. This section summarizes the main findings.

The main design parameter for AFs is the HRT (Bodik et al., 2002). Young (1991) reported that media specific surface area has little effect on the treatment provided that a specific surface of about 100 m^2 m^{-3} is guaranteed. Reyes et al. (1999) were able to successfully treat synthetic soluble wastewater with packed waste tyre rubber that had a far smaller specific surface (~ 5 m^2 m^{-3}).

Table 8 compiles performance data on AFs treating low strength wastewater under mesophilic conditions. Most published data represents laboratory-scale systems and little information is available on the treatment efficiency of full-scale implementations.

Most investigations showed good BOD_5-removal rates of at least 80% at HRTs of one day and less. Manariotis and Grigoropoulos (2007) demonstrated however that AF-treatment deteriorates substantially at HRTs below 10 h.

Inamori et al. (1986) reported good treatment for feed concentrations as low as 100 mg BOD_5 l^{-1}.

Table 8: Performance data on AFs treating low strength wastewater under mesophilic conditions

System type	Substrate	Vol.	BOD_5 in	COD in	HRT	OLR	BOD_5 rem.	COD rem.	T	Reference
		l	mg l^{-1}	mg l^{-1}	h	kg COD m^{-3} d^{-1}	%	%	°C	
Full-scale	Domestic ww	56,000	100-150		12 - 18	0.1 - 1.2	60 - 70	50 - 71	15 - 25	Young (1991)
Bench-scale	Synth. sol.	n.a.	200		7.5 - 30	0.2	80		30	Inamori et al. (1986)
Bench-scale	Synth. sol.	n.a.	100		30		80		30	Inamori et al. (1986)
Bench-scale*	Synth. complex	6	210	940	24 - 48		80	60 - 70	30 - 35	Reyes et al. (1999)
Bench-scale	Domestic & synth. sol. ww	1.3	440	780	20		95	95	23	Bodik et al. (2002)
Bench-scale	Domestic ww	n.a.			24	0.15 - 0.34		74 - 79	25	Manariotis and Grigoropoulos (2006)
Bench-scale	Synth. sol.	n.a.		280 - 370	24			80	26 - 28	Manariotis and Grigoropoulos (2007)

*growth support with only 5 m² m^{-3} specific surface area; ww = wastewater; sol. = soluble; synth.= synthetic

2.5. Communal wastewater characteristics in developing countries

2.5.1. General

The main design parameters for communal DEWATS are the estimated per capita wastewater production, the peak flow factor and the per capita organic load. These parameters are strongly dependent on water availability, climate, culture and income. Water scarcity for example would logically lead to lower wastewater production and higher concentrations (FAO, 1992). Literature on per capita wastewater and organic waste production is however mainly available on western countries. Engineers in developing countries are therefore forced to use such design values in the absence of more suitable estimates which may lead to oversized systems and resource wastage (Campos and vonSperling, 1996).

This section reviews the existing literature on wastewater characteristics in developing countries.

2.5.2. Feed flow characteristics

2.5.2.1. Per capita wastewater production

Wastewater production is influenced by numerous factors and intrinsically varies from one climatic zone to another, from country to country, from rural to urban areas and from city to city (UNEP, 2014). The "International Benchmarking Network for Water and Sanitation Utilities" (IBNET) from the World Bank's "Water and Sanitation Program" (WSP) provides a publically accessible database for international water and sanitation utilities performance data (WSP, 2014). Table 9 summarizes the most recent residential water consumption values for a number of DEWATS implementation relevant countries. Communal wastewater production values are estimated as being 80% of the residential water consumption. No detailed information was available as for which population sections these values are representative. It can however be assumed that the data has a strong bias towards urban, high-income communities. The majority of the poor urban and rural populations in these countries relies primarily on water from private shallow wells and would therefore not be reflected in these numbers. The values given by the data for Kenya and Tanzania are remarkably low. The reason for this is not further investigated.

Table 9: Communal ww production in selected countries based on residential water consumption data as given in the IBNET/WSP database (WSP, 2014), all values in "l cap^{-1} d^{-1}"

Continent	Country	Residential water consumption	Estimated communal ww production*	Year of inquiry
Africa	Kenya	36	29	2010
Africa	South Africa	190	152	2009
Africa	Tanzania	29	23	2009
Asia	India	83	66	2009
Asia	Indonesia	117	94	2004
Asia	Cambodia	101	81	2007
Asia	Vietnam	115	92	2011
Asia	Philippines	117	94	2009

*estimated as being 80% of the residential water consumption; ww = wastewater

Campos and vonSperling (1996) analysed wastewater data from low-income communities in Brazil. The authors found that the average household income correlated with the per capita wastewater production. They concluded that the generally adopted text book values based on data from western countries overestimate this value for low to middle income areas in Brazil which was found to be 50 l cap^{-1} d^{-1} to 100 l cap^{-1} d^{-1}.

The WHO (WHO/UNEP, 1997) proposes different communal wastewater production ranges for industrial, developing and arid regions (see Table 10). Table 10 also contains further data from other authors on various African and Asian countries. However also here, most of the data stems from water and sanitation utility companies and certainly represents the urban rich more than the poor or rural population.

Crous (2013) measured an average water consumption of 47 l cap^{-1} d^{-1} at community ablution centres in South African informal settlements.

Table 10: Per capita communal ww production data from various sources

Continent/ Region	Country	ww prod. per cap*	Details	Comments	Reference
General		85-200	Industrial regions		WHO/UNEP (1997)
General		65-125	Developing regions		WHO/UNEP (1997)
General		35-75	(Semi-) arid regions		WHO/UNEP (1997)
Africa	Yemen	80	City of Sana'a		WHO/UNEP (1997)
West Asia		100			UNEP (2014)
West Asia	Jordan	90	City of Amman		FAO (1992)
East Asia	Developing countries	160-200		Water supply demand	UNEP (2014)
East Asia	Indonesia	160		Feed to septic tanks	UNEP (2014)
East Asia	Vietnam	150		Values used to calculate sewage by municipalities	UNEP (2014)
East Asia	Vietnam	125	Cities > $3*10^6$ pop.	Estimated values, not measured	UNEP (2014)
East Asia	Vietnam	69	Cities 1 - $3*10^6$ pop.	Estimated values, not measured	UNEP (2014)
East Asia	Vietnam	39	Cities <10^6 pop.	Estimated values, not measured	UNEP (2014)
South Pacific	Fiji	270			UNEP (2014)
East Asia	Thailand	74	Bangkok	Rural areas ^	Tsuzuki (2010)
East Asia	India	143	Cities > 10^5 pop.	#	CPCB (2009)
East Asia	India	97	Cities $5*10^4$ - 10^5 pop.	#	CPCB (2009)

*in l cap^{-1} d^{-1}; ^ Estimated through water usage data for toilet, bathroom, laundry and kitchen; # Estimated as 80% of the per capita water supply; ww = wastewater

2.5.2.2. Flow fluctuations

Communal wastewater flow characteristically fluctuates within seasonal, weekly and diurnal periods. These fluctuations depend on numerous factors and certainly vary from site to site depending on climatic characteristics and water usage habits. Figure 13 presents an example of a typical diurnal wastewater flow pattern with low flow at night and during the afternoon and flow peaks in the morning and evening. The relative amplitude of these fluctuations can be regarded as being stronger the smaller the community is, since varying water usage habits across households are less evened out.

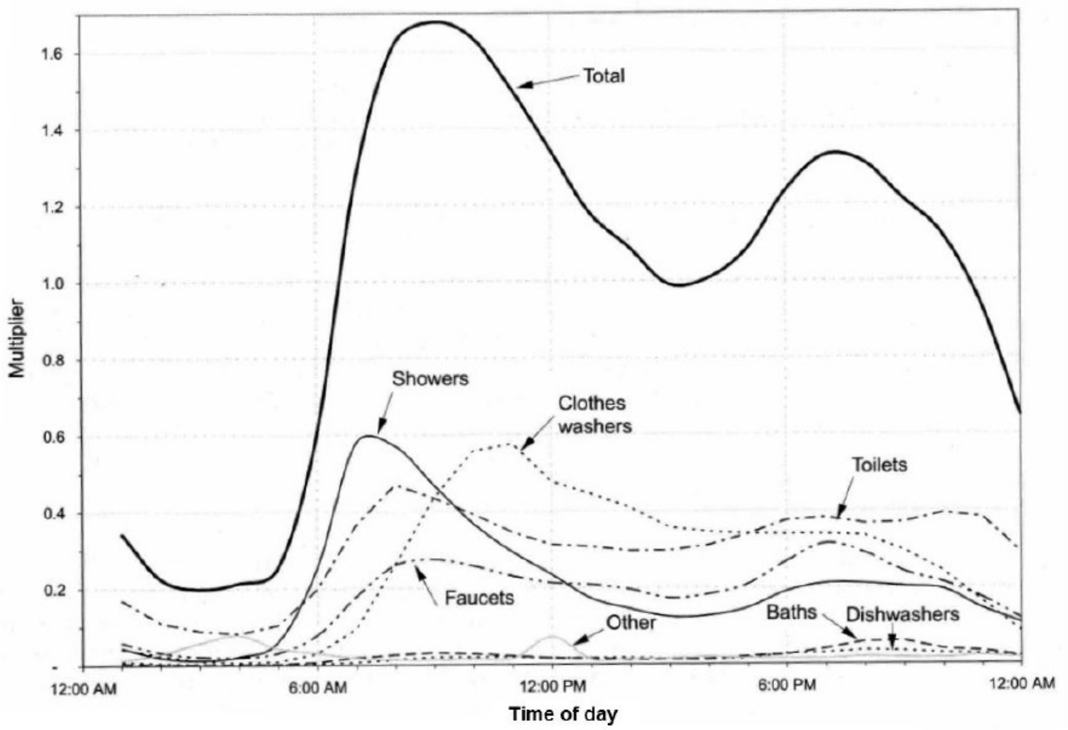

Figure 13: Diurnal variation of domestic water consumption (Haestad et al., 2004)

2.5.3. Typical concentrations

Campos and vonSperling (1996) analysed wastewater data from low-income communities in Brazil. They concluded that the generally adopted text book values underestimate the wastewater concentration of low-income communities which was generally above 300 mg BOD_5 l^{-1}.

The UNEP (UNEP, 2014) confirms that local wastewater characteristics strongly depend on local conditions and habits such as nutrition level, staple food composition and kitchen habits. They therefore "*vary from country to country, from rural to urban areas and from city to city*" (UNEP, 2014) as well as from dry to wet climate. The ranges for general wastewater concentration values for developing and emerging countries reported by WHO/UNEP (1997) are therefore very large (see Table 11). Water scarce areas like Jordan for example feature very high concentrated wastewater. Communal wastewater concentrations can therefore not be generalized and need to be assessed from case to case.

Table 11: Communal wastewater concentration characteristics in developing and emerging countries

Continent/ Region	Country	Parameters (in mg l^{-1})				Comment	Reference
		COD	BOD_5	NH_4-N	PO_4-P		
General		280-2500	120-1000	30-200*	4 to 50		WHO/UNEP (1997)
Africa	Kenya		448	67		Municipal ww in Nairobi	UNEP (2014)
Africa	Kenya		940	72		Municipal ww in Nakuru	UNEP (2014)
West Asia	Jordan	1830	770	150	25	Municipal ww in Amman	FAO (1992)
West Asia	General		530	75	15		UNEP (2014)
South Pacific	Fiji		450				UNEP (2014)
Central and South America	General	350-450	200-250	25-60	5 -10		UNEP (2014)
Caribbean Islands	General	350-450	200-250	25-60	5 -10		UNEP (2014)

*as Kjeldahl-N; ww = wastewater

2.5.4. Per capita pollution loads

The generally assumed per capita pollution loads for the dimensioning of WWTPs are 60 g BOD_5 cap^{-1} d^{-1} and 120 g COD cap^{-1} d^{-1} based on data from developed countries (Tchobanoglous et al., 2003).

Campos and vonSperling (1996) however reported that the average household income in Brazil correlates with the per capita BOD_5 production. They concluded that the generally adopted text book values based on data from western countries overestimate the per capita organic load production for low to middle income areas in Brazil which were typically below 54 g BOD_5 cap^{-1} d^{-1}. Mara (2003) confirms that the per capita BOD_5 load tends to increase with income.

The values proposed by Tchobanoglous et al. (2003) are therefore probably not representative for many of the situations in which DEWATS have to perform. The WHO (WHO/UNEP, 1997) for instance reports that significantly inferior per capita loads may occur (see Table 12). Various authors report per capita BOD_5 and COD loads in Africa and Asia which are only half the value valid for western countries.

Henze et al. (1997) compiled information on wastewater characteristics from several countries. They did however not specify which social class is represented or whether the data applies to rural or urban areas. It can be assumed that the values are rather biased towards higher income, urban dwellings: daily per capita BOD_5 load in Brazil and Uganda is 55 g cap^{-1} d^{-1} to 70 g cap^{-1} d^{-1} and 30 g cap^{-1} d^{-1} to 40 g cap^{-1} d^{-1} in Egypt and India. The commonly used DEWATS design procedure (Sasse, 1998) suggests a daily per capita BOD_5 load 30 g cap^{-1} d^{-1} to 65 g cap^{-1} d^{-1}. In practice DEWATS engineers generally use a per capita load of 60 g BOD_5 cap^{-1} d^{-1} for their design (personal communication, BORDA).

Table 12: Per capita pollution load values reported for developing and emerging countries

Continent/ Region	Country	Parameters (in g cap^{-1} d^{-1})				Comments	Reference
		COD	BOD_5	NH_4-N	PO_4-P		
General		70- 150	30- 60	8-12$^{\#}$	1-3		WHO/UNEP (1997)
Africa	Morocco		50			Rural areas	Abarghaz et al. (2011)
Africa	Kenya		23				UNEP (2014)
Africa	Zambia		36				UNEP (2014)
Southern Africa		100		10	2.5	Load to VIP^	UNEP (2014)
West Asia			53	7.5	1.5		UNEP (2014)
West Asia	Iran	60	40-45	7-8	0.9-3.7	Peri urban area Tehran	Miranzadeh (2005); Rezagholi (1997)
East Asia	Thailand		35				Tsuzuki et al. (2007)
East Asia	Thailand	81.2	46.4	11.5*	1.9	Peri urban area Bangkok	Tsuzuki et al. (2013)

$^{\#}$ as Kjeldahl-nitrogen; * as total nitrogen; ^ Ventilated Improved Pit Latrine (VIP)

2.6. Knowledge gaps in literature

Laboratory and pilot scale investigations tend to confirm the average design up-flow velocity of 0.5 m h^{-1} used in the existing BORDA ABR design. Performance data on full-scale ABR implementations however is extremely scarce and no study linking full-scale plant treatment to hydraulic system load could be found. Laboratory and pilot scale investigations have therefore never been confirmed by investigations on full-scale plants operating under non-ideal conditions and exposed to natural load fluctuations. The extensive research on full-scale UASB reactors cannot fill this knowledge gap in spite of the similarities of the two reactor types since the compartmentalisation of the ABR appears to induce a strongly different reactor behaviour towards feed fluctuations. Effective sludge stabilisation also plays a more important role in the ABR treatment because ABRs are designed for much longer periods of sludge accumulation. Research on full-scale UASB reactors however does indicate that the regular periods of low load typical for communal wastewater could be beneficial for ABR treatment.

The characteristics of wastewater produced in developing countries have been described by a number of authors showing large variations across different regions and countries. Most available data is however based on reports from water and sanitation utility companies and therefore represents the urban, middle to high income population able to afford connection fees. Data on low-income communities however is still very scarce making load predictions for DEWATS dimensioning in such areas difficult.

3. METHODOLOGY

3.1. Social parameters

User numbers and the average monthly household incomes of communities connected to DEWATS were investigated by communicating with the respective heads of the Community Based Organisations (CBO). Each communal DEWATS has an associated CBO which is responsible for the operation and maintenance management of the system. The CBO members are themselves generally part of the connected community and know all community members well.

3.2. Testing integrity of Small Sewerage Systems

For DEWATS performance assessment, knowledge on the integrity of the reticulation system conveying the household wastewater to the treatment plant is essential. It is unfortunately also extremely intricate to thoroughly test small diameter piping such as used in DEWATS projects for blockages and breakages. Due to time and capacity constraints, integrity testing had to be limited to a methodology enabling only a rough assessment of the situation in the field, allowing to at least identify the existence of severe blockages and breakages. The method consisted in pouring food dye concentrate and at least 15 l of water into the household connection located furthest away from the DEWATS. A positive test result indicating system integrity would be concluded if traces of the food dye were observed at the plant feed. This test was conducted with positive results prior to all flow measurements in Indonesia and India described in Chapters 4 and 6.

3.3. Flow measurements

The anaerobic DEWATS treatment steps do not hydraulically buffer feed flow fluctuations (Reynaud, 2008). Flow measurements performed at the plant effluent pipe therefore yield information on short-term (diurnal) feed fluctuations. They have the advantage of not being handicapped by high wastewater solid content as would be the case for measurements performed directly at the plant inlet. Measurements were performed with magnetic induction flow meters and data loggers. In some rare cases, mechanical flow meters were used for flow measurements. Measurements were not performed on public or religious holidays unless stated otherwise.

3.4. Physical parameters and sludge characteristics

3.4.1. Precipitation

Precipitation in the tropics can occur very locally and should therefore ideally be conducted right on site. Daily precipitation data was gathered with a simple pluviometer consisting of a covered bucket with a funnel of known diameter connected to the lid.

3.4.2. Biogas production and CH_4 biogas content from biogas digesters

Digester biogas production was measured by connecting biogas meters to the digester gas outlets. The specifications of one of these meters were: "Make: Krom/Schroder Make; Model: MAGMOL BK-G4, 2006; Max. flow: 6 m^3 h^{-1}; Min. flow: 0.04 m^3 h^{-1}; P_{max}: 0.5 bar; Temperature range: - 20°C to +50°C".

The CH_4 content of biogas was estimated by measuring the CO_2 biogas fraction with a "Brigon Testoryt" and assuming all other gas fractions to be negligibly small. The accuracy of this estimation was checked through measurements performed by external laboratories.

3.4.3. Interpretation-criteria for assessment of storm-water exposure

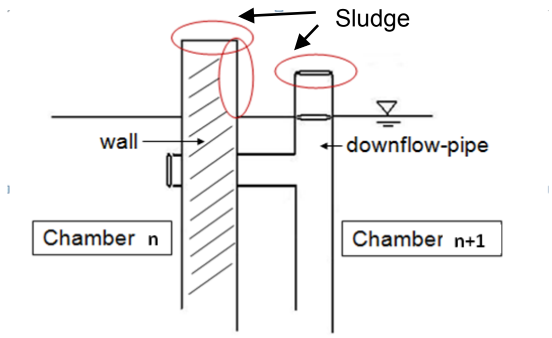

Figure 14: Criteria for exposure to storm water, side view of two ABR chambers

Criteria for exposure of an ABR to storm water were observations such as sludge on partition walls or on down flow pipes as shown in Figure 14.

3.4.4. Determination of sludge levels and sludge sampling

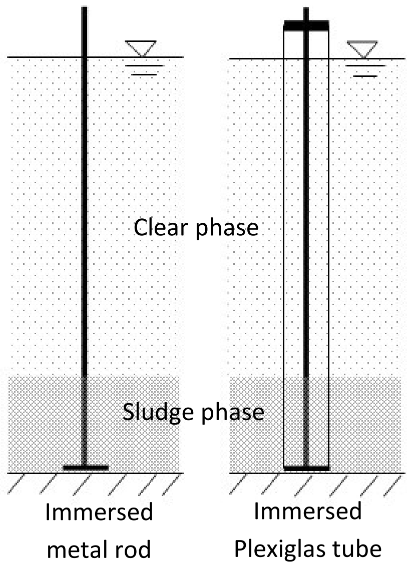

Figure 15: Schematic depiction of the sludge core sampler as used in this study, cross section of a reactor chamber containing sludge

Sludge heights were measured with a specially devised Plexiglas core sampler (see Figure 15) by first immersing the metal rod with the bottom plate in the reactor chamber. The Plexiglas tube is then lowered onto the metal rod and screwed on tight. The sampler is extracted from the chamber to measure settled sludge heights after 5 min of settling time. The content of the core-sampler is then decanted to remove most wastewater from the sample. The exact sample volume after decanting is recorded in order to determine the dilution of settled sludge by wastewater. All solid determinations and activity tests are done with homogenised aliquots of these samples.

Sludge accumulation rates were calculated through linear regression of total sludge-volumes in ABR chambers over periods undisturbed by desludging events.

3.4.5. Sludge Total Solids (TS) and Volatile Solids (VS) measurements

Total Solids (TS) and Volatile Solids (VS) sludge measurements are done following APHA (1998). All measurements are performed in triplicate with a standard deviation of triplicates generally below 10%. Results with higher standard deviations are reported as such. The TS and VS-concentration of settled sludge is calculated using the dilution factor determined when sampling the sludge (see point above).

3.4.6. Specific Methanogenic Activity (SMA) measurement

The Specific Methanogenic Activity (SMA) test investigates the acetoclastic methanogenic activity of an anaerobic sludge by measuring the amount of CH_4 produced by a known amount of sludge (expressed as VS) under ideal substrate (acetic acid) saturated conditions. It is expressed as „ml CH_4 (as COD-equivalents[7]) g VS^{-1} d^{-1}“.

Acetoclastic methanogenic activity accounts for up to 70% of the methane production in the anaerobic digestion of communal wastewater and for most of the conversion of COD (Seghezzo, 2004). Since methanogenesis represents the last and often most sensitive step in the chain of anaerobic digestion processes, the SMA of a sludge is often used as an indicator for its general anaerobic activity (Souto et al., 2010).

[7] The factor f_{bg} which represents the COD value of wet CH_4 volume unit at 20°C is 1/385 g COD ml CH_4^{-1} (Soto et al., 1993). Following the Ideal Gas Law, this leads to a factor of 1/445 at 28°C and 950 m altitude which is representative for measurements in Bangalore and of 1/396 at 28°C and 0 m altitude which is representative for measurements in Yogyakarta.

The informative value of the SMA test is however reduced by the normalization to VS because VS does not differentiate between dead organic material and methanogenic MO biomass. Different sludges with similar VS concentrations could therefore contain different amounts of methanogenic MOs. As a result it is impossible to differentiate between non existing methanogens and existing but inactive methanogens only based on the SMA value. An observed difference in SMA values therefore only allows a qualitative comparison on the average acetoclastic methanogenic activity, not on the amount of methanogens per se.

The substrate used in all SMA tests was sodium acetate since it has pH-stabilizing properties as opposed to acetic acid of which the addition to a solution would lead to significant pH reduction.

Following Soto et al. (1993) maximum SMA (SMA_{max}) should be determined on the linear section of the cumulative methane-production curve during the first hours of the experiment, when VFAs are still high (see Figure 16). The reaction kinetics are therefore substrate saturated and the influence of other processes can be considered negligible.

Consequently it is crucial to ensure the correct substrate to inoculum ratio during the tests in order to produce representative data. Too little substrate would lead to non-saturated conditions or a too short phase of non-saturated conditions. Too much substrate on the other hand would shock the sludge (Pietruschka, 2013) and lead to a lag-phase during which the MO's adapt and little or no methane production occurs (see Figure 16).

Cho et al. (2005) defined the SMA_{max} as the peak on a SMA vs. time plot (see Figure 17). This is the value used in this study to compare different SMA results across reactor chambers and plants. Only the first 5 h of methane production were considered to determine the SMA_{max} value of a sludge (see Figure 17) since potential later peaks could be due to acclimatisation of the sludge to the substrate. These peaks would not represent the state of the sludge when it was sampled.

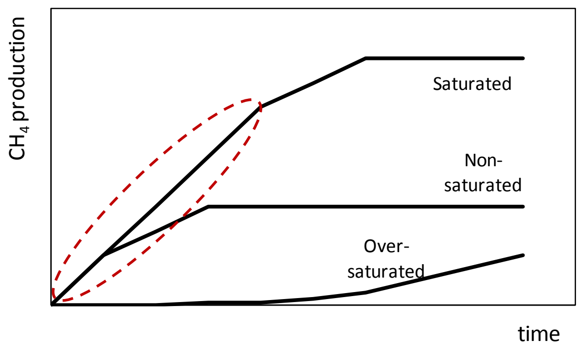

Figure 16: Idealised representation of typical CH_4 production curves under substrate saturated, non-saturated and over-saturated conditions, the dotted mark shows the curve section indicating substrate saturation.

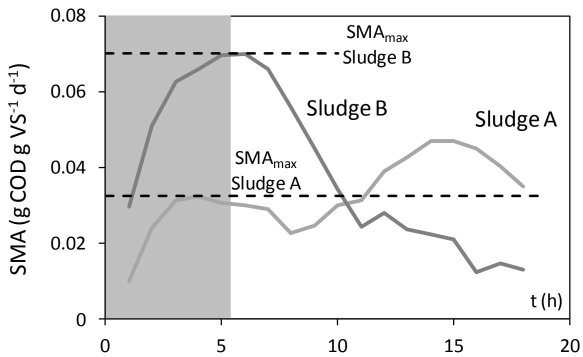

Figure 17: Showcase data to illustrate the SMA_{max} value determination, coloured area indicates the five first relevant hours of the test

There is no existing standard SMA method and methods mentioned in literature vary considerably (Souto et al., 2010). Pietruschka (2013) proposed a methodology for DEWATS-sludge adapted to research conditions in developing countries that was further refined and tested as part of this study (for details see Appendix A2). The detailed SOP resulting from this can be accessed as explained in Appendix A6.

The main outcomes for the SMA method testing are:

- The tests should be conducted with 1 g COD l^{-1} substrate concentration and 150 ml sludge of medium viscosity (still pourable) resulting in an approximate S/I ratio of 0.05 g COD g VS^{-1}
- The tests should be conducted with a single substrate addition
- DEWATS-sludge storage times should not exceed one week since storage was clearly shown to have an adverse, and in some cases strongly adverse, effect on the responsiveness and activity of acetoclastic methanogens
- Standard deviation of triplicate measurements was found to be very small with tests done at the Yogyakarta laboratory, especially during the most decisive first 10 h of the experiments. Results produced there are therefore based on tests conducted with duplicate runs. The SMA investigation results produced by the Bangalore laboratory team are based on triplicate runs since these had considerable standard deviations probably due to leaky pipe connectors.
- Duplicate sequential SMA measurements of samples taken from the same sampling points up to three months apart have a standard deviation of 1% to 12%.

In Indonesia SMA measurements were done in May 2013, at the end of the wet season. They were repeated in the dry season (September 2013) in order to assess whether an extended period without storm-water intrusion would lead to a significant increase of SMA. The last strong rain (120 mm d^{-1}) however was recorded very late in the year, on June 17th, about eight weeks before sludge sampling in August 2013. The last rain (10 mm d^{-1}) even occurred later on July 25th, or about four weeks before sludge sampling. Assuming that rain does affect the methanogenic population through washout, this was a very short period in which to expect any measurable change. Also, precipitation measurements were done at a 2 km distant site. Local rain occurrences affecting the plant can therefore not be ruled out.

3.5. Wastewater sampling

In Indonesia samples were taken from the reactor supernatant using a sampling cup attached to a long handle. In India access to the reactor supernatants was more difficult due to a comparably large freeboard. Samples there were extracted just below water level with the help of a suction device. Both sampling methods are regarded as producing comparable results. Samples were taken close to the effluent pipes of each chamber, thus approximately representing the effluent of the chamber they were taken from. Any scum on the surface would be moved aside prior to sampling to avoid sample contamination. Samples were then immediately put on ice and processed within the time-limits specified by APHA. For more details refer to the complete procedure as explained in Appendix A6.

3.6. Physico-chemical parameters

3.6.1. Alkalinity, pH, electric conductivity and turbidity

All following measurements were carried out onsite immediately after sampling.

Alkalinity of fresh and wastewater was determined using a Merck titration kit. pH measurements were done with handheld devices (HI 8424 and WTW Sentix-41) or indicator paper (Merck 1.09564) in the case of the field measurement campaign presented in Chapter 5. HI DiST4 were used for electric conductivity investigations. Turbidity was measured with a WTW 350 IR handheld Turbidimeter.

3.6.2. Total and fractionated COD and BOD_5

COD measurements were performed using Hanna Instrument (HI 83214) and Merck (Nova 60) Spectrophotometers. Reagents were HI 93754B-MR and Merck 14541. Soluble COD (COD_s) was measured after filtering the samples with Whatman No. 1 filter paper (pore size 11µm). Particulate COD (COD_p) was determined by subtracting COD_s from the total COD (COD_t) measured with the unfiltered sample.

WTW Oxitop IS 6s were used for BOD_5 determinations.

3.6.3. Non-biodegradable COD

Anaerobic processes can only remove the biodegradable fraction of the COD and produce non-biodegradable COD. Thus non-biodegradable COD will inevitably be found in the effluent. In order to accurately assess the treatment efficiency of a reactor, this non-biodegradable fraction needs to be known, since it represents the effluent COD which cannot be removed by the treatment.

The soluble non-biodegradable COD is reported to remain unchanged throughout anaerobic treatment (Melcer and Dold, 2003) and should not vary much over time since it depends on rather stable operational factors and user habits. The total non-biodegradable COD should be reduced throughout the DEWATS due to particle retention. Since ABR effluent generally contained only small amounts of particulate organics, this study will only report soluble non-biodegradable COD measurement results. The measurement was done by storing a sample at room temperature over 3 months, regularly monitoring its fractionated COD concentrations. The concentrations typically dropped over time due to the metabolism of the MO remaining inside the sample and eventually reached a stable minimum value defined as the non-biodegradable COD (see Figure 18). The detailed measurement procedure can be found as explained in Appendix A6. Two to three measurement campaigns were conducted depending on the plant. Each measurement campaign comprised of taking two effluent samples. Duplicate COD_s concentration measurements were conducted on both samples weekly (first month), biweekly (second month) and monthly (final month). Figure 19 presents one typical dataset (measured in BWC/Bangalore) in order to showcase the data analysis. The data points represent the averages of duplicate concentration measurements done on both samples. In this case an average of COD_s concentration of 100 mg COD_s l^{-1} is regarded as being the best approximation for this community. The lower value measured in September was possibly influenced by rain.

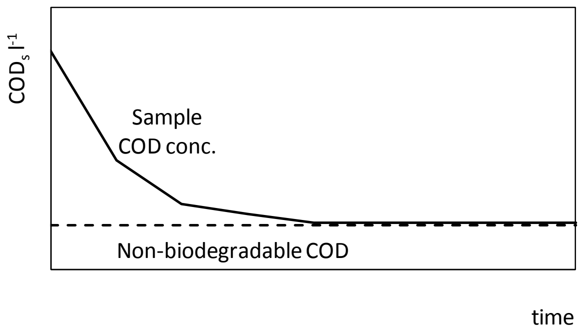

Figure 18: Idealised representation of a typical concentration curve during a non-biodegradable COD concentration measurement

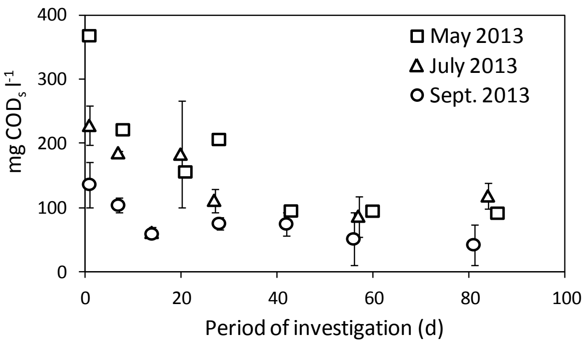

Figure 19: Showcase dataset of a non-biodegradable COD concentration measurement, error-bars indicate the standard deviations of data

3.6.4. Nutrients (PO_4 and NH_4)

Phosphate and ammonia concentrations were measured with a Merck Nova 60 Spectrophotometer and cell tests (catalogue numbers: and 1.00798,0001 and 1.14752,0001) after filtration.

3.7. Loading rates

The hydraulic retention time (HRT) and organic loading rate (OLR) were calculated with the following equations, with $V_{reactor}$ being the total active reactor volume, Q the average daily flow and $COD_{p,in}$ the average feed COD concentration:

$$HRT = \frac{V_{reator}}{Q} \qquad \textit{Equation 2}$$

$$OLR = \frac{COD_{p,in} * Q}{V_{reactor}} \qquad \textit{Equation 3}$$

The confidence limits of the HRT take a daily flow variation of 20% into consideration. Similarly the confidence limits for the OLR which additionally include the standard error of means of COD_t concentration measurements.

3.8. Mass balance calculations

3.8.1. Mass balance across biogas digesters

The COD mass balance across the biogas digester of one case study (see Section 6.3.8.3) was estimated following Equation 4 in order to estimate the COD_t feed concentration ($COD_{t,in}$) of the plant.

$$COD_{t,in} = COD_{t,out} + \frac{Daily\ CH_4\ production * f_{bg}}{Q} \qquad \textit{Equation 4}$$

Soto et al. (1993) cites the factor f_{bg} which represents the COD value of wet CH_4 volume unit at 20°C as 1/385 g COD ml CH_4^{-1}. Following the Ideal Gas Law, this leads to a factor of 1/445 at 28°C and 950 m altitude which is representative for measurements in Bangalore.

The equation is based on the assumption that the particulate COD accumulation inside the digester is small enough not to be considered which is not entirely accurate since sludge certainly does accumulate. This sludge however has extremely long retention times and BGD are known to be able to operate for many years without being desludged. The biogas production consequently originates from (long) accumulated and recently discharged organics which is supported by an observed stable production rate. A BGD therefore operates under a pseudo-steady state and it appears legitimate to simplify the calculation accordingly for the purpose of a rough estimation.

It is also important to realize that the amount of solubilised CH_4 leaving the reactor through the effluent wastewater stream could be considerable. Sarathai (2010) reports solubilised CH_4 to represent up to 10% in COD mass balances performed on laboratory-scale ABRs. The above-mentioned approach, although valid for a first approximation, therefore certainly underestimates the real average feed COD concentration.

3.8.2. COD_p mass balance across ABRs

The theoretical amount of sludge accumulating (l y^{-1}) inside an ABR excluding volume reduction through anaerobic digestion (V_{sludge}) is based on Equation 5. $COD_{p,in}$ and $COD_{p,out}$ are the average COD_p concentrations (g COD m^{-3}) measured at the ABR in- and outflow. Q is the average daily flow (m^3 d^{-1}). Ekama (2009) indicates that the COD_p to VSS ratio of organic wastewater particles (f_{SS}) stays approximately constant throughout the treatment and is about 1.48. VS_{SS} is the VS concentration of settled sludge (g VS l^{-1}).

$$V_{sludge} = \frac{(COD_{p,in} - COD_{p,out}) * Q}{f_{ss} * VS_{ss}} \qquad \textit{Equation 5}$$

The measure of dispersion for COD_p that was used in this case was the *average error of means*. It is considered a more appropriate description of reality than the commonly used standard deviation since it reduces the mathematical effect of outliers and takes the sample size into consideration (Davis and Goldsmith, 1977).

The confidence range for V_{sludge} takes the error of means of COD_p concentrations, a Q variation of 20% and the standard deviation of VS concentration measurements into account.

3.9. Calculating design reactor chamber performance

Design reactor chamber performance of the case study ABRs and AFs presented in Chapter 6 were computed using the design calculation spread-sheet proposed by Sasse (1998). The reactor effluent concentration values represented as the "initial design"-curves were produced by varying the "number of reactor chambers" parameter number inside the spread-sheet while keeping all other parameters constant. The computation of the reactor effluent concentration values represented as the “adapted design prediction” curves additionally required the adaptation of the feed concentration and daily flow values to field measurement outcomes.

3.10. Statistical tests

Statistical tests were used in order to assess whether the means of two or more datasets were significantly different from each other, for instance to assess the significance of reduction by a reactor. The tests used were paired and unpaired sample t-Tests when comparing two datasets and one-way between subjects ANOVA for the comparison of more than two datasets. Prior to these tests data was tested for normality with the Shapiro-Wilk test with an acceptance threshold p of 0.01.

4. FIELD DATA ON DESIGN RELEVANT AND OPERATION RELEVANT PARAMETERS

4.1. Objectives

The main design parameters for communal DEWATS are the estimated per capita wastewater production and the average diurnal flow peak-flow factor. Very little literature is available concerning DEWATS implementation-relevant communities in developing countries, forcing designers to use unsubstantiated estimations for the sizing of the plants. National effluent standards often stipulate maximum concentrations expressed as "mg BOD_5 l^{-1}". The comparative ease of conducting COD instead of BOD_5 concentration measurements in DEWATS implementation areas causes the need to assess the general BOD_5 to COD ratio in DEWATS effluent. Because of the remoteness of many sites, regular effluent monitoring is often impossible. In order to interpret available concentration data from effluent grab-samples, it is therefore essential to understand the typical variations of DEWATS effluent. Information on effluent nutrient content is important in the context of not only compliance with national discharge standards but also its impact on receiving water-bodies and its reuse value for agriculture. Biogas-production is often a welcome by-product of the DEWATS treatment process, but the yield estimations for BGD fed with communal wastewater have not yet been compared to field measurements. The desludging of reactors is the regular DEWATS maintenance activity which requires the largest amount of funds and the highest level of sophistication as regards logistics. It is therefore crucial for city planners to have a good understanding of the required desludging periods of such systems. The current estimate for this period (two to three years) is largely based on experience with septic tanks and has not yet been validated by formal measurement campaigns.

This chapter addresses these gaps and presents data on per capita wastewater production of communities connected to DEWATS, hydraulic peak flow factors, DEWATS effluent characteristics and their fluctuation over time, biogas-production and sludge build-up rates. The investigations have been conducted over several y at numerous communal systems in Indonesia, India and South Africa.

4.2. The plants

Due to local requirements and constraints, each of the investigated systems is unique in terms of system configuration and size. The configuration always consists of a settling unit (either a BGD or settler), followed by an ABR with a varying number of compartments. In some of the systems further anaerobic treatment is achieved through an AF. Polishing steps such as PGF and ponds such as implemented in India and South Africa are not considered in this survey. The communal DEWATS presented in this chapter are either SSS, CSC or SBS systems. All systems are exposed to tropical or sub-tropical (Newlands Mashu in South Africa) climates. Table 13 lists the plants from which the field data was used in this chapter to investigate various design relevant and operation relevant parameters.

Table 13: Plants from which the field data was used in this chapter to investigate various design relevant and operation relevant parameters

Plant information				Effluent characteristics				Design parameters		
Name	Plant code	Country	Type	BOD_5/COD ratio	Effluent COD variation	NH_4-N	PO_4-P	Per cap ww prod.	Sludge build-up	Biogas prod.
Al Futuh	AF	Indonesia	SBS	X						
Al Hikmah	AH	Indonesia	SBS	X				X	X	X
Beedi Workers Colony	BWC	India	SSS		X	X	X	X	X	X
Dawung Wetan	DW	Indonesia	CSC	X						X
Friends of Camphill	FOC	India	SSS	X	X		X	X		
Gambiran	GB	Indonesia	SSS	X	X	X		X	X	
Gatak Gamol	GG	Indonesia	SSS	X				X	X	X
Kandang Menjangan	KM	Indonesia	CSC	X						
Karang Asem	KA	Indonesia	CSC							X
Kaweron	KW	Indonesia	CSC	X						
Keturen	KT	Indonesia	CSC	X						
Kragilan	KG	Indonesia	SSS	X				X		
Makam Bergolo	MB	Indonesia	CSC	X						X
Margo Mulyo	MG	Indonesia	SSS					X		
Minomartani	MM	Indonesia	SSS	X	X	X	X	X	X	
Newlands Mashu	NLM	South Africa	SSS		X	X	X	X		
Panjang Wetan	PW	Indonesia	CSC	X						
Playen	PY	Indonesia	SSS					X		
Plombokan	PB	Indonesia	CSC	X						
Roopa Nagar	RN	India	SSS					X		
Sahabat Kurma	SH	Indonesia	SSS					X		
Sangkrah	SK	Indonesia	CSC	X	X	X	X			X
Santan	ST	Indonesia	SSS	X				X	X	
Wiroyudan	WY	Indonesia	SSS					X		

4.3. Results and discussion

4.3.1. Hydraulic characteristics of DEWATS feed-flow

4.3.1.1. Per capita wastewater production

Table 14 presents the outcomes of fifteen wastewater production measurement campaigns at twelve SSS and one SBS DEWATS.

All plants were built in Central-Java/Indonesia with the exception of RN and BWC which are located in Bangalore/India. Both communities have very limited access to fresh water and particularly low average household incomes. Also, in the case of Roopa Nagar (RN) only black-water and grey-water from bathrooms were discharged to the DEWATS. This explains the low wastewater production values comparable to the values proposed by the WHO for arid regions (see Table 10) (WHO/UNEP, 1997).

The wastewater production measured in KG is surprisingly low, especially since this plant is located in a water-rich area with a connected community with above average income. The data has therefore possibly been affected by an inaccurate water meter or blocked piping.

The observed average per capita wastewater production rates of the remaining systems vary from 62 l cap^{-1} d^{-1} to 91 l cap^{-1} d^{-1} with an average value of 81 l cap^{-1} d^{-1}. This is significantly lower than the flows generally expected in western countries of 170 l cap^{-1} d^{-1} to 340 l cap^{-1} d^{-1} (Tchobanoglous et al., 2003). It corresponds however very closely to the values proposed by the WHO for developing regions (see Table 10) (WHO/UNEP, 1997) as well as to measurements performed in rural areas in Thailand (Tsuzuki et al., 2010). Design information on twenty-six Indonesian SSS showed that systems are currently either designed with 80 l cap^{-1} d^{-1} or 100 l cap^{-1} d^{-1} (see Section 5.2.7).

Table 14: Wastewater production of connected communities, dates behind plant codes indicate y during which measurements were conducted at the same plant

Plant code	Number of people	M	RSD	n	ww prod.	Peak flow	Peak flow factor	Average income class*
		m^3 d^{-1}	%	d	l cap^{-1} d^{-1}	m^3 h^{-1}		
RN	608	15.9	21%	3	26	0.8	1.2	A
BWC 2012	575	16.5	3%	4	29	1.5	2.2	A
KG	480	16.9	31%	10	35	1.0	1.5	C
BWC 2010	605	23.5	4%	6	39	1.8	1.8	A
SH	168	10.3	1%	2	62	1.1	2.6	B
WY	271	20.1	6%	2	74	1.5	1.8	B
PY	213	16.1	23%	9	76	0.8	1.2	B
AH	478	36.8			77	3.1	2.0	
ST	450	36.4	5%	6	81	2.7	1.8	C
GB	195	16.6	13%	7	85	1.5	2.2	B
NLM	420	35.9	17%	107	86	2.5	1.7	
GG	103	9.1	16%	6	88	0.8	2.1	A
MG	125	11.0	10%	2	88	0.6	1.4	B
MM	251	22.9	5%	7	91	2.2	2.3	C

* the following denotations are used to characterize average household income: A= < 50 USD $month^{-1}$; B= 50 USD $month^{-1}$ to 100 USD $month^{-1}$; C= > 100 USD $month^{-1}$; ww = wastewater

Wastewater production in poor communities in Brazil has been reported to depend on the average household income (Campos and vonSperling, 1996). This does not seem to be the case in Central-Java where water is generally abundant with shallow well water freely available to all income groups. The available data shows no correlation between measured daily per capita wastewater production values and the average monthly household income (see Figure 20).

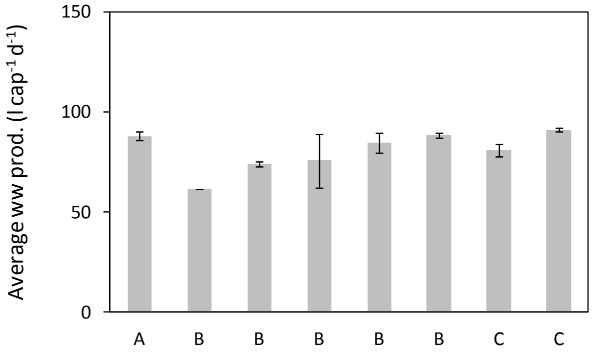

Figure 20: Average per capita wastewater production at Indonesian sites with site-specific standard deviation and dependent on average income group (A= < 50 USD month^{-1}; B= 50 USD month^{-1} to 100 USD month^{-1}; C= > 100 USD month^{-1})

4.3.1.2. Variation of daily flow over time

Daily flows can vary considerably over the duration of the measurement campaigns (see Table 14) with a maximum relative standard deviation of 31% observed in Kragilan. Most relative standard deviations are however below 20%. It must nevertheless be noted that, due to logistical restrictions, most of the available data represents short measurement campaigns that were generally performed for one week or less at a time. On one hand they do not indicate the variation of daily flow over longer periods. On the other hand they might be over-proportionally influenced by extreme single events.

Multiple measurement campaigns were performed in BWC and in MM and indicated considerable variation of daily flow over time. The flow reduction in BWC was traced back to the deterioration of local water access (see Section 6.3.4.2). The increase observed in MM however could not be explained but may partly be due to rainwater infiltration into the reticulation system (see Section 6.5.4.2). This will need to be further investigated in future.

The dataset available from NLM represents 111 d of continuous flow-measurements and is by far the largest and therefore most reliable dataset available for this study (for further details on this plant please see Pillay et al. (2014)). It represents the flow of a group of low-income households in Durban/South Africa. The relative standard deviation of the daily flow was 17%. Based on this and the other available data the variation of typical DEWATS dry weather feed flow is estimated to be 20%.

Figure 21 and Figure 22 and shows the averages of the measured diurnal flow fluctuations. These were measured at the plant effluent. Previous research had shown that the anaerobic treatment steps settler, ABR and AF do not alter the fluctuations of the feed (Reynaud, 2008). The shapes of the flow curves are typical for communal wastewater which generally has two peaks, one in the morning and one in the evening. The morning-peak is the strongest as it is typical for household discharge (Haestad et al., 2004) and lasts for 2 h to 3 h. The average peak-flow factor over all plants is 1.9 with 20% relative standard deviation (see Table 14) which is very close the design assumption proposed for the dimensioning of DEWATS by Sasse (1998).

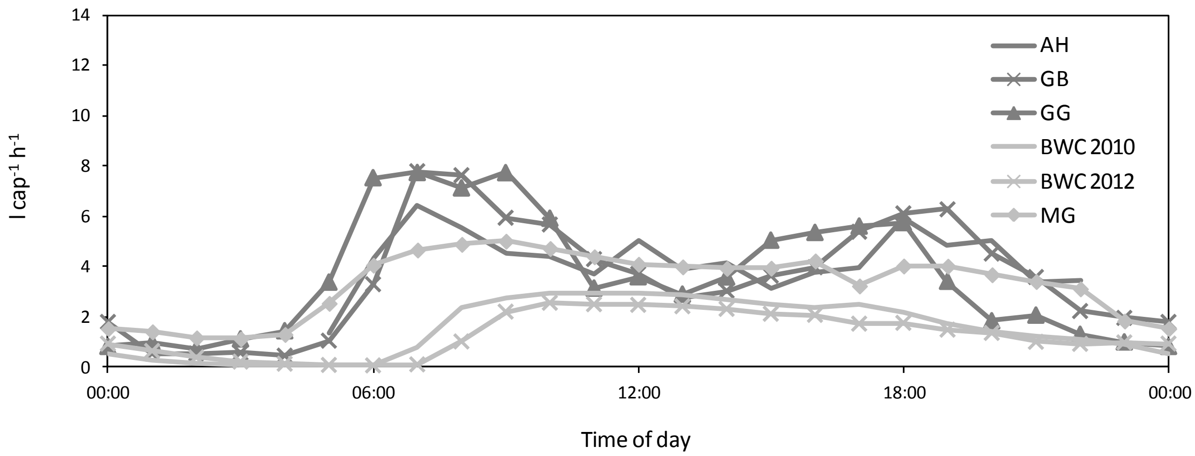

Figure 21: Average per capita diurnal flow fluctuations measured during six measurement campaigns at five sites

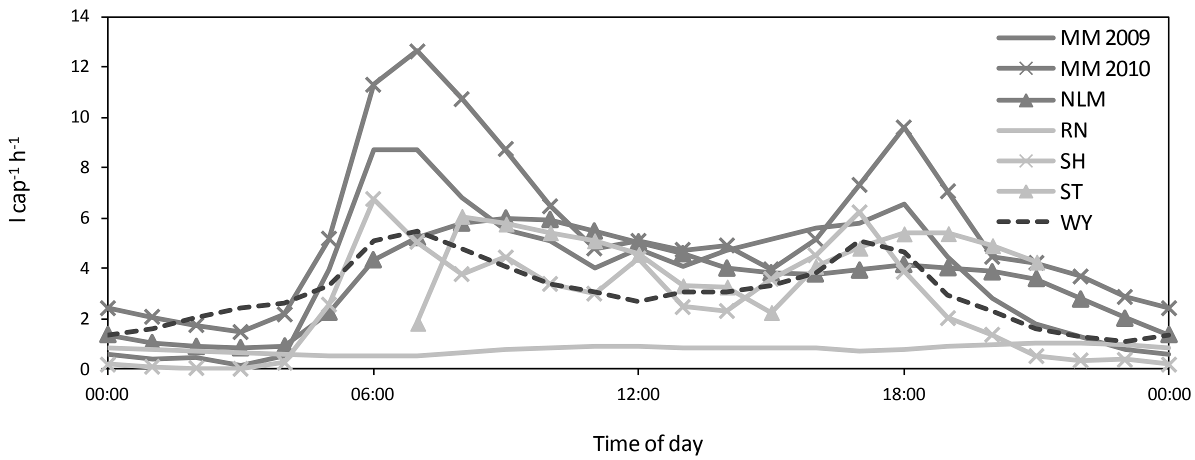

Figure 22: Average per capita diurnal flow fluctuations measured during seven measurement campaigns at six sites

4.3.2. Characteristics of DEWATS effluent

4.3.2.1. BOD_5/COD ratio in DEWATS effluents

A total of eighty measurements were performed on anaerobic treatment effluent flows from sixteen different DEWATS plants (see Figure 23) which show an average BOD_5/COD ratio of 0.46 (or a COD/BOD_5 ratio of 2.2) with a relative standard deviation of 38%. Kerstens et al. (2012) contains a dataset with thirty-two measurements from eight DEWATS plants and an average BOD_5/COD ratio of 0.4 with a relative standard deviation of 16%. Rochmadi et al. (2010) reported to have measured average communal DEWATS effluent concentrations of 22 mg BOD_5 l^{-1} and 61 mg COD l^{-1} which corresponds to a BOD_5/COD ratio of 0.36.

The average BOD_5/COD ratio described in this study is surprisingly high and only slightly lower than screened North-American raw wastewater which is reported to have a ratio of 0.49 (Dixon et al., 1972; Smith and Eilers, 1969). Wastewater after biological treatment is reported to have a significantly lower ratio of 0.1 to 0.25 (Mara and Horan, 2003).

While this could indicate a comparably low content of nonbiodegradable COD in the wastewater treated by DEWATS it could also mean that significant amounts of biodegradable organics leave the DEWATS after the last anaerobic treatment step untreated. The latter explanation is supported by the often high measured effluent BOD_5 concentrations (see Figure 23).

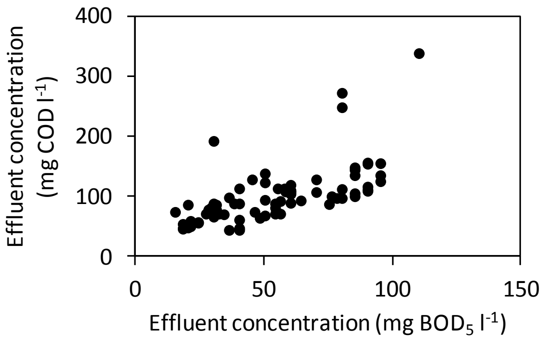

Figure 23: COD vs. BOD_5 effluent concentrations

4.3.2.2. Variation of effluent COD concentration over time

Effluent COD concentration measurements from the last anaerobic treatment step were performed over extensive periods at 6 sites, as can be seen in Table 15. The relative standard deviation over those measurement periods was between 13% to 17% for plants with effluent concentrations above 100 mg COD l^{-1}. Standard deviations were found to be considerably higher for plants with low effluent concentrations, which is probably due to the higher measurement error of the method in these concentration ranges.

Table 15: Long-term variation of COD effluent measurements found in seven different systems

Plants			COD effluent variation			
Plant Code	Plant type	Period of sampling (months)	M*	SD*	RSD	n
BWC	SSS	41	320	59	18%	37
FOC	SSS	98	82	30	36%	33
GB	SSS	49	127	22	18%	68
MM	SSS	67	77	26	39%	122
NLM	SSS	6	406	55	13%	18
SK	CSC	39	167	27	16%	8
ST	SSS	25	108	12	11%	19

*****in mg COD l^{-1}

Figure 24 shows the average hourly effluent COD concentrations from hourly measurements taken on five consecutive days from the 19th to the 23rd of July 2008 in Minomartani, Indonesia (Reynaud, 2008). All samples represent the effluent from the last AF. No rain was recorded on any of these days. All values taken at the same time of day were found to be normally distributed using the Shapiro-Wilk normality test[8]. A one-way between subjects ANOVA test was conducted and it was found that no significant difference exists at the $p < 0.05$ level between the values measured at different times of the day [$F(13, 54) = 1.32$, $F_{crit} = 1.91$]. This therefore indicates that the time of day at which effluent samples are drawn does not significantly influence the measurement outcome. Total average of all

[8] The data subsets representing 10:00 and 19:00 could not be tested since they included only four data points. All other data subsets fulfilled the minimum requirement of five data points to conduct a Shapiro-Wilk normality test.

measurements (n= 68) is 54 mg COD l^{-1} with a standard deviation of 10 mg COD l^{-1}. This corresponds to a relative standard deviation of 20% which is comparably high. As explained above, the relative standard deviation is expected to be lower for higher average effluent concentrations.

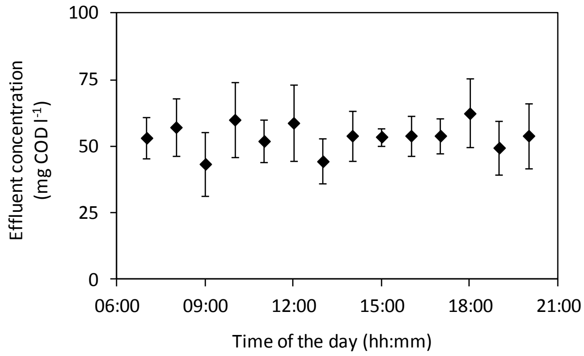

Figure 24: Average hourly effluent COD-concentrations from hourly measurements done on five consecutive days from the 19th to the 23rd of July, 2008 in Minomartani, Indonesia, error-bars indicate the standard deviation of hourly measurements (Reynaud, 2008)

4.3.2.3. Nutrients

The nutrient concentrations measured at the effluent of the last anaerobic treatment steps of 6 sites are shown in Table 16. These concentration ranges have been confirmed by several authors. (Kerstens et al., 2012) measured average DEWATS effluent concentrations of 61.7 mg NH_4-N l^{-1} and of 4.3 mg PO_4-P l^{-1}. Also Foxon (2009) measured similar average ABR effluent concentrations of 43 mg NH_4-N l^{-1} and of 14 mg PO_4-P l^{-1}. Garuti et al. (1992) reported an approximate concentration of 50 mg NH_4-N l^{-1} at the effluent of the ABR they investigated.

The retention and anaerobic treatment processes within the ABR and AF do not intrinsically affect the nutrients inside the wastewater. The observed effluent concentrations can therefore be assumed to also represent the feed concentrations of the plants. For this reason available feed flow and user data was used to calculate the approximate per capita ammonia and phosphorous loads. The resulting ammonia values (average across plants = 5.6 g NH_4-N cap^{-1} d^{-1}) all fall above the range reported by Tchobanoglous et al. (2003) (2 to 4 g NH_4-N cap^{-1} d^{-1}) and slightly below the range proposed by the WHO (WHO/UNEP, 1997) (8 to 12 g TKN cap^{-1} d^{-1})[9]. The resulting per capita phosphorous loads (average across plants = 0.8 g PO_4-P cap^{-1} d^{-1}) are slightly below or within the ranges reported by Tchobanoglous et al. (2003) (1 to 2 g PO_4-P cap^{-1} d^{-1}) and by WHO/UNEP (1997) (1 to 3 g PO_4-P cap^{-1} d^{-1}). The observed nutrient loads can therefore be considered to be in accordance with literature values.

The per capita ammonia loads vary little across communities. The high concentrations measured at some of the sites are thus attributed to low per capita wastewater production resulting in little dilution.

[9] Total Kjeldahl Nitrogen (TKN) comprises ammonia and nitrogen bound to organic compounds. It is therefore intrinsically larger than the NH_4-N value.

Table 16: Effluent ammonia and phosphorous concentrations of seven different SSS

Plants		Ammonia (NH_4-N) effluent conc.					Phosphorous (PO_4-P) effluent conc.				
Plant Code	Period of sampling*	M #	SD #	RSD	n	Per cap +	M #	SD #	RSD	N	Per cap +
BWC	41	123	40	33%	27	5.0	18	3	17%	10	0.7
FOC	98						18	7	39%	10	1.6
GB	49	76	20	26%	51	6.5	6	1	21%	6	0.5
GG	1	78	3	4%	2	6.8	11	0	1%	2	1.0
MM	67	49	4	8%	7	5.5	6	1	11%	6	0.6
NLM	6	61	21	34%	9	5.2	9	1	12%	4	0.8
ST	25	50	8	17%	8	6.0	4	1	20%	8	0.5

* in months; # in mg l^{-1}; + in g cap^{-1} d^{-1}

4.3.3. Biogas-production in communal DEWATS applications

Biogas production measurements were carried out over varying time periods, recording the cumulative gas production after variable periods (hourly to daily). As expected, the gas production was found in all cases to be very constant: all the coefficients of determination of the hourly measurements presented in Figure 25 range from 0.975 to 0.999. Daily biogas measurements performed in BWC for over 60 d showed a coefficient of determination of 0.989. Future biogas measurement campaigns therefore require much fewer readings.

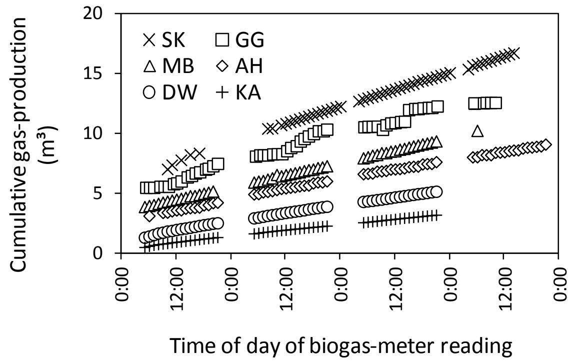

Figure 25: Cumulative biogas production over three to four days measured at six plants

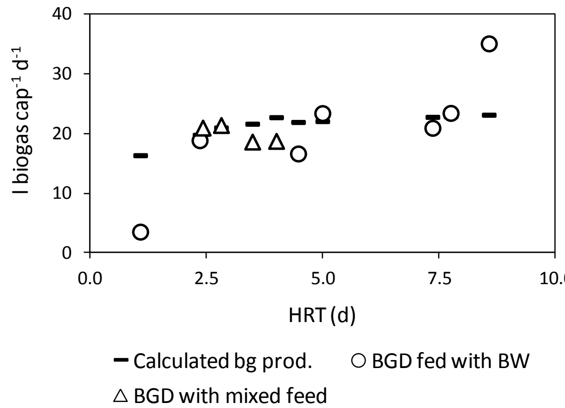

Figure 26: Per capita biogas production depending on the HRT of the pre-treatment

The main design parameter currently used for communal BGDs is the HRT. Figure 26 compares the per capita biogas production to the average HRT of the BGDs and to calculated predictions based on Sasse (1998). The measurements were conducted at eight sites. Each round data point represents one site each, each triangular data point represents the outcomes of several measurement campaigns performed at the same plant.

The average biogas-production of all measurements is 20 l cap^{-1} d^{-1} with a relative standard deviation of 36% across the systems. No directly comparable literature was found on biogas-digesters treating purely communal wastewater at such low retention times. Garuti et al. (1992) reported a biogas production of approximately 11 l biogas cap^{-1} d^{-1} with 73% methane content at the communal wastewater fed ABR they investigated. Table 17 summarizes per capita biogas production rates of black-water-fed biogas digesters reported in literature. Lohri et al. (2010) and Pipoli (2005) reported similar or lower rates than observed in this study. The comparably high value of 41 l cap^{-1} d^{-1} reported by Zurbrügg et al. (2011) is due to the addition of kitchen waste which is known to significantly increase biogas production (Lohri et al., 2010). It is remarkable that the BGDs presented here had similar biogas production as reported in literature although they were operated at far shorter HRTs.

Table 17: Documented biogas production of communal biogas-digesters

l biogas cap^{-1} d^{-1}	CH_4 biogas content	Feed	HRT (d)	Reference
28	>70%	black-water	15	Lohri et al. (2010)
41	approx. 60%	black-water + kitchen waste	37	Zurbrügg et al. (2011)
13	approx. 65%	faeces	20	Pipoli (2005)

The observed per capita biogas production rates are comparably constant across all investigations for HRTs between 2.5 d and 7.5 d. In other words, no significant increase of per capita biogas-production can be observed with HRTs above 2.5 d. Since the biogas-production correlates with COD removal the data indicates an optimal BGD design with an HRT of about 2.5 d.

Surprisingly, for similar HRTs the BGD fed with black-water and grey-water show a similar biogas production to systems purely fed with black-water (see Figure 26). This would indicate that under the situations under which DEWATS-BGDs operate, the HRT has a stronger influence on their treatment efficiency than the feed-concentration or feed-composition.

Figure 26 contains the prediction for biogas-production as given by the commonly-used design spread sheet for “one chamber settlers” (Sasse, 1998) under the prevailing loading conditions. With the exception of two outliers, the field data shows a good match to the prediction.

Methane concentration estimations performed with a Brigon Testoryt on the biogas produced by the digester in BWC yielded a CH_4 biogas content of approximately 83%. This value is surprisingly high but was confirmed by measurements performed by an external laboratory using gas-chromatography which yielded a CH_4 biogas content of 81%. Methane biogas content was measured by Rochmadi et al. (2010) at a communal DEWATS-BGD in Indonesia which yielded an even higher value of 88%. Lohri et al. (2010) reports 78% CH_4 content of biogas produced by BGDs fed with black-water.

4.3.4. Sludge accumulation rates in ABRs

Sludge accumulation rates were determined through linear regression of total sludge-volumes in ABR chambers over periods undisturbed by desludging events (see Section 3.4.4). Figure 27 presents the accumulation rates of six systems normalized over the number of connected users and depending on the HRT of the pre-treatment step. No clear correlation can be observed between sludge accumulation rates and the HRT of the primary treatment or its type.

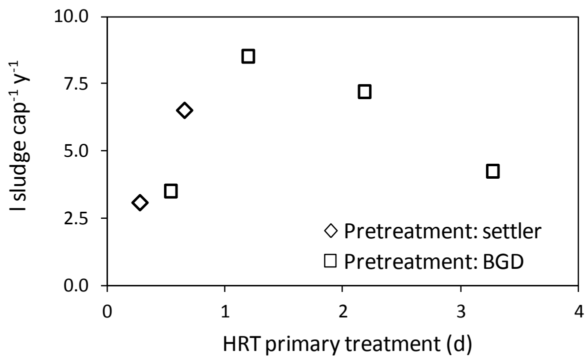

Figure 27: Per capita settled sludge accumulation depending on the HRT of the pre-treatment

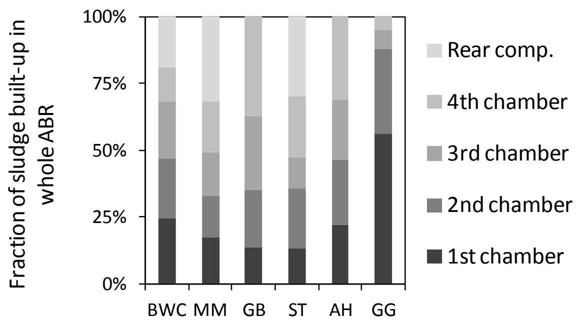

Figure 28: Fraction of total ABR sludge build-up inside chamber as measured in 6 plants

The average accumulation was found to be 5.5 l cap^{-1} y^{-1} with a large standard deviation of 40%. Foxon (2009) reports 0.9 m³ settled sludge accumulation per year when running an ABR at 42 h HRT. With a total reactor volume of 3 m³ and an estimated per capita wastewater production of 80 l cap^{-1} d^{-1}, this corresponds to a sludge accumulation rate of about 63 l cap^{-1} y^{-1}. This is more than seven to twenty times greater than the rates presented in Figure 27. The reason for this is hypothesised to be the nature of the feed which in Foxon's study, although being screened, was not pretreated.

No further literature values could be found in order to directly compare these results. Accumulation rates of onsite primary treatment reported in literature are given in Table 18. Sasse (1998) also cites Garg (unknown year) with a build-up rate of 30 l cap^{-1} y^{-1} in septic tanks.

The sludge accumulation in the investigated ABRs is evidently much lower than in a septic tank. The first, obvious, reason is that ABR feed is pretreated whereas this is not the case for septic tank influent. It is further hypothesised that anaerobic sludge stabilisation, and therefore volume reduction, occurs more efficiently in an ABR than in a septic tank.

Table 18: Documented sludge accumulation rates of onsite primary treatment technology

Sludge accumulation*	Technology	Comment	Reference
340	Septic tank		Koottatep et al. (2014)
40 - 1640	Percolation tank	depending on soil type	Koottatep et al. (2014)
18 - 70	Pit latrines		Still (2002)
90	Septic tank		Gray (1995)
60	Septic tank	after 3.5 y of operation	Philip et al. (1994)

* in l cap^{-1} y^{-1}

Figure 28 however also shows that about 50% of all sludge build-up inside the ABR can be expected to happen beyond the second chamber (except in GG, where almost all accumulation occurs in the first two chambers). The minimum and maximum measured accumulation rates imply an approximate sludge accumulation of 1.5 l cap^{-1} y^{-1} to 4.2 l cap^{-1} y^{-1} in the rear compartments. With a typical area of 2.5 m x 0.7 m, a total of 5 ABR chambers and 300 connected users, this leads to a 90 mm to 240 mm sludge height increase per y in the 3 rear ABR chambers. DEWATS ABR O&M manuals state that an ABR needs to be desludged after 2 y to 3 y when sludge-blankets have reached a height of about 1 m. The available measurements however suggest an ABR desludging frequency of the last three ABR chambers of at least 4 y. Since highest sludge activity is expected to be found in the first reactor chambers (see Section 6.7.2.4) it is proposed to never desludge these.

Sludge-heights inside settlers however have to be expected to accumulate faster than inside ABRs.

4.4. Conclusions

The datasets presented here enable the comparison of a number of design estimations with field-data. It is expected that this will consolidate the basis of future DEWATS designs and support monitoring and operation as well as maintenance procedures.

The available data on per capita wastewater production in SSS show an average of 85 l cap^{-1} d^{-1} in Central Java/ Indonesia where water is freely available. The per capita wastewater production in poor and water scarce areas in Bangalore/ India was found to be as low as about 30 l cap^{-1} d^{-1}.

Wastewater production in Central Java was not found to correlate with average household income.

Long-term fluctuations in wastewater production of communities connected to DEWATS were found to be about 20%. The average diurnal peak-flow factor is 1.9 with a relative standard deviation of 20% and the strongest peak generally occurring in the morning for a duration of 2 h to 3 h.

The average BOD_5/COD ratio in DEWATS effluent was found to be 0.46 with a relative standard deviation of 38%.

Effluent COD concentrations above 100 mg COD l^{-1} show a variation of about 13% to 17% over several years. Effluent COD concentrations below 100 mg COD l^{-1} vary more strongly. Diurnal variations of effluent COD concentrations were found to be statistically negligible.

Nutrient concentrations in the effluent of anaerobic DEWATS treatment steps are high and can exceed 100 mg NH_4-N l^{-1} and 15 mg PO_4-P l^{-1}. This is attributed to the comparatively low per capita wastewater production in certain project areas since the per capita nutrient loads remained approximately constant across all sites.

The measured average biogas-production of communal DEWATS BGDs is 20 l cap^{-1} d^{-1} with a relative standard deviation of 36% across the systems.

No significant increase of per capita biogas production can be observed with HRTs of above 2.5 d and it is proposed to use this value for the dimensioning of BGDs operating under DEWATS-typical circumstances. Field data compares reasonable well to the biogas production estimation by Sasse (1998).

The sludge build-up rate in ABRs is on average 5.5 l cap^{-1} y^{-1} with a relative standard deviation of 40% across reactors. Approximately 50% of the total sludge accumulation occurred downstream of the first two compartments. Based on the available data, previously estimated desludging intervals of 2 to 3 y could be extended to at least 4 y. It is proposed never to desludge the first two ABR chambers since most sludge activity is expected to take place in the first chambers. Settlers are expected to need more frequent desludging and it should be investigated whether this excess sludge could be transferred to the ABR.

5. SCREENING STUDY

5.1. Objectives

Many assumptions made during the DEWATS dimensioning process remain unverified to date. As a result of this, the relation between loading and treatment efficiency of the systems remains uncertain. It is however a criteria which directly influences the dimensioning and therefore the costs. Also, due to their small size, the fluctuating nature of communal wastewater, the varying commitment of operators and many other factors, DEWATS plants are exposed to extremely diverse conditions. Although DEWATS are known to generally fulfil local discharge standards, until now, no survey has been broad enough, both geographically and in terms of numbers, to assess the tolerance of the systems towards such variations. This chapter addresses these gaps by presenting and discussing monitoring results covering 108 systems in six Indonesian provinces gathered during a survey conducted in 2011. The objectives are:

- ...to identify factors potentially affecting the system performance.
- ...to relate system-loading to effluent quality.
- ...to provide a broad view on the effluent concentrations which can be expected from DEWATS currently operating under field conditions and tropical climates.

To fulfil the first objective, available information on factors potentially affecting the system performance was compared to effluent concentrations. The considered factors were categorized as design details, feed characteristics and applied operation and maintenance practices.

5.2. Survey-specific methodology

5.2.1. The survey

The data presented in this chapter was produced during a survey commissioned and co-financed by the "Water and Sanitation Program" of the World Bank and led by Tri Wahyudi Purnomo and the author. The survey was conducted from September to November 2011 by BORDA staff on a random selection of DEWATS implemented by BORDA's partner network in Indonesia. The survey comprised of a total of 298 communal BORDA systems from which data was gathered during once-off field-visits and community meetings. From the total pool of visited DEWATS, effluent concentration measurements were performed on 108 systems which are further considered in this chapter.

Apart from technical issues, the survey also considered non-technical factors concerning social, financial and institutional aspects that exceed the scope of this thesis. A detailed discussion of these can be found in Eales et al. (2013).

5.2.2. The surveyors

The surveyors were mainly BORDA staff with 1 y to 5 y experience with BORDA projects. Teams performing field investigations always consisted of at least two surveyors of whom at least one was a BORDA staff member. The teams were in most cases supported by local partner NGO staff or experienced field facilitators from local governments. The surveyors were trained over 4 d on how to conduct the surveys, field measurements and data-input. The training was followed by two weeks of survey implementation in the field followed by reassessment of the staff. Crosschecks on surveyors and questionnaires were performed by contacting some of the visited CBOs.

5.2.3. The plants

One-off effluent COD_t concentration measurements were performed at 108 plants. Loading estimation was possible for 74 of these systems.

Visited plants were SSS, CSC, Mixed or SBS systems. They were chosen by minimizing the travel-time for the maximum possible number of visited systems. Initially, the geographical positions of all existing BORDA plants were visualized on maps of each province. Figure 29 shows exemplarily the projects mapped in Central Java. Clusters of high geographic project density were then identified and geographically isolated projects excluded from further consideration. In clusters with numbers of projects too large to be completely covered by this survey, the number of visited projects was randomly halved by choosing every second plant on the alphabetically ordered project code list. This was done in such a manner as to keep the ratios between project types constant within the respective cluster. This methodology had to be used for plants in East Java and West Java.

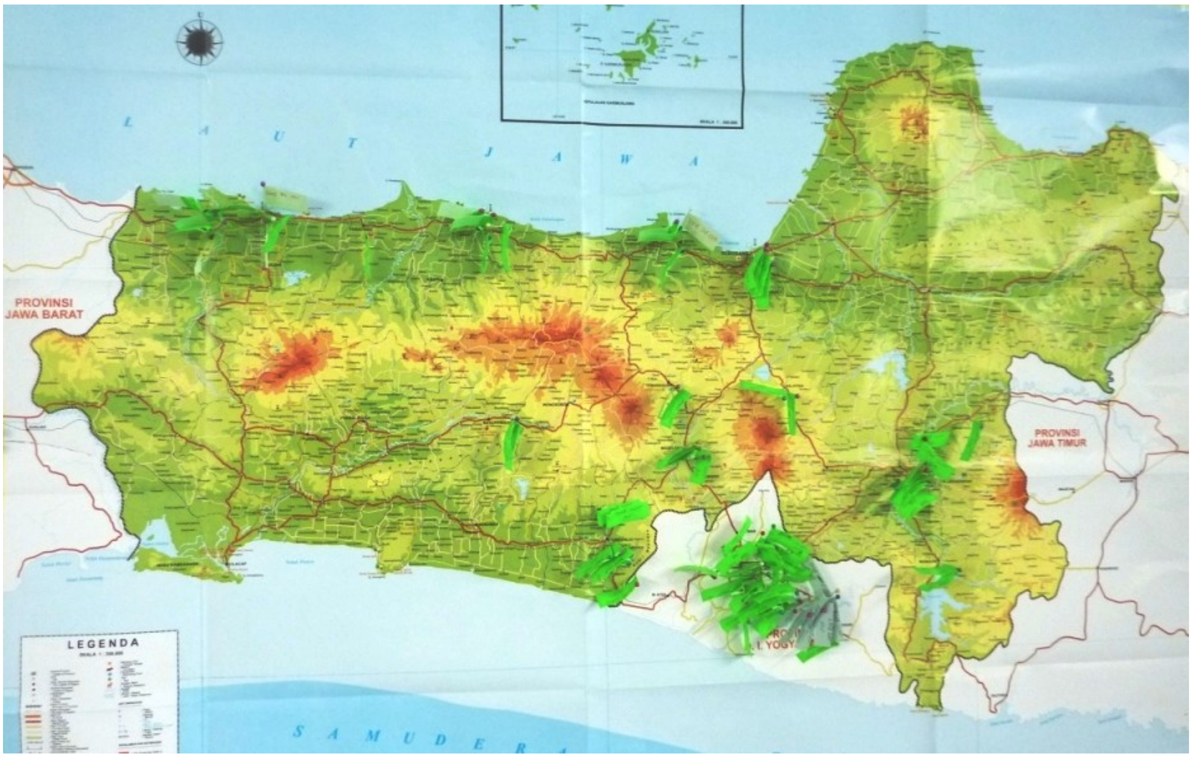

Figure 29: Map of Central Java where each flag represents the location of one DEWATS

The investigated systems are often unique in terms of reactor configuration and size because of local requirements and constraints taken into account at the design stage. Their setups always consist of a settling unit (either a BGD or settler) followed by an ABR with a varying number of compartments. Further anaerobic treatment is achieved by an AF. BGDs at the visited sites are always fed purely with black-water. Grey-water bypasses this treatment step and joins the treatment in the following reactors. Communal BORDA DEWATS plant configurations in Indonesia do not generally include aerobic treatment steps such as PGFs. Table 19 lists the number of plants for which effluent concentration and loading data was available by system type, whether or not their design includes a BGD, and the province in which they are located. The plants were built between 1998 and 2009 and had been operating for at least 12 months before monitoring. It is thus assumed that they had all reached stable operating conditions by the time the data was collected.

All projects have been implemented using a community participative approach. The plants were all exposed to similar temperatures of 28°C to 32°C throughout the year. In the case of Central-Java, East-Java and Bali, investigations were conducted at the end of the dry season, therefore reducing the potential influence of storm water on the system conditions. The plant visits in West-Java and Sumatra however coincided with the beginning of the wet-season.

Table 19: Number of plants depending on system type, pre-treatment and location presented in this chapter

System type				BGD as pre-treatment	
SSS	CSC	Mixed	SBS	Yes	No
21/15*	72/44	11/11	4/4	87/59	21/15
Province					
North Sumatra	West Sumatra	West Java & Banten	Central Java & Yogyakarta	East Java	Bali
6/6	9/5	22/6	39/35	22/16	10/6

*"Number of plants at which effluent concentrations were measured"/"Number of plants for which loading could be estimated"

5.2.4. Wastewater parameters and compliance

The pH was measured using Merck indicator paper. The COD_t measurements were conducted on grab-samples and analysed with Merck Nova 60 and Hanna Instruments HI 83214 spectrophotometers. The measurement-accuracy of the devices was regularly checked throughout the campaign with the help of standard tests every time the spectrophotometer had been moved. Fresh-water conductivity was investigated as a substitute to salinity-measurements with Hanna Instruments DiST4 handheld devices.

National discharge standards often define maximum effluent COD_t concentrations (see Appendix A1). Indonesian and Vietnamese national standards for the discharge of treated wastewater to open water bodies do not define maximum COD_t but only BOD_5 concentrations: 100 mg BOD_5 l^{-1} and 50 mg BOD_5 l^{-1} respectively. The maximum allowed TSS concentration in both cases is 100 mg l^{-1} (see Appendix A1 for a more complete set of parameters).

COD_t measurements were chosen over BOD_5 measurements for field investigations due to their comparable ease of handling and affordability. Investigations on combined COD_t and BOD_5 concentrations of communal DEWATS effluent showed an average ratio of 2.2 (see Section 4.3.2.1). Kerstens et al. (2012) measured a slightly higher average ratio of 2.5 during their investigations on BORDA DEWATS in Java. No effluent TSS concentration measurements were performed as part of the study presented here. The dataset published by Kerstens et al. (2012) however suggests a DEWATS effluent COD_t to TSS ratio of 2.5 to 3.

Based on this, effluent concentrations below 110 and 220 mg COD_t l^{-1} will in the following be considered as being compliant with Vietnamese and Indonesian effluent standards respectively.

Effluent samples were taken as single grab-samples from the effluent of the last treatment step, the AF.

The available effluent COD_t data therefore represents one measurement per plant. Measurements were taken at varying times of the day. The variation of COD_t concentration across the day was found not to be statistically significant (see Section 4.3.2.2).

Long-term variation of COD concentration in communal DEWATS effluent is approximately 15% with variations of above 30% for concentrations below 100 mg COD_t l^{-1} (see Section 4.3.2.2). The uncertainty of effluent COD_t measurement values is simplified to 20% in the following.

5.2.5. Influence of external factors on effluent concentrations

Information was gathered on design details, feed characteristics and applied operation and maintenance practices during the survey and through the examination of project documentation. Collected design details included: location (province), location (coastal or inland), system type, inclusion of BGD in the design and date of implementation. Information about feed characteristics was available on exposure to storm-water (see Section 3.4.3 for more details), occurrence of rain 24 h before sampling, salinity of fresh-water and general water scarcity at the site. Data on applied operation and maintenance practices (O&M) in the projects was gathered concerning the existence of a CBO and operator as proxies for the existence of a management and maintenance structure. Information on the occurrence of desludging of systems older than 3 y and O&M training of the operator and users were recorded as indicators for the functioning of the management structure. Additionally information on the use of biogas was gathered for each system including a BGD in its design.

Statistical testing of this data was attempted by segregating the effluent concentration dataset into subgroups depending on the respective parameter characteristics. The effluent concentration data of these subgroups were tested for normality using Shapiro-Wilk tests. Most of the test runs however indicated that the subgroup data were not normally distributed, even after logarithmic or potential data transformation. Their difference of means could therefore not be tested for significance using unpaired sample t-Tests (for 2 subgroups) or one-way between subjects ANOVA (for more than two subgroups). As a result, the subgroup-data is presented as effluent concentration frequency distributions to assess difference between subgroups visually.

5.2.6. Approximation of system loading

System loading is expressed as the "number of connected people per m³ reactor volume". The number of connected people is used as a substitute for the unavailable information on organic and hydraulic load and was estimated during meetings with the heads of the communities. The error of this estimate due to faulty or non-planned connections, fluctuations in population and per capita loading rates is estimated to be 20%.

The total reactor volume is calculated with values found in the available plant documentation, with the simplifying assumption that BGD, settler, ABR and AF have comparable treatment efficiency per reactor volume.

Data of 129 Indonesian BORDA DEWATS plants was available for which the design number of connected users could be normalized with the total reactor volume. This was done in order to identify whether a significant difference exists in the design sizes of the different system types. The plants were CSC, SSS, Mixed systems or SBS. Table 20 presents statistical information on the design user number per m³ reactor volume depending on system type and whether their design contained a BGD or not. SSS, CSC and SBS generally have the same size for the same number of assumed users. Mixed systems appear to be on average slightly larger with 3.8 users per m³ reactor volume, but the standard deviation of the available data indicates that the difference to the other system types is not significant. There also is no significant difference between systems designed with or without BGD. It is therefore concluded that all system types are by and large similarly sized during design. The available load estimations can therefore be compared across system types.

Table 20: Statistical information on the design load (cap m^{-3} reactor volume) of 129 BORDA DEWATS differentiating between system types and BGD inclusion to the design

	SSS	CSC	Mixed	SBS	No BGD	With BGD	All
Mean	5.15	5.01	3.80	5.07	5.15	4.84	4.94
SD	1.60	1.70	0.84	1.38	1.56	1.64	1.61
RSD	31%	34%	22%	27%	30%	34%	33%
Max	9.31	9.97	5.62	8.65	9.31	9.97	9.97
Min	2.79	2.48	2.77	3.43	2.79	2.48	2.48
n	32	73	12	12	40	89	129

5.2.7. Design system performance

The design and performance calculations proposed by Sasse (1998) were used to compute predicted effluent concentrations of a typical DEWATS design depending on system load. This was done in order to create benchmark values to which the available performance data could be compared.

The main treatment-influencing design parameter (at constant temperature) is the hydraulic load (see Section 2.3.7).

Typical design per capita wastewater production values were determined through available design information on 85 Indonesian DEWATS. Table 21 summarizes the statistical characteristics of this data depending on system type. SSS and Mixed systems were in most of the cases designed with about 80 l cap^{-1} d^{-1} to 100 l cap^{-1} d^{-1}. CSC and SBS were generally designed with slightly lower design per capita wastewater production values since users of communal ablution facilities were assumed to produce less wastewater than household members. Maximum design values are however similar in all cases. Minimal design values vary stronger across system types and can be as low as 55 l cap^{-1} d^{-1} for SSS. The comparably strong variation of design values is due to varying case-specific safety factors and budgeting constraints influencing the design (personal communication with senior BORDA design staff).

Table 21: Per capita wastewater production such as used for the design of 85 Indonesian BORDA DEWATS

	SSS	CSC	Mixed	SBS
Mean	91	65	94	75
SD	12	12	9	17
RSD	14%	18%	10%	23%
Max	102	100	105	100
Min	55	41	75	55
n	26	42	11	6

Field investigation results on per capita wastewater production at SSS plants in Java/Indonesia and Bangalore/India are presented in Section 4.3.1.1 with minimal and maximal values of 20 and 130 l cap^{-1} d^{-1}. Most of the field measurements however confirmed the design assumptions of 50 to 100 l cap^{-1} d^{-1}. The number of investigated flow rates however was too small to interpret the measured rates below and above this range as outliers. Also, 20 l cap^{-1} d^{-1} appears plausible for CSC and SBS systems if users mainly use the toilets. Crous (2014) and Roma et al. (2010) found that the water demand in South African CSC type installations in informal settlements is 36.6 and 35 to 40 l cap^{-1} d^{-1} respectively. Water is used in these systems for showers, toilets, hand wash basins and laundry. Zimmermann et al. (2012) reported 20 l cap^{-1} d^{-1} as design value for a school based sanitation facility treating wastewater with DEWATS technology in South India.

The confidence limits of the effluent concentration modelling predictions were therefore computed with per capita wastewater productions of 20 and 130 l cap^{-1} d^{-1}.

Further benchmark plant design specifications are: 250 users, per capita BOD_5 production of 60 g d^{-1} cap^{-1}, time of most water flow of 12 h and wastewater temperature of 28°C. The number of reactor chambers is two, five and two with reactor sizes of 13.8 m³, 24.0 m³ and 17.2 m³ for settler, ABR and AF respectively. The AF filter medium characteristics were set at 100 m² m^{-3} specific surface, 40% voids in filter mass and 950 mm medium height.

Load variations were achieved by keeping the plant setup constant and varying the user number. For each load, the design and performance calculations proposed by Sasse (1998) yield a predicted effluent COD concentration. Finally the confidence limits of the design performance predictions take 20% uncertainty of the measured COD into account. The results of this are shown on Figure 56.

5.3. Results and discussion

5.3.1. Design information of plants

Figure 30 and Figure 31 present the design user numbers and total reactor volumes of systems for which effluent COD_t concentration data was available. This design information was retrieved from project documentation and was only available for 76 and 77 systems respectively. By far most visited systems were designed for 200 to 400 connected users with a total reactor volume of 35 m³ to 90 m³. The two largest systems projected for 750 and 856 users were built with 126 m³ and 150 m³ total reactor volume respectively.

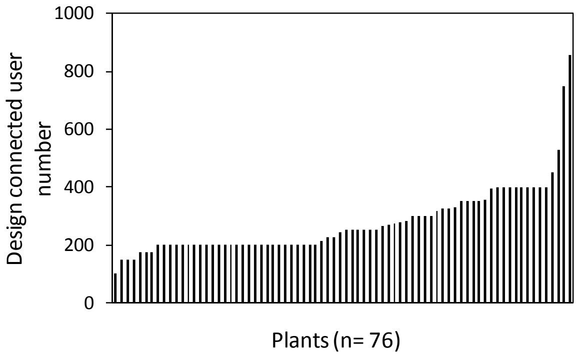

Figure 30: Design user-number of visited plants

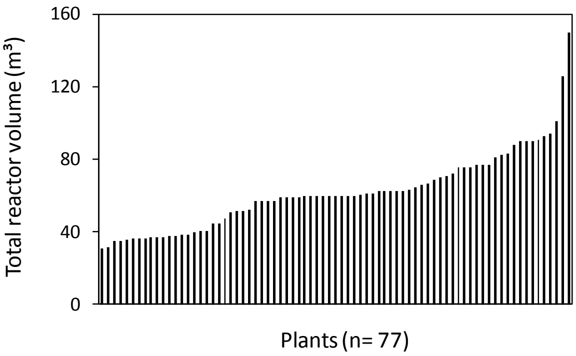

Figure 31: Size of visited plants

5.3.2. Effluent concentrations

All measured effluent pH values were between 6.5 and 7.5, and therefore in the range of national effluent standards (compare to Appendix A1). These values also fall into the range necessary for the establishment of an active anaerobic MO population and indicate generally good anaerobic treatment conditions (Batstone et al., 2002).

The available COD_t effluent concentration data quality was cross-checked by comparing it to conductivity measurements that were done on the well water commonly used at the sites. Water-conductivity is used as a direct indicator for salinity which interferes with the used COD measurement method at concentrations above 2,000 mg Cl^- l^{-1} (APHA, 1998)[10]. The raw-water measurement of this parameter is representative for the condition throughout the DEWATS treatment since salinity is not altered by anaerobic digestion. Figure 32 shows the relationship between conductivity and salinity of a solution. The critical value for COD analysis is thus exceeded at a conductivity of about 6 mS cm^{-1}. Figure 33 puts the measured effluent COD_t concentrations in relation to the raw-water conductivity. Six of the concentration measurements were therefore performed on wastewater having almost or exceeding a conductivity of 6 mS cm^{-1} and are thus excluded from further analysis. Five of these six sites were located in the coastal cities of Semarang, Pekalongan and Padang within a few hundred metres of the sea-shore. The cause of the high conductivity measured at the sixth site which is located inland in the town of Surakarta, approximately 100 km away from the closest coast, remains unknown. Data in relation to conductivity values below 6 mS cm^{-1} is further discussed in Section 5.3.4.2.

[10] High salinity of groundwater is a common problem in high density population coastal areas. It is caused by sea-water intrusion into over-exploited aquifers.

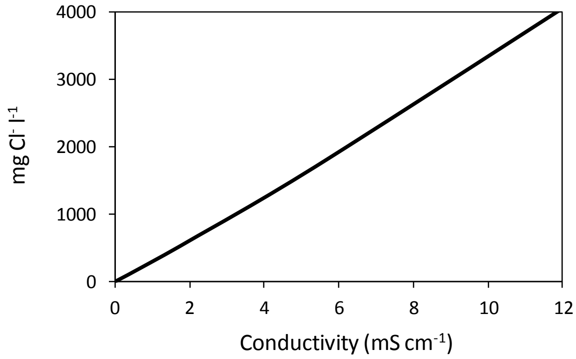

Figure 32: Relationship between conductivity and Cl^- concentration in a solution (Lide, 1997)

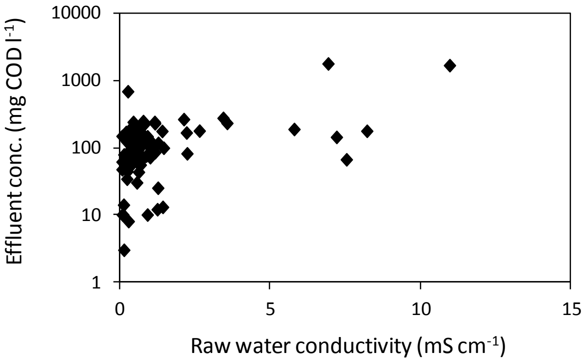

Figure 33: DEWATS-effluent concentration vs. raw-water conductivity measurement results

Two of the samples removed from further analysis because of their elevated conductivity yielded extremely high effluent concentrations above 1,500 mg COD_t l^{-1} (see Figure 33). Such high concentrations could also be explained by high solid content of the sample since it is very uncommon for communal wastewater to contain such levels of solubilised organic material. Turbidity or filtered COD concentration were unfortunately not measured. Pictures of the samples however indicate that the samples were free from obvious particulate contamination (see Figure 34 and Figure 35). The high measured COD concentrations are therefore attributed primarily to analytical errors due to the high salinity of the water.

Figure 34: Sample with measured COD concentration of 1,747 mg COD_t l^{-1}

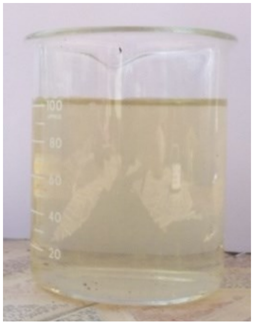

Figure 35: Sample with measured COD concentration of 1,649 mg COD_t l^{-1}

Figure 36: Sample with measured COD concentration of 676 mg COD_t l^{-1}

Figure 37: Sample with measured COD concentration of 416 mg COD_t l^{-1}

The concentrations of two samples with low conductivity were particularly high with 676 mg COD_t l^{-1} and 416 mg COD_t l^{-1} measured at one CSC and one SSS respectively. This is surprising since both of the plants at which these measurements were performed were comparably low loaded (1.8 and 2.7 cap m^{-3} respectively, compare to next Section 5.3.3). The available sample pictures (Figure 36 and Figure 37) also do not indicate obvious sample contamination through particulate particles, although the wastewaters certainly were turbid. It is concluded that the COD_t measurements performed on these samples were most probably not representative for the general treatment of the plants they represent. The values were therefore not considered in further analyses. Field measurements would have to be repeated in future in order to verify this decision.

The available COD_t effluent concentration data quality was further cross-checked by comparing effluent concentrations with reported rain events within the 24 h prior to sampling. This has been done since most DEWATS are known to be affected by rain water intrusion (see Section 5.3.4.2) and some of the measurements had to be performed during the wet season.

Figure 38 presents the COD_t concentration measurements from plants not affected by high raw-water salinity. It highlights the effluent concentrations of the eighteen plants at which rain was reported within 24 h prior to sampling. Most visited plants in Sumatra are represented in this group as well as two plants located in West Java. In all eighteen cases, plant visits were performed at the beginning of the local wet season.

The results in Figure 38 strongly suggest that most effluents with concentrations below 50 mg COD_t l^{-1} had been diluted by rain water. Especially concentrations below 25 mg COD_t l^{-1} are unrealistically low and dilution through rain appears a very plausible explanation for these results. Acknowledging the influence of rain on low effluent concentration must lead to the exclusion of all measured concentrations coinciding with reports on rain. They were therefore considered as being not representative for system treatment and were not used for further analyses.

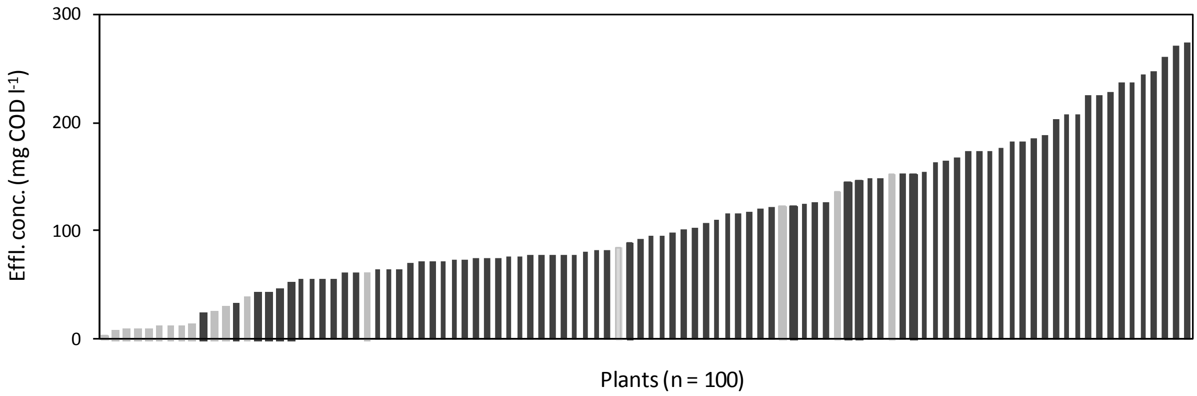

Figure 38: Effluent COD_t concentrations and rain occurrence prior to sampling (light columns represent sites where it rained within 24 h prior to sampling) at visited plants with raw-water conductivity below 6 mS cm^{-1} (n=100)

Figure 39 shows the measured effluent COD_t concentrations of the 82 visited systems not affected by high raw-water salinity and rain events and depending on system type.

The effluent concentrations vary widely from 47 to 204 mg COD l^{-1}, 25 to 274 mg COD l^{-1}, 65 to 185 mg COD l^{-1} and 55 to 208 mg COD l^{-1} for SSS, CSC, Mixed and SBS respectively. CSCs therefore exhibit the widest effluent concentration variation range of all four system types whereas the remaining three are similar in that respect.

Kerstens et al. (2012) investigated the effluent concentrations of eight BORDA DEWATS systems built in Java of which five were SSS without and two were Mixed systems with BGDs as primary treatment. Effluent concentrations were measured on four different days for each system. The mean COD_t effluent concentrations were 122 (± 22) mg COD_t l^{-1} and 131 (± 53) mg COD_t l^{-1} for SSS and Mixed systems respectively[11].

[11] Numbers in brackets represent standard deviations

Rochmadi et al. (2010) reported a COD_t effluent concentration of 61 mg COD l^{-1} for BORDA DEWATS systems built in Indonesia without detailing whether this is the outcome of single or multiple measurements at one or multiple systems.

Nguyen et al. (2007) reported an average COD_t removal of 88% by a four chamber ABR. The reactor was designed for the treatment of wastewater from twenty households and livestock breeding in Vietnam. Feed COD_t concentrations are reported to be high, averaging around 2,500 mg COD_t l^{-1} which results in an average effluent concentration of 300 mg COD_t l^{-1}.

Foxon (2009) observed average effluent concentrations of 130 (± 29) mg COD_t l^{-1} during the operation of a pilot plant ABR fed with screened communal wastewater.

The system setups presented by Nguyen et al. (2007) and Foxon (2009) did not include an AF as opposed to the plants presented here. The effluent concentrations they observed would therefore be expected to be larger than those of the systems presented here.

The available literature on treatment efficiencies of ABR-type systems operating under or close to field conditions therefore only confirms effluent concentrations up to a maximum value of about 200 mg COD_t l^{-1}. Higher observed effluent concentrations are therefore considered to be above literature values.

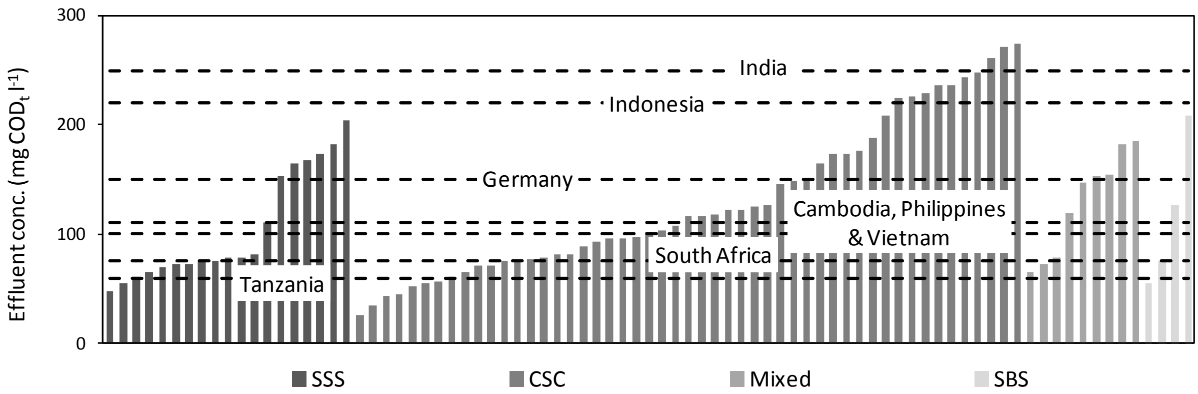

Figure 39: Effluent COD_t values at visited plants not affected by rain water and with raw-water conductivity below 6 mS cm^{-1} (n = 82), the dotted red lines represent national standard discharge COD concentration values for various countries

Figure 40 a and b present the frequency distributions for SSS and CSC effluent concentrations. (Data-points available for Mixed and SBS were too few to produce meaningful histograms.) Most SSS effluent concentrations were below 100 mg COD_t l^{-1} or between 151 and 200 mg COD_t l^{-1} with very few intermediate values between 101 and 150 mg COD_t l^{-1}. Values measured at CSC outlets were more evenly distributed across the concentration ranges with most values being between 51 and 150 mg COD_t l^{-1} and 12% of all effluent concentrations being between 201 and 250 mg COD_t l^{-1}.

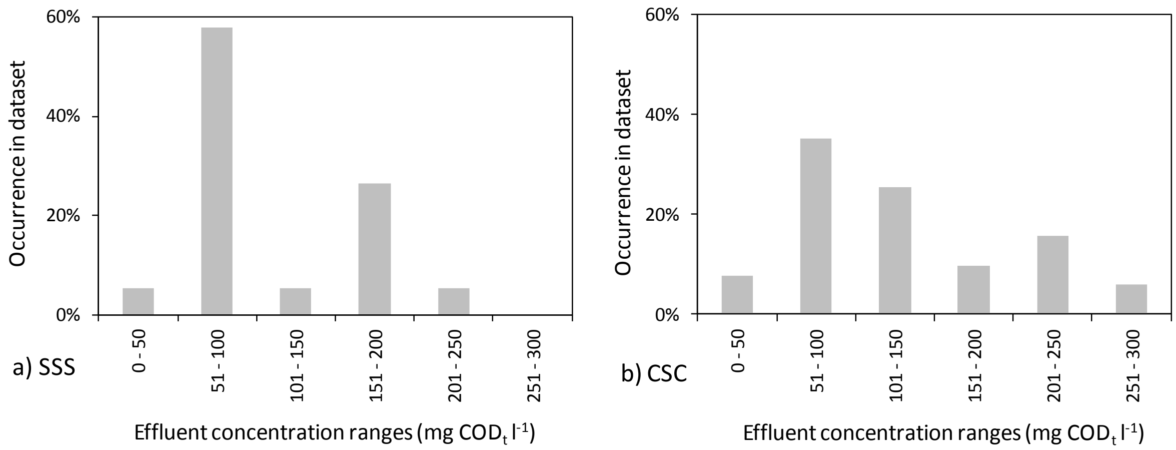

Figure 40 a and b: Histograms showing the effluent concentration frequency distribution for SSS and CSC system types

The dotted lines in Figure 39 represent the widely varying national discharge standard COD_t concentration values relevant for communal DEWATS as stipulated for Tanzania, South Africa, Cambodia, the Philippines, Vietnam, Germany, Indonesia and India (see Appendix A1 for further details). The maximal discharge standard value for agricultural reuse of wastewater for South Africa is 400 mg COD_t l^{-1}. It therefore exceeds the presented concentration range and is not shown in the figure.

Table 22 summarizes the percentages of effluent concentration measurements complying with national discharge standards depending on system type. The available data indicates generally good compliance of 88% to 96% with standards for discharge into surface waters in Indonesia and India. The available data however suggests insufficient treatment for countries with more stringent regulations such as Vietnam, the Philippines, Cambodia and especially South Africa and Tanzania. In these cases the system design would have to be adjusted, for instance by adding aerobic polishing steps such as PGFs. Other possibilities are ground percolation if the hydro-geological conditions permit or agricultural reuse of treated wastewater for which the regulations are less stringent (see South African regulations).

The compliance levels vary slightly across system types with SSS generally having the largest fraction of plants adhering to the regulations. The statistical significance of these results however will have to be checked through further monitoring campaigns.

Table 22: Percentage of effluent COD_t concentration measurements complying with various national discharge standards for discharge to open water bodies (maximal effluent COD_t concentration is given in brackets)

	SSS	CSC	Mixed	SBS	All
Tanzania (60 mg COD_t l^{-1})	11%	14%	0%	25%	12%
South Africa, open water body (75 mg COD_t l^{-1})	37%	22%	22%	25%	25%
Cambodia (100 mg COD_t l^{-1})	63%	43%	33%	50%	47%
Philippines (100 mg COD_t l^{-1})	63%	43%	33%	50%	47%
Vietnam (110 mg COD_t l^{-1})	63%	49%	33%	50%	51%
Germany (150 mg COD_t l^{-1})	68%	69%	56%	75%	67%
Indonesia (220 mg COD_t l^{-1})	100%	80%	100%	100%	88%
India (250 mg COD_t l^{-1})	100%	94%	100%	100%	96%
South Africa, agricultural use (400 mg COD_t l^{-1})	100%	100%	100%	100%	100%

5.3.3. Plant loading

Out of the 108 systems for which the effluent concentration was measured, available data allowed the reactor load estimation for 74 plants.

Figure 41 shows the loading of 54 of these plants for which effluent concentration data was available which was not influenced by raw-water salinity or rain water. The loading is expressed as estimated user number per m³ total reactor volume. The average design load range represents the mean value of 4.9 and standard deviation of 1.6 cap m³ as shown in Table 20.

Twenty-three and therefore about half (43%) of the plants were less loaded than the average design load range. This phenomenon is observable for 54% of the CSCs and only for 15% of the SSS. Twenty-seven or 50% of the estimated loads fell within the average design load range whereas four plants exceeded it.

Both plants at which by far the most (> 12) users were connected per m³ reactor volume were boarding schools.

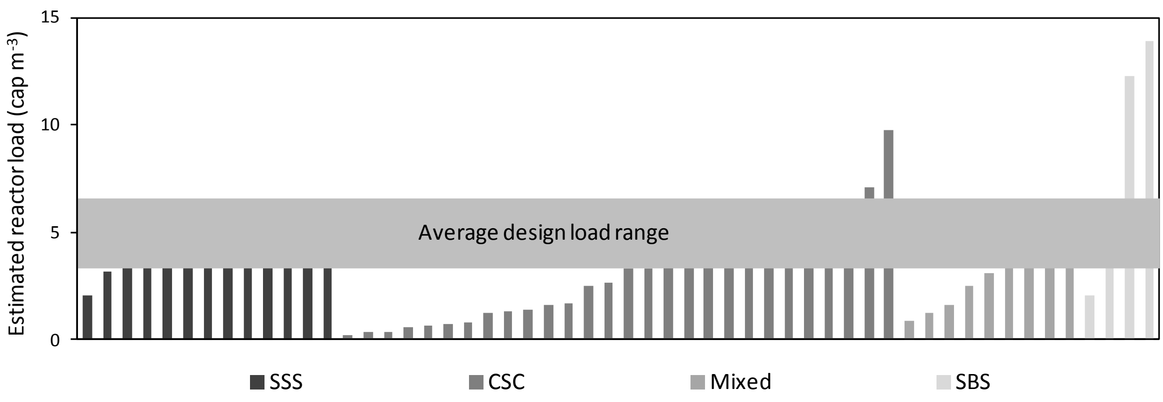

Figure 41: Loading estimation of plants (n = 54), the confidence range of the average design load is computed with the average load of 4.9 and the standard deviation of 1.6 cap m^{-3} (see Section 5.2.6)

5.3.4. Effluent concentration vs potentially influencing factors

5.3.4.1. Potentially influencing factors "Design details"

Figure 42 to Figure 44 present the effluent concentration frequency distributions depending on the potentially treatment-influencing design factors "Province in which the plant is built", "Plant built inland or coastal", "BGD inclusion to design" and "date of implementation".

Towns were defined as coastal towns when one of their administrative borders coincides with the shore line. These towns are: Brebes, Denpasar, Gianyar, Kendal, Medan, Pasuruan, Pekalongan, Semarang, Sidoarjo, Surabaya and Tegal. This grouping additionally includes Tangerang. Although not directly on the coast, this heavily industrialized zone is known to have very stressed aquifers reportedly leading to salt-water intrusion and land subsidence. Purnama and Marfai (2012) summarizes existing reports on saline-water intrusion to aquifers in the areas of Tangerang, Semarang, Surabaya, Denpasar and Gianyar.

None of the distributions shows a significant correlation between effluent concentrations and considered factors. It was however noticed that the majority of high effluent concentrations (> 200 mg COD_t l^{-1}) was measured in coastal cities.

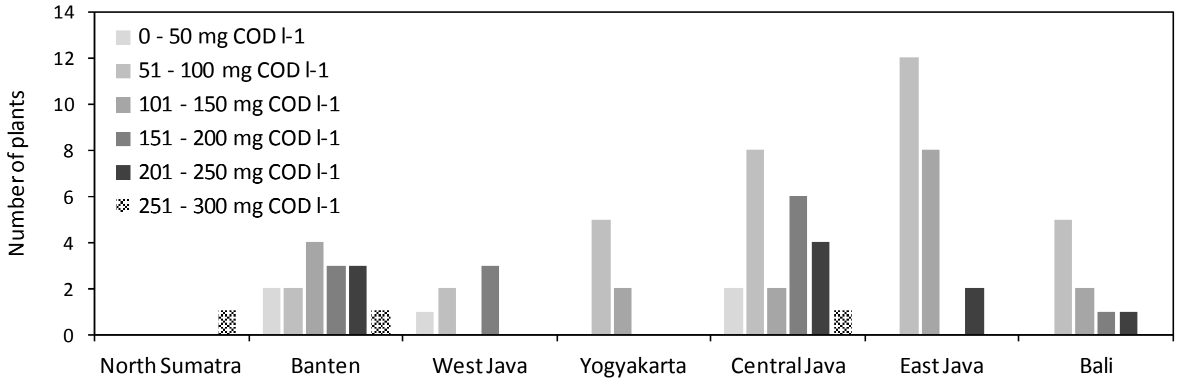

Figure 42: Histograms for the effluent concentration frequency distribution depending on province

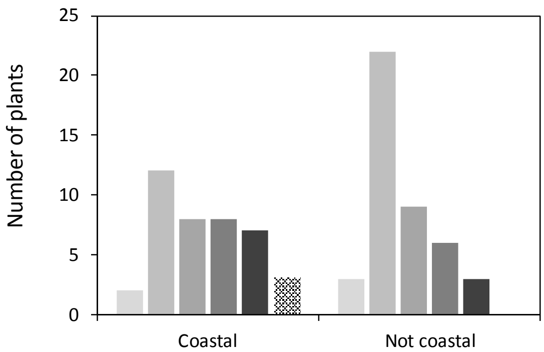

Figure 43: Histograms for the effluent concentration frequency distribution depending on whether the plant is built in a coastal town or inland

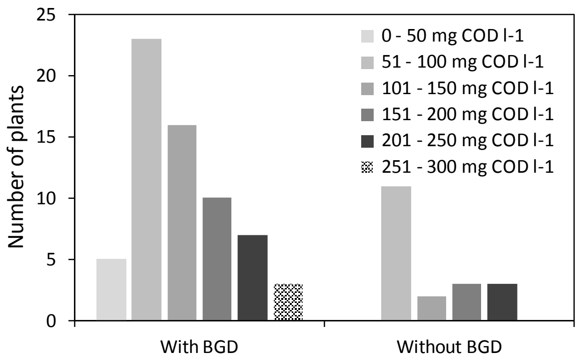

Figure 44: Histograms for the effluent concentration frequency distribution depending on BGD inclusion to design

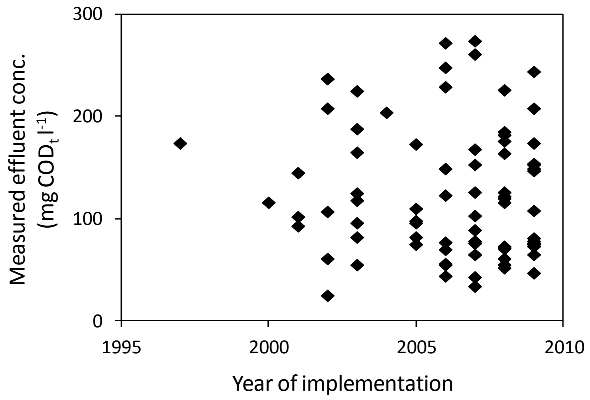

Figure 45: Effluent concentration and year of implementation

5.3.4.2. Potentially influencing factors "Feed characteristics"

Storm-water intrusion could be assumed to only play a role in SSS and Mixed systems where the sewer lines are by far longer and more exposed than in CSC and (generally) SBS. However, out of the twelve effluent concentrations below 50 mg COD_t l^{-1} which were probably rain affected (see Section 5.3.2) eleven were measured at CSCs and one at a Mixed system. Also, scum and water marks on the reactor walls indicating strong water level fluctuations within the reactors and therefore storm water intrusion (see Section 3.4.3 for more details) were not only made in all investigated SSS and mixed systems but also in most of the CSC and SBS (see Figure 46). The data therefore indicates that all system types were prone to be storm water affected during the wet seasons.

Figure 47 shows it made no obvious difference to the effluent concentrations whether signs of overflow were observed in plants.

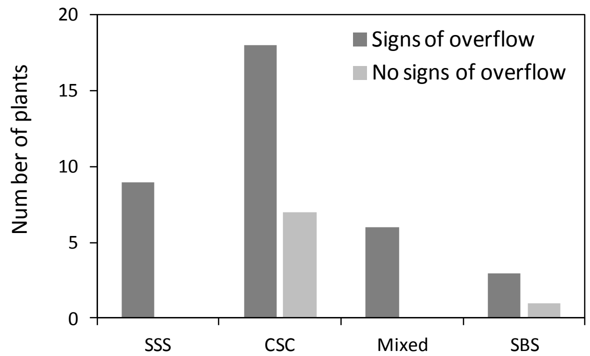

Figure 46: Observed signs of storm water exposure depending on system type

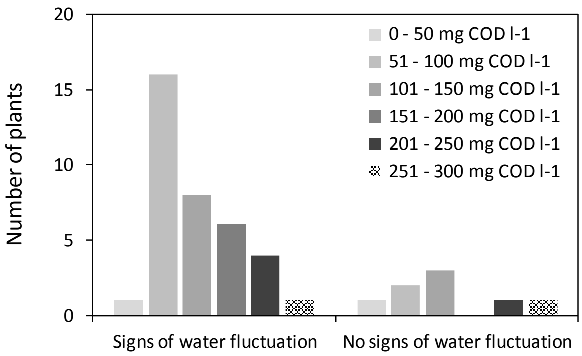

Figure 47: Histograms for the effluent concentration frequency distribution depending on observation of signs of strong water level fluctuations

The islands of Sumatra, Java and Bali were all formed through volcanic activity due to the subduction of the Indian oceanic plate beneath the Eurasian continental plate. Their hydro-geology is similar and most of the aquifers occur in Quaternary volcanic rock with water conductivity of natural springs therefore being approximately similar across the islands and reported to be about 0.1 mS cm^{-1} to 0.25 mS cm^{-1} in Central Java (Irawan et al., 2009). Higher groundwater conductivities inland however could be due to pollution.

Figure 48 compares raw-water conductivity measured in coastal towns and inland indicating that the electric conductivity of coastal groundwater samples was generally higher than inland ones. Almost all values above 1 mS cm^{-1} for instance, were measured at sites close to the coast. An unpaired 2-sample t-test (significance level 5%) confirmed a significant difference between coastal (M = 1.15, SD = 0.85) and inland samples (M = 0.44, SD = 0.23); t(68) = 1.72, P = $3*10^{-7}$ (Data was transformed potentially prior to testing.)

It was therefore concluded that the wastewater treated by DEWATS in coastal areas tended to have a higher salinity than inland.

Yeole (1996) states that electrical conductivity higher than 5 mS cm^{-1} indicates a salinity-content inhibitory to anaerobic digestion. This hypothesis could not be tested with the available data since the adopted COD measurement methodology cannot be used for samples with conductivities above 6 mS cm^{-1} (see Section 5.3.2). There was however anecdotal information given by one of the field investigators that one of the BGD operating at high salinity (approximately 11 mS cm^{-1}) produced large amounts of biogas. This is in line with other publications which report good anaerobic treatment at high wastewater salinity (Kimata-Kino et al., 2011; Liu and Boone, 1991; Ozalp et al., 2003). All authors also mention the sensitivity of anaerobic process to the change in salinity which, in the case of communal DEWATS, may occur at coastal systems through rain-water intrusion during wet seasons.

Figure 49 puts the measured effluent concentrations into relation with the conductivity of the raw-water measured at the sites. Although no clear correlation appears between the two parameters, high conductivity tends to coincide with elevated effluent COD concentrations. As mentioned above, good anaerobic treatment performance has been reported for systems operating under very high salinity concentrations. No literature however could be found on the inhibition of strongly under-loaded anaerobic systems through salinity. It appears plausible that such systems with lower mass transfer driving forces and therefore slower growth processes would be less resilient to feed water salinity.

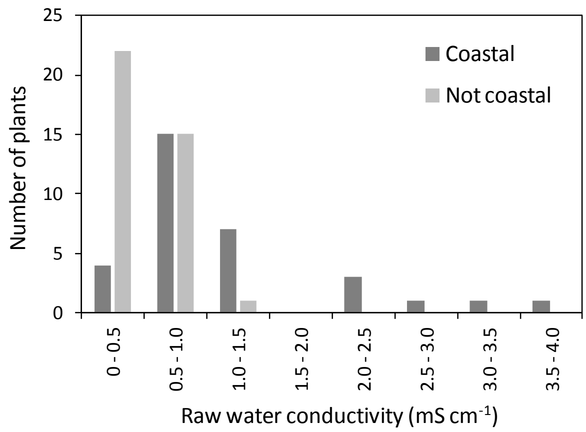

Figure 48: Histograms for the raw-water conductivity frequency distribution depending on whether a plant is built in a coastal area or inland

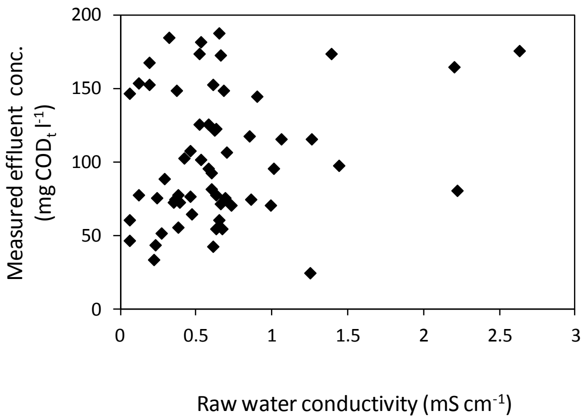

Figure 49: Raw-water conductivity and effluent concentration

5.3.4.3. Potentially influencing factors "Applied O&M practices"

Figure 50 to Figure 55 present the effluent concentration frequency distributions depending on potentially treatment influencing O&M factors. Factors are CBO existence, operator existence, biogas use, desludging of systems after three years of operation, occurrence of O&M training of operator and occurrence of O&M training of users.

None of the distributions shows a significant correlation between effluent concentrations and considered factors.

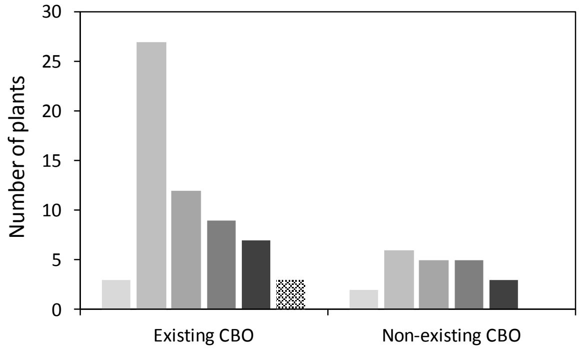

Figure 50: Histograms for the effluent concentration frequency distribution depending on CBO existence

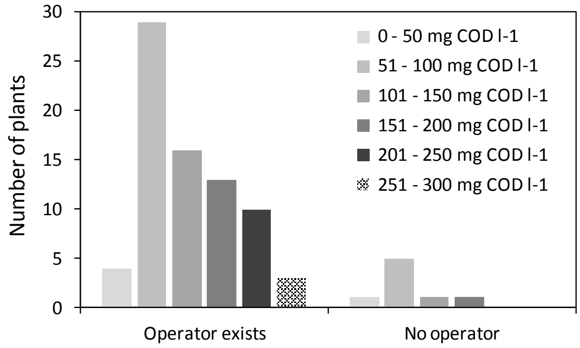

Figure 51: Histograms for the effluent concentration frequency distribution depending on operator existence

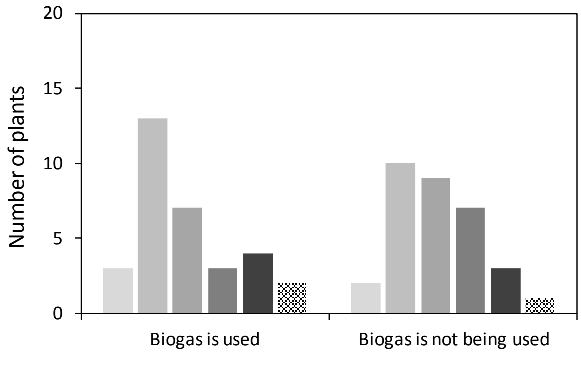

Figure 52: Histograms for the effluent concentration frequency distribution depending on biogas usage

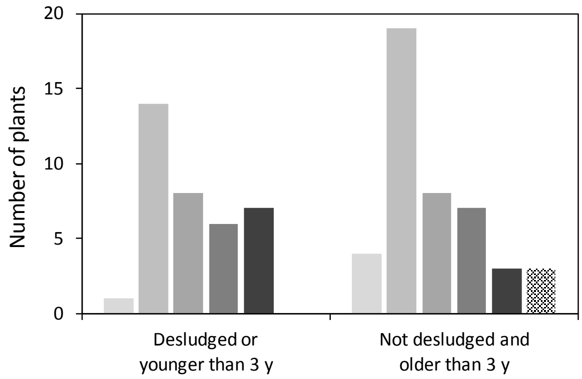

Figure 53: Histograms for the effluent concentration frequency distribution depending on desludging

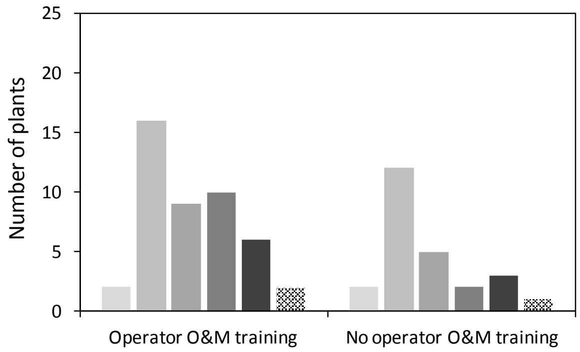

Figure 54: Histograms for the effluent concentration frequency distribution depending on operator O&M training

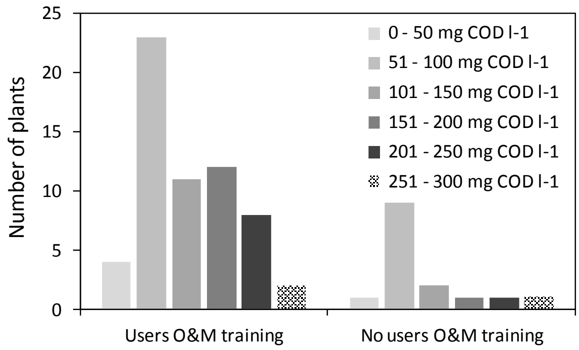

Figure 55: Histograms for the effluent concentration frequency distribution depending on user O&M training

5.3.5. Effluent concentration vs plant loading

Table 23 presents system characteristics of the 54 plants for which effluent concentration measurements and loading estimations were available. The table excludes all plants where the wastewater was presumably diluted by storm water on the day of sampling. COD data invalid due to high salinity of the sample was also not considered. It furthermore excludes two plants of which the particularly high effluent concentration measurement results were suspected to be not representative (see Section 5.3.2).

Table 23: Number of plants depending on system type, pre-treatment, location and year of implementation presented in this section

System type				BGD as pre-treatment	
SSS	CSC	Mixed	SBS	Yes	No
13	28	9	4	41	13
Province					
North Sumatra	West Sumatra	West Java & Banten	Central Java & Yogyakarta	East Java	Bali
1	0	6	26	16	5
Year of implementation					
2005	2006	2007	2008	2009	
2	10	12	13	17	

Figure 56 relates the load estimations of the plants presented in Table 23 to measured effluent concentrations. The average design system load range was computed as defined in Section 5.2.6. The upper and lower limits of the design performance prediction were calculated as described under Section 5.2.7.

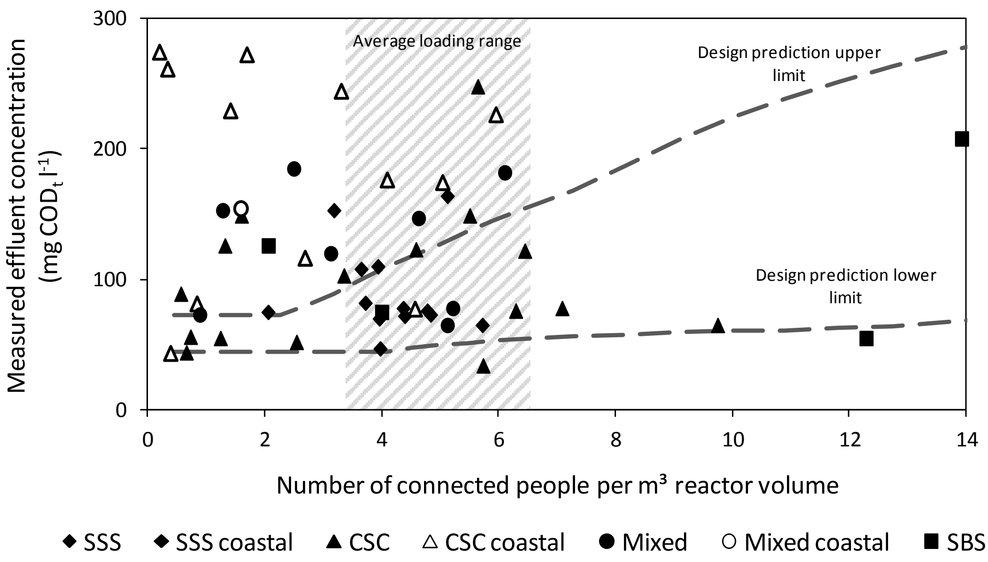

Figure 56: Effluent concentration values plotted against estimated plant loading expressed as number of connected people per m³ total reactor volume (n= 54). The curves "Design prediction upper/ lower limit" delimit the confidence range of the design system performance predictions taking into account a per capita wastewater production of 20 to 130 l cap^{-1} d^{-1} and 20% uncertainty in the COD concentration measurement, the confidence range of the average design load is computed using the average load of 4.9 and the standard deviation of 1.6 cap m^{-3} (see Section 5.2.6)

Most plants fall within or below the average design system load range with only four plants having a higher load. Under-loaded plants are mainly CSCs and Mixed systems. Most SSS fall within the average design system load range.

Surprisingly, the data indicates that reduced plant loading does not guarantee an improvement in effluent concentrations, for either of the system types. The data shows no clear correlation between loading and effluent concentration contrary to that generally expected from literature (see Section 2.3.5). Also the existing DEWATS design tool predicts an effluent concentration reduction with reduced plant load (see Figure 56). The effluent COD concentration measured at a large number of systems is however considerably higher than predicted by the design-tool, even exceeding national discharge standards (see Figure 56).

The lack of correlation between loading and effluent concentration is most apparent for CSCs of which low loaded systems show low, medium and high effluent COD_t concentrations above 200 mg COD_t l^{-1}.

Low system loading would limit the establishment of a stable anaerobic MO community which would consequently lead to little or no COD removal (Bischofsberger et al., 2005; Shen et al., 2004). But even when considering the uncertainties in measurement and loading estimation, the spread of effluent concentration values is such that other factors than the loading must strongly be influencing the treatment of these plants.

Table 24 therefore compares the available information on potentially treatment-influencing factors for the systems depending on their effluent concentration and estimated load. The plants are divided into two groups: "Complying with design" for concentrations within or below design expectations and "Not complying with design" for concentrations exceeding the design expectations.

The occurrence of flow surge signs, water scarcity and the various indicators for applied O&M practices does not seem to significantly differ between groups. It was evident however that most plants built in coastal areas had effluent concentrations above design range (also see the data points highlighted in Figure 56). It was shown in Section 5.3.4.2 that on average, wastewater treated by DEWATS in coastal areas has a significantly higher salinity than inland. Based on the available data it is therefore hypothesised that elevated raw-water salinities observed at plants built close to the coast, or the seasonal variation of salinity, may have negative effects on the treatment.

This however will have to be investigated further, also since the dataset does contain data points which contradict this general trend. Six systems for instance that had effluent concentrations comparable to design prediction are built in coastal towns.

Also a number of plants performing comparably poorly are inland and certainly not affected by elevated wastewater salinity. The reason for their poor performance remains unclear.

Table 24: Comparing potentially treatment-influencing factors of DEWATS with effluent concentrations within or above design effluent concentration range

Potentially treatment-influencing factors		Complying with design	Not complying with design
General	Total number of systems	33	21
	SSS	11	2
	CSC	16	12
	Mixed	3	6
	SBS	3	1
	BGD	22	19
	Systems in a coastal town	3	10
Feed	Signs of flow surge	16	11
	Fresh water conductivity > 1 mS cm^{-1}	1	7
	General water scarcity	1	1
Applied O&M practices	No CBO	1	1
	No operator	5	0
	No biogas usage	4	7
	Older than 3 y and never desludged	17	14
	No O&M training operator	8	7
	No O&M training users	9	1

Figure 56 shows that most plants loaded within the average loading range and built inland produce in sixteen out of nineteen cases effluent concentrations within the range predicted by the design. Their effluent concentrations are mostly around or below 100 mg COD_t l^{-1}.

The four plants loaded above the average design load are two CSCs and two SBS and their effluent concentrations were all within or below design predictions. Surprisingly two of these systems had very low effluent values below 100 mg COD_t l^{-1}. In principle this supports the view that DEWATS could generally be designed smaller while still complying with discharge standards. The number of systems indicating this is however too small to be able to draw strong conclusions. Further investigations are needed in order to confirm this.

5.4. Conclusions

In this chapter treatment indicators of an unprecedentedly large number of DEWATS were compared. Conclusions are however limited by factors typical for research on sanitation in developing countries and are based on intrinsically error-prone single effluent concentration measurements of plants with varying reactor configuration, each exposed to a unique combination of treatment-influencing circumstances. Statistically meaningful conclusions on factors influencing the system efficiency can therefore in most cases not be drawn. The data however enables a number of important observations:

Many systems were under-loaded. This is especially true for CSC and Mixed systems, less so for SSS and SBS. The effect of this on the effluent concentration was difficult to assess due to the surprisingly large spread of concentration data for low loaded plants.

The field-data and field-observations have shown that all DEWATS types including CSC were exposed to storm-water which possibly impeded the treatment processes. Chapter 6 further discusses this issue.

Water treated by DEWATS in coastal areas of Sumatra, Java and Bali tended to have an elevated level of electric conductivity, most probably due to sea water intrusion to over-exploited aquifers. A large proportion of DEWATS with effluent concentrations above design predictions is built in coastal areas suggesting a possible negative impact on the treatment because of elevated salinity or the variation of salinity inside reactor chambers due to the combined effect of salty ground-water and seasonal rain-water influence. Literature reporting good anaerobic treatment of high saline wastewater may not be directly comparable to the here presented situation because of the frequent low organic plant loading.

The dataset did not provide indications that any of the following potentially influencing factors had a statistically meaningful influence on the effluent concentration: location (province), system type, inclusion of BGD in the design, date of implementation, exposure to storm-water, general water scarcity at the site, existence of a CBO and operator, occurrence of desludging of systems older than 3 y, O&M training of the operator and users and use of biogas for systems including a BGD in their design.

It can however not be ruled out that single systems were influenced by these factors, especially since the reason for poor treatment could not be identified for a large number of investigated plants. Each project was exposed to a specific set of circumstances which creates a multi-dimensional space in which the effects of single factors are difficult to isolate.

This obviously also affects the confidence with which conclusions can be drawn on the relation between system loading and effluent concentration. Data on low loaded plants is erratic. However, most plants built inland with loads close to design assumptions appear to produce effluent concentrations within the range of design predictions. Most high loaded plants perform surprisingly well with low effluent concentrations which supports the view that DEWATS are robust towards high loads. Whether this robustness allows future systems to be designed significantly smaller could not be established within the survey presented in this chapter. Future research will need to address this important question by excluding external non-quantifiable influences that the plants discussed in this chapter have been exposed to.

The data indicates guaranteed maximum concentrations of 200 mg COD_t l^{-1} for the effluent of the anaerobic DEWATS treatment if the influence of saline water can be excluded. It is however important to realize that this value was deduced from systems that were hydraulically over-loaded for large parts of the year due to storm-water intrusion. It is hypothesised that their treatment would improve significantly if their maximum hydraulic load was actually close to their respective design-value. The currently observed treatment-efficiencies however imply the need for anaerobic DEWATS effluent to be further treated through a polishing step in order to comply to the comparably stringent effluent regulations of countries such as Vietnam, Cambodia and the Philippines.

5.5. Future research needs

It is advisable to, at least partly, repeat the here presented survey in order to allow more robust conclusions to be drawn. Effluent COD investigations should include fractionated COD measurements, performed as multiple measurements, if possible on different days. EC measurements should be performed on samples taken from a representative number of wells and other water sources used by one community.

A number of future research questions arise from the observations above. They are formulated as hypotheses that should be investigated in future.

- Rain water intrusion has a negative effect on the anaerobic treatment of DEWATS.
- Elevated raw-water salinity affects the treatment of DEWATS.
- Elevated raw-water salinity affects the treatment of low loaded DEWATS more than higher loaded plants.
- High loaded plants perform better than normal loaded plants.

The first hypothesis is further tested in Chapter 6.

6. CASE STUDIES

6.1. Objectives

Field research activities were performed at four DEWATS case study sites from 2009 to 2013 in order to better understand the field conditions under which DEWATS operate and how these conditions affect the performance of the different treatment modules in order to deduce recommendations for future design and operation. The investigations focussed on the treatment module ABR but also considered pre-treatment and AF.

The following four Sections 6.3 to 6.6 each present the field-data and information gathered at one site. Each section covers connected communities, design details, general field observations, load estimations, settled sludge characteristics (TS and VS concentrations, accumulations and methanogenic activities) and presents and analyses system COD removal rates. Each section also contains a short single plant discussion of the respective system data concerning the plant feed characteristics, the observed effects of flow surges on the plant, the estimated reactor loadings and the observed reactor operations.

Section 6.7 compiles the single plant discussion outcomes and further deepens the data analysis.

Section 6.8 extracts and compiles the conclusions drawn from the data analysis depending on reactor modules (pre-treatment, ABR and AF), plant design, plant operation and future research needs at the case study sites.

6.2. General information on case studies

The first DEWATS presented in this chapter is located in Bangalore, India and the following two in Yogyakarta, Indonesia. All four systems treat communal wastewater produced by nearby households connected to the plants by small sewerage systems (SSS). Wastewater sources are kitchens, bathrooms, toilets and laundry washing. All four systems operate in densely populated urban areas in which the population is predominantly Muslim.

Bangalore is the capital city of the Indian state of Karnataka. It has a tropical savannah climate which is comparably moderate due to the high elevation of the city (914 m). The average yearly precipitation is approximately 970 mm. Figure 57 presents typical precipitation and temperature values for Bangalore.

Yogyakarta is situated on the island of Java at 106 m altitude and experiences a tropical monsoon climate. The year is divided into a dry (June to September) and a wet season (October to May) with particularly high precipitation levels from November to April. The regularity of this climatic pattern however has somewhat lessened in the years in which the investigations were performed. Unusually extended wet seasons were observed with rain falling until late July in 2012 and 2013. The average yearly precipitation is approximately 2,200 mm. Figure 58 presents typical precipitation and temperature values for Yogyakarta.

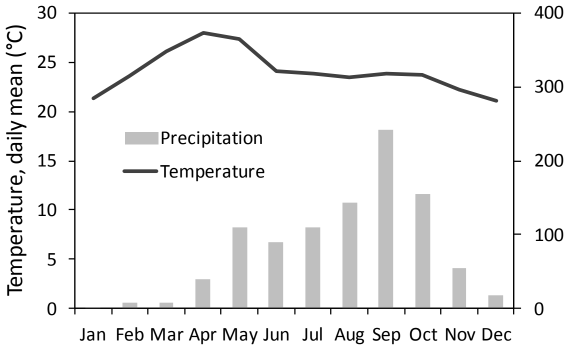

Figure 57: Climatic data Bangalore

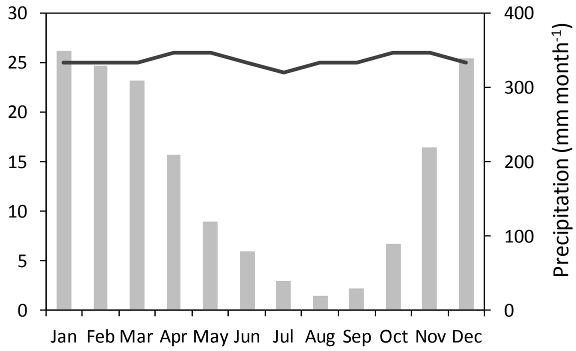

Figure 58: Climatic data Yogyakarta

6.3. Case study A: Beedi Workers Colony (BWC)

6.3.1. The community

The main income source of this community is the production of traditional “Beedi” cigarettes. The households survive on approximately 120 to 200 USD per month (based on estimation of head of CBO in 2012) which classifies them as “low-income” in India.

Before the implementation of the DEWATS project, the wastewater of this community was directly discharged into the nearby storm-water drain. No home industries apart from cigarette production have been reported. The community water supply depends on a bore-well within the colony from which water is pumped to each household for a duration of 10 min each day.

6.3.2. System setup and technical details

Table 25 summarises the setup, technical properties and design values of the plant. The 120 households of the community are connected by a small sewerage system to two parallel biogas-digesters (BGD 1 & 2). BGD 1 additionally treats the kitchen and toilet wastewater of an adjacent office building. The common effluent stream of both digesters is further treated in an ABR with four identical parallel streets each having twelve chambers. The following final treatment step is a planted gravel filter out of which the effluent is finally discharged into a percolation pit.

The plant was first put into operation in 2007 but after operational difficulties was completely restarted in January 2010. Improvements included the rehabilitation of all piping and household connections, the complete desludging of all reactors and seeding of digesters and three ABR streets (streets 1, 2 and 3) (Miller, 2011). All ABR data presented in this chapter was measured in street 4 which was not seeded. Also, the data evaluation focuses on the first five chambers since the currently generally implemented ABR design includes five chambers.

A significant reduction of wastewater production was recorded in late 2011. The main cause for this was found to be the reduced access to water by the community due to the lowering of the ground water table (Pradeep et al., 2012). On April 13th, 2012 the flow to the first ABR street was shut off by the research team which increased the load to the two remaining streets to a level similar as that in 2010. The hydraulic load to street 4 is therefore considered approximately constant over most of the entire investigation period from 2010 to end 2013[12].

An additional sewer line was laid in February 2013 which directed all wastewater which had been previously connected to BGD 1, to BGD 2, therefore by-passing BGD 1 (see Figure 59). This was done for two reasons: firstly to investigate the loading capacity of the biogas digester. Secondly to observe the ABR treatment under higher organic loading rate since it was anticipated that the organic concentration in the digester effluent would increase after load increase.

The available data is therefore divided into two operational phases:

[12] This is based on the assumption of even flow distribution across all streets. This assumption was not verifiable onsite.

- “Phase I” (March 2010 to February 2013) during which two parallel biogas digesters pretreated the raw wastewater. The organic and hydraulic load to the investigated ABR street (street 4) is assumed to have been approximately constant.

and

- “Phase II” (March 2013 to November 2013) during which only one biogas digester pretreated the complete feed flow. The hydraulic load to the ABR was comparable to Phase I. The organic load to the ABR was larger than during Phase I.

No desludging has been performed since the recommissioning of the plant in 2010.

Table 25 contains design information of the system sourced from the design documentation.

Table 25: Plant setup and design properties, picture showing the ABR with the first compartments towards the front of the picture and connected houses in the background

Plant name	Beedi Workers Colony	
Country/ Town		India/ Bangalore
Design	*Plant setup (reactor sizes)*	BGD (2 X 28.55 m^3), 12 ABRs (156.1 m^3), PGF (220 m^2)
	Connected households	120
	Connected users	600
	Per cap ww production	60 l cap^{-1} d^{-1}
	Per cap organic load	30 g COD cap^{-1} d^{-1}
	Daily flow, Q_d	35 m^3 d^{-1}
	*Hydraulic load ABR**	0.2 m^3 m^{-3} d^{-1}
	Average $v_{up,max}$	0.9 m h^{-1}
	*Organic load ABR**	0.37 kg COD m^{-3} d^{-1}
Operation	*Start of operation*	01.02.2010

* only considering five ABR chambers

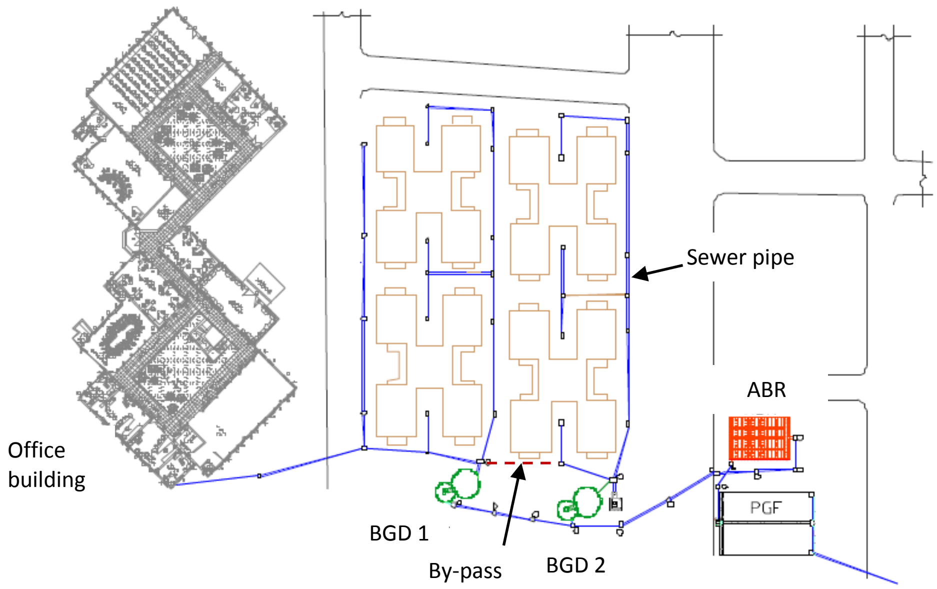

Figure 59: Schematic diagram (top-view) of the DEWATS plant in Beedi Workers Colony/ Bangalore and connected houses with sewer piping, two parallel biogas digesters (BGD 1 & 2), ABR and planted gravel filter (PGF), the dashed line indicates where the sewer line was built in 2013 to by-pass BGD 1 and double the load to BGD 2, Figure adapted from Miller (2011)

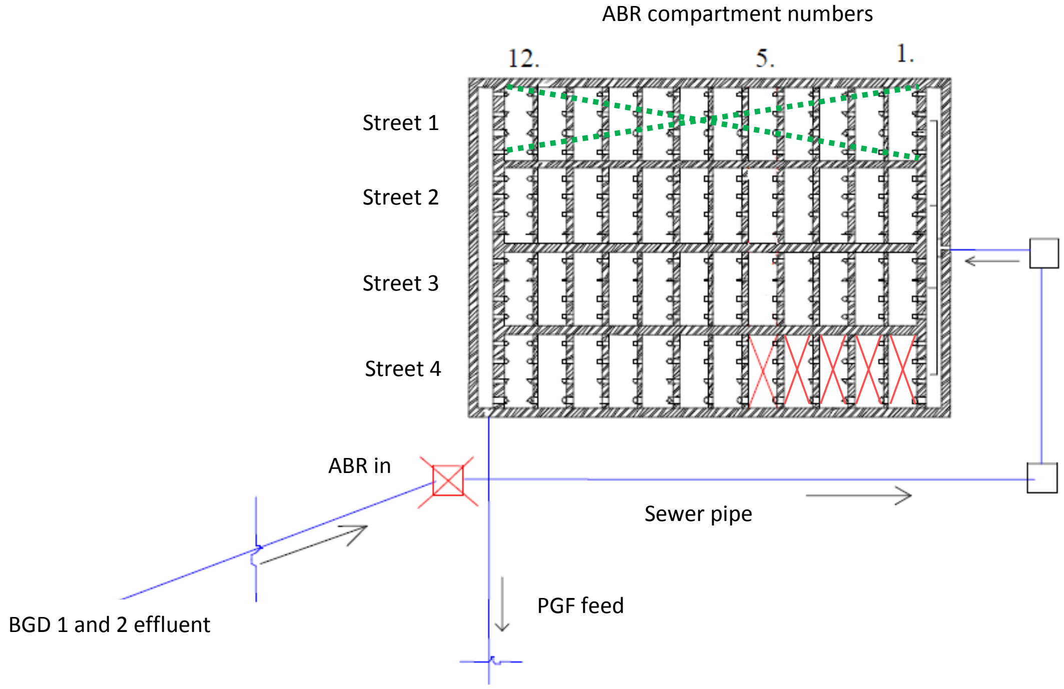

Figure 60: Top view and selection of sampling points (crosses) of the ABR at BWC, sewer pipes and four parallel ABR streets, the dashed line indicates the ABR street that was shut off in 2012 in order to increase the load to the remaining two streets, water depth of system 1,800 mm, Figure adapted from Miller (2011)

6.3.3. Field observations

Figure 61 shows ABR chamber supernatants as photographed on October 13th 2013. Signs of water level fluctuation on pipes and chamber walls are not obvious. There is basically no scum on the water surface. Occasional very small gas bubbles can be observed on the supernatant with no obvious difference between chambers.

Figure 61: ABR chamber supernatants as photographed on 13.10.2013

6.3.4. Monitoring results: load estimation and exposure to flow surges

6.3.4.1. Users

Investigations reported in Miller (2011) yielded a total number of 605 connected persons in 2010. This number rose in 2011 to 654. Piping investigations in 2012 however have shown that approximately 79 users were not connected due to broken piping which is believed to have been the case since mid 2011. The piping breakages were reconfirmed in 2013. The number of persons connected during the period from 2011 to 2013 is therefore estimated to be 575.

6.3.4.2. Flow

The influence of the office building on the DEWATS feed is approximated through the office staff number (62) and by estimating their water consumption at 50 l cap^{-1} d^{-1} with 80% being discharged as wastewater (CPCB, 2009). This results in a daily wastewater production of approximately 2.5 m³ d^{-1}. This value is verified by comparing the DEWATS effluent on working and non-working days:

In 2010 the average wastewater production measured during office working days was 24.9 m³ d^{-1} with a standard deviation of 10% over the measurement days. The flow measured on an office non-working day was about 3 m³ less than during working days (Miller, 2011).

In 2012 the average wastewater production measured during office working days was 19.0 m³ d^{-1} with a standard deviation of 3% over the measurement period. The flow measured on an office non-working day was 15.9 m³ d^{-1}, which represents about 3.1 m³ d^{-1} less than during the week. For details refer to Pradeep et al. (2012).

The estimated office building wastewater production of 2.5 m³ d^{-1} has therefore been shown to be plausible and is used for all further calculations concerning the community's wastewater production.

Figure 62 to Figure 66 show the average diurnal flow patterns as calculated from data recorded in 2010, 2011, 2012 and 2013. All error-bars indicate the standard deviation of the hourly flows over the measurement periods. Data which was obviously influenced by rain was not included.

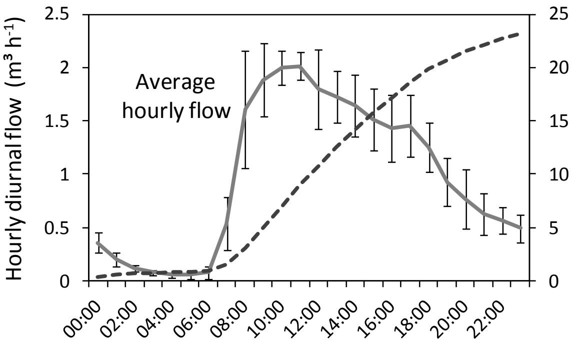

Figure 62: Average flows measured in 2010, averages were calculated with data from 8 d (22.07.2010 to 29.07.2010)

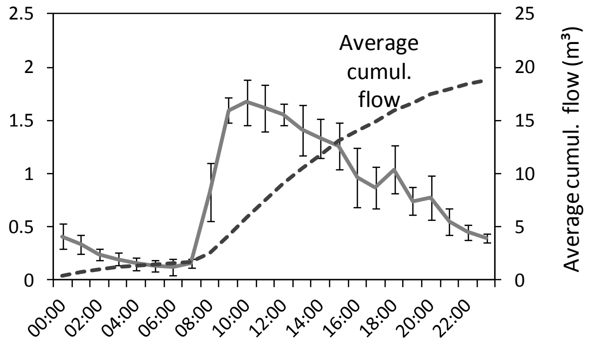

Figure 63: Average flows measured in 2011, averages were calculated with data from 6 d (12.09.2011 to 17.09.2011)

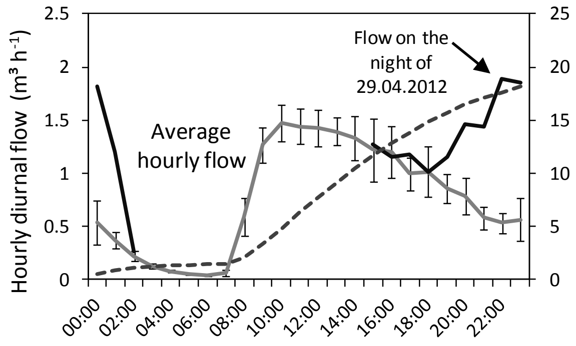

Figure 64: Average flows measured in 2012, averages were calculated with data from 8 d (23.04.2012 to 30.04.2012)

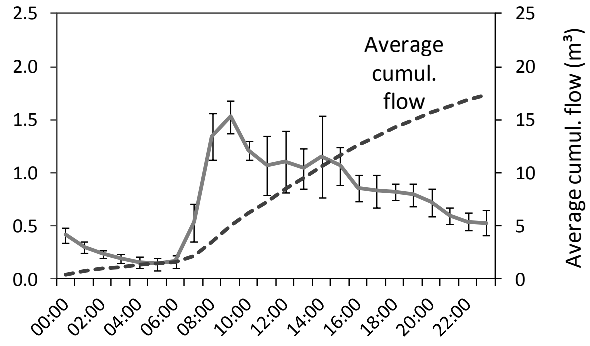

Figure 65: Average flows measured in 2012, averages were calculated with data from 6 d (28.09.2012 to 03.10.2012)

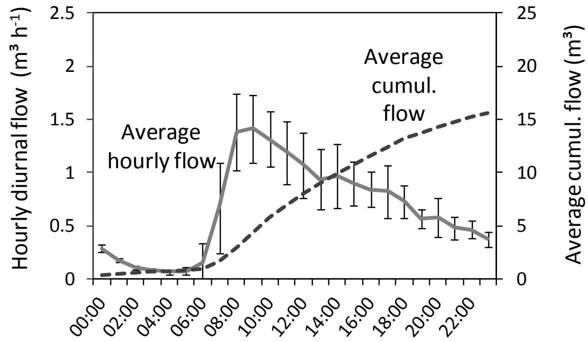

Figure 66: Average flows measured in 2013, averages were calculated with data from 8 d (26.06.2013 to 03.07.2013)

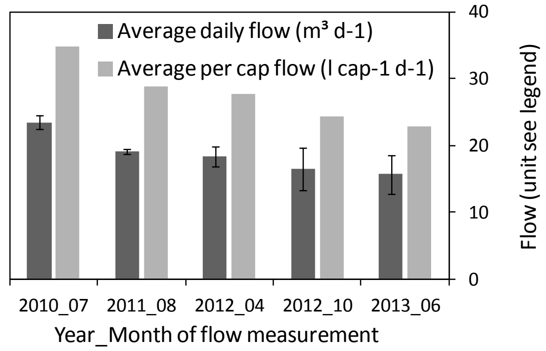

Figure 67: Average daily and per capita flow resulting from measurements taken from 2010 to 2013

The average daily flow in 2010 was 23.5 m³ d^{-1} with a 4% standard deviation across measurement days resulting in an average community per capita wastewater production of 39 l cap^{-1} d^{-1}. Figure 67 summarizes the information given in Figure 62 to Figure 66: the flow to the plant gradually reduced until June 2013 when an average of 15.7 m³ d^{-1} with 19% standard deviation was measured resulting in an average community per capita wastewater production of 27 l cap^{-1} d^{-1}.

Exposure of the system to extreme hydraulic peak loads due to rain water infiltrating the piping system was documented repeatedly. On April 29th, 2012, shortly after a strong rain fall during the early evening, the effluent flow measured at the rear of the ABR peaked with triple the average flow normally measured at this hour of the day (see Figure 64).

6.3.4.3. Summary of system loading results Phase I and II

Table 26 summarizes the available information on load parameters for this plant. The connected number of users was close to 100% design expectation during the complete period of investigation. The average daily feed flow however was always far below design value because of the extremely low per capita wastewater production of the community caused by water scarcity. It decreased steadily from 65% design flow in 2010 to 44% in 2013. The hydraulic load to street 4 was kept approximately constant by closing off street 1 in April 2012.

The system was found to be exposed to sudden strong flow rate increase during the wet season because of storm-water entering the sewer system.

Table 26: Summary of load parameter values, data influenced by storm-water is excluded

Time of measurement	Phase	User number	User number vs design	Average daily flow	Average daily flow vs design	HRT ABR 1 to 5	Per capita ww prod.	Average v_{up} max in ABR*
		User	%	m³ d^{-1}	%	d	l cap^{-1} d^{-1}	m h^{-1}
Design		600	100%	36.0	100%	1.3**	60	0.9
2010_07	I	605	101%	23.5	65%	1.9**	35	0.4**
2011_09	I	575	96%	19.1	53%	2.4**	29	0.3**
2012_04	I	575	96%	18.4	51%	1.8***	28	0.4***
2012_10	I	575	96%	16.6	46%	2.0***	25	0.4***
2013_07	II	575	96%	15.7	44%	2.2***	23	0.4***

* calculated with maximal average hourly flow; ** all 4 streets open; *** 3 streets open, street 1 blocked

6.3.5. Monitoring results: sludge composition, build-up and activity

6.3.5.1. Climatic factor precipitation

Figure 68 illustrates when sludge investigations took place during Phase II and relates them to the precipitation measured during that period. Rain was not measured during Phase II before April 2013 but significant precipitation at that time of the year would be very unusual. On the other hand strong rainfall is probable on the days in August for which no precipitation measurements are available since this is the wettest period of the year. Based on the precipitation data, Phase II was divided into “Phase II - Dry season” (April to June 2013) and “Phase II - Wet season” (July to October 2013).

The available precipitation data for Phase I was too incomplete to be further interpreted.

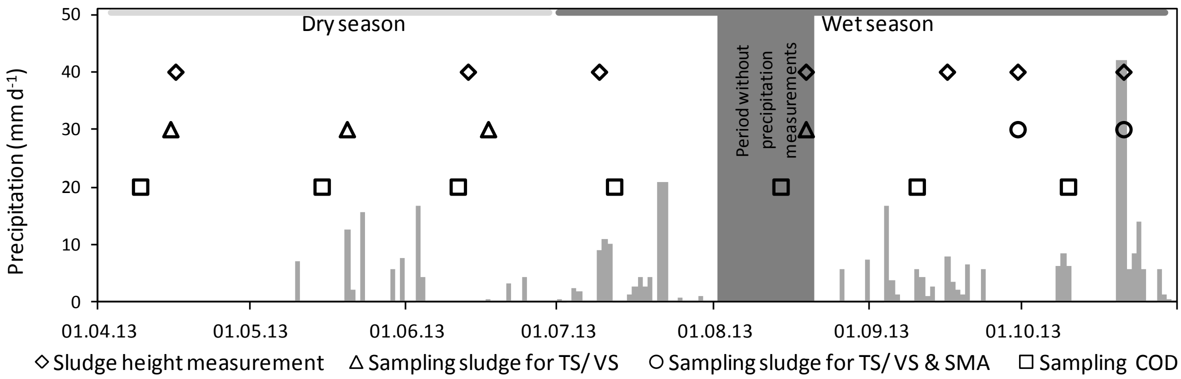

Figure 68: Precipitation data and sludge sampling and height measurement dates in Phase II

6.3.5.2. Sludge heights

Figure 69 a and b show a selection of available sludge height data in each ABR chamber after the start of operation in 2010. (Showing all available data-points would confuse the chart. A compilation of all sludge height data can be accessed as explained in Appendix A6) Most data points represent the average of duplicate measurements. The ABR chambers were never desludged.

During Phase I sludge increased continuously in each chamber over time. The sludge level was always highest in the first chamber and constantly decreased towards the rear.

During Phase II however the sludge heights in the first two chambers rather decreased over time after reaching a certain maximum of 50 cm to 80 cm. The sludge levels in ABR 4, ABR 5, ABR 6 and ABR 7 on the other hand constantly increased until September 2013. In October all levels measured in ABR 5 to ABR 7 decreased dramatically whereas an equally strong increase occurred in the following reactors. In October field staff also noticed washed out sludge in the channel behind the ABR. This strong sludge migration and washout clearly correlates with the high intensity rainfall recorded in late October (see Figure 68).

During Phase II the highest sludge levels were found in the middle chambers.

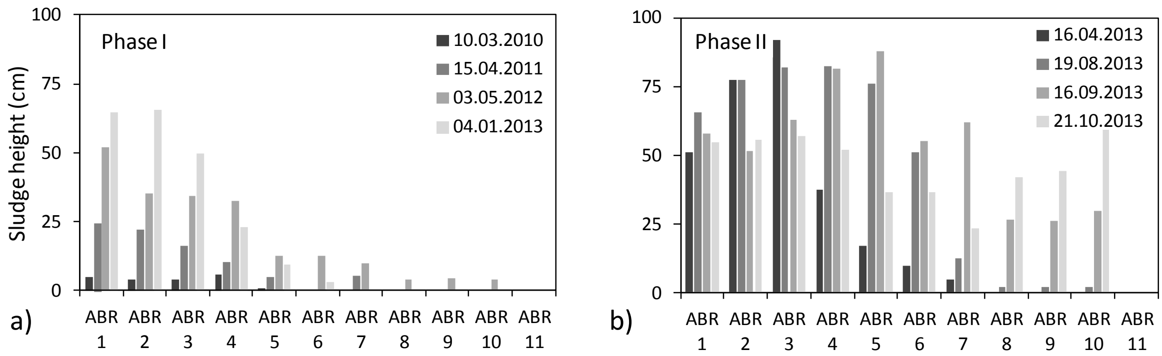

Figure 69 a and b: Selection of settled sludge levels measured in Beedi Workers Colony, Phase I and II

6.3.5.3. Sludge volume increase

Figure 70 shows the increase of total ABR sludge volume over the time of operation. Linear regression of the Phase I values indicated an approximate sludge increase of 2 l d^{-1} or 0.7 m^3 y^{-1} in the chambers ABR 1 to ABR 5. The amount of sludge washed into the rear chambers over that period was minimal.

The sludge build-up in the chambers ABR 1 to ABR 5 during "Phase II – Dry season" (based on the measurements taken from 04.01.2013 to 17.06.2013) was estimated at 2.1 m^3 y^{-1} with about 0.4 m^3 y^{-1} accumulating in the chambers downstream.

During "Phase II – Wet season" large amounts of sludge started to migrate from the first 5 chambers to the chambers beyond. The sludge build-up in all 12 ABR chambers in this period (based on the measurements carried out from 17.06.2013 to 16.09.2013) was approximately 11.7 m^3 y^{-1}. The last sludge height investigation on October 21st, directly after the strong rainfall presented in Figure 68 indicated further strong washout from the first 5 chambers and a general sludge volume reduction in the complete ABR.

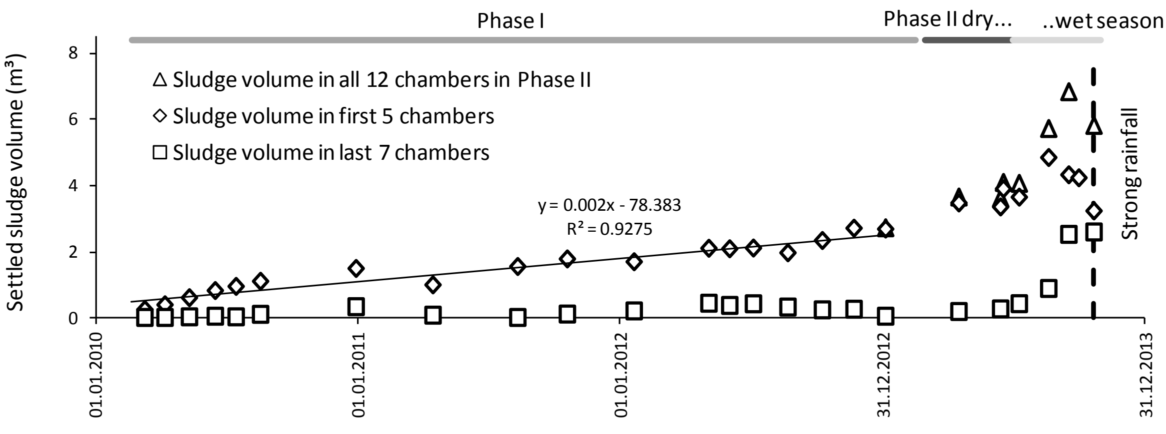

Figure 70: Sludge volume evolution in Beedi Workers Colony ABR chambers

6.3.5.4. Sludge Total and Volatile Solids concentrations

No sludge TS and VS concentration data was available for Phase I. The composition of settled sludge was measured for reactor chambers ABR 1 to ABR 6 in 2013 as part of the SMA investigations during Phase II. The results are shown in Figure 71 a and b. A tendency can be observed of the TS and VS concentration being highest in the first two ABR chambers and constant or lower in the following reactors. The across ABR chamber average TS and VS concentrations of settled sludge were approximately 50 g l^{-1} and 26 g l^{-1} respectively with a respective standard deviation of 30% and 29%. The sludge concentrations measured directly after the strong rain at the end of October 2013 suggest a shift: most dense sludge was no longer found in the first two but rather in the rear chambers.

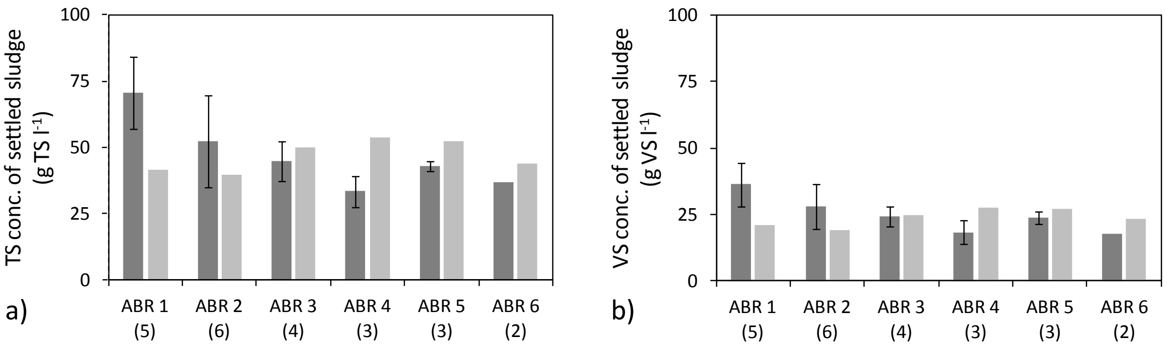

Figure 71 a and b: Settled sludge TS and VS average concentration profiles, number of measurements in brackets, error-bars indicate standard deviations of multiple measurements

6.3.5.5. Specific methanogenic activity (SMA) of sludge

Two SMA measurement runs were conducted for this study. Both sludge sets were sampled during the "Phase II – Wet season" period, one before and one right after the strong rainfall recorded on October 20th (see Figure 68).

The SMA results with sludge sampled on September 30th indicated an uneven distribution between the chambers with highest SMA_{max} values measured in ABR 2 and 4 of approximately 0.1 g COD g VS^{-1} d^{-1}. The second run yielded a SMA_{max} value of approximately 0.15 g COD g VS^{-1} d^{-1} in the first and steady decrease in the consecutive chambers.

The results suggest a strong increase in chamber 1 and 3 and a decrease in chamber 4 and 6. The SMA_{max} values were comparably similar in the chambers 2 and 5. In general the highest acetoclastic methanogenic activity was observed in the first chambers.

The observed high variability of measurement results is difficult to interpret since the SMA measurement was a comparably new method for the laboratory team in Bangalore and these were the first successful measurement runs produced by that team. The effect of storm water intrusion on the SMA_{max} value can therefore not be ascertained but further measurement runs are needed in order to approximate the variation of measurement results under undisturbed operational conditions.[13] It is however conceivable that the strong rainfall recorded on October 20th washed active sludge from the digester into the first ABR chambers, therefore increasing the SMA.

[13] Sequential SMA_{max} investigations of the same sludge performed in the Yogyakarta laboratory had shown a maximal variation of 12% over 2 months of operation (see Section 3.4.6)

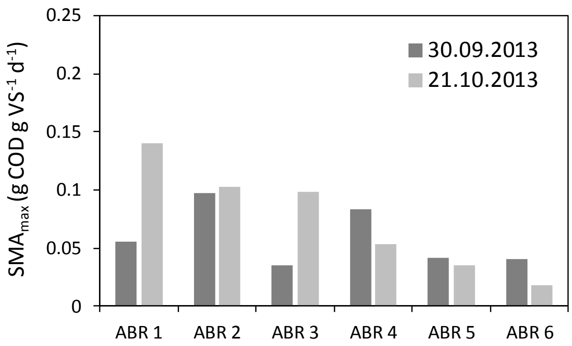

Figure 72: SMA_{max} values of sludge sampled from different reactors in 2013 at BWC, all sludges were processed within one week after sampling, all values were derived from single measurements

6.3.6. Monitoring results: alkalinity, pH, temperature and nutrient concentrations

The alkalinity of the well water used by the households in the community was measured six times in 2010 yielding an average alkalinity of 468 mg $CaCO_3$ l^{-1} with a standard deviation of 59 mg $CaCO_3$ l^{-1}. It was assumed to stay constant over the entire period of investigation.

Figure 73 to Figure 76 present the operational parameters alkalinity, pH, turbidity and ammonium concentration as measured in the ABR feed and the supernatants of the different ABR chambers.

The water alkalinity more than doubles over the pre-treatment after which it reaches approximately 1100 mg $CaCO_3$ l^{-1} in Phase I.

An independent samples t-test was conducted to compare the alkalinity values measured at the ABR feed during Phase I and II (excluding data measured during the wet season). There was a significant difference in the values measured in Phase I (M = 1144 mg $CaCO_3$ l^{-1}, SD = 193 mg $CaCO_3$ l^{-1}) and Phase II (M = 1414 mg $CaCO_3$ l^{-1}, SD = 189 mg $CaCO_3$ l^{-1}); $t(15) = -2.45$, $P = 0.027$. It is therefore statistically supported that the alkalinity at the ABR feed significantly increased from Phase I to II.

The pH was stable across reactor chambers with a slight increase of the median value in ABR 1 (Figure 74). All median values slightly rose in Phase II which concurs with the increase in alkalinity discussed above.

The wastewater temperature was always between 23°C and 31°C with some variation across seasons (further details in next section).

Measurements show an obvious reduction of turbidity in the first ABR chamber in both phases. An independent samples t-test was conducted to compare the turbidity values measured at the ABR 1 and ABR 5. There was a significant difference in the values measured at the ABR 1 (M = 91 NTU, SD = 21 NTU) and ABR 5 (M = 60 NTU, SD = 21 NTU); $t(54) = 5.57$, $P = 1.2 \times 10^{-6}$.

This statistically supports that turbidity was significantly reduced throughout the first five ABR chambers. The difference in reduction between both phases will further be discussed in the next section.

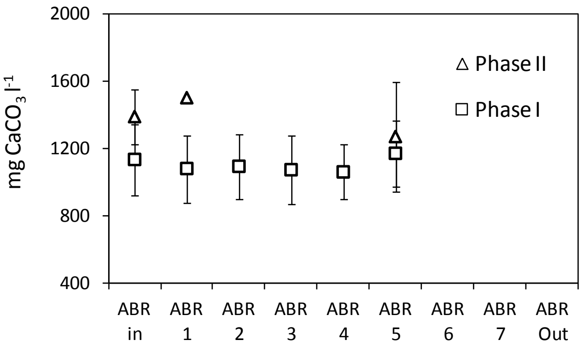

Figure 73: Average alkalinity concentration profile across reactor chambers as measured in Phase I and II, error-bars indicate standard deviations, 6 to 36 data points per sampling point

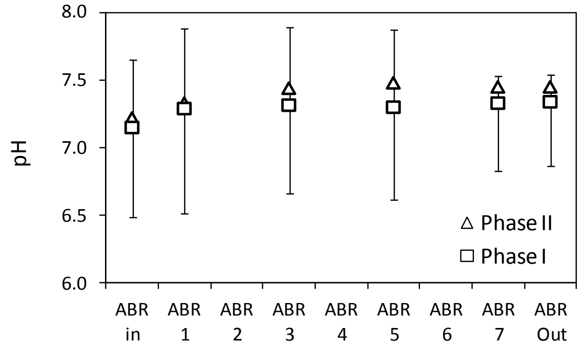

Figure 74: Median pH profile across reactor chambers as measured in Phase I and II, error-bars indicate maximum and minimum measured values in Phase I, 4 to 36 data points per sampling point

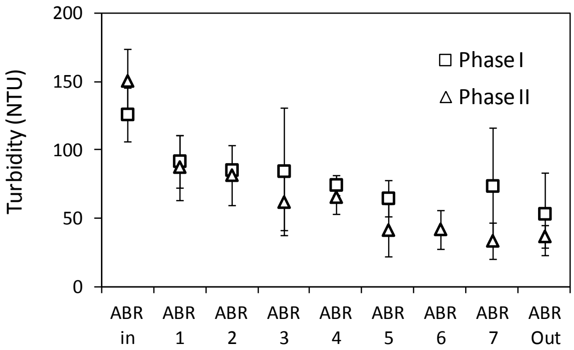

Figure 75: Average wastewater turbidity profile across reactor chambers as measured in Phase I (not 2010) and II, error-bars indicate standard deviations, 4 to 28 data points per sampling point

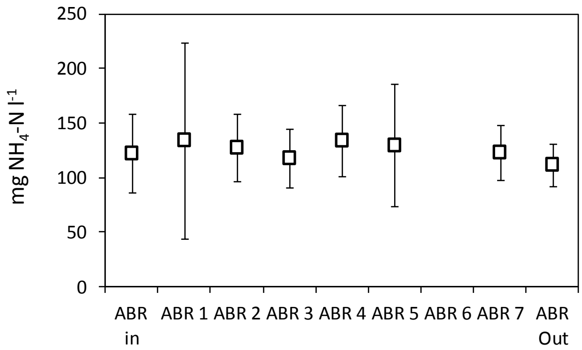

Figure 76: Average wastewater NH4-N concentration profile across reactor chambers as measured in Phase I, error-bars indicate standard deviations, 4 to 23 data points per sampling point

Nutrient investigations indicate 122 mg NH_4-N l^{-1} (n= 10) and 16 mg PO_4-P l^{-1} (n= 11) in the ABR feed and 130 mg NH_4-N l^{-1} (n = 23) and 18 mg PO_4-P l^{-1} (n = 18) in the effluent. All four values are averages from four sampling campaigns, each with duplicate measurements.

6.3.7. Monitoring results: reactor COD concentrations and COD removal rates

Figure 77 presents COD_p data as measured at the ABR feed and in ABR 5. The data shows no clear correlation with the seasonal factors included in the figure. COD_p is difficult to measure accurately and intrinsically prone to large methodological error since it requires a difficult filtering step. The dataset presented in Figure 77 for instance included 3 incorrect, since negative, values which had to be removed for analysis. Two extremely high COD_p outliers were also excluded based on comparison with turbidity measurements.

COD_p data was generally very variable throughout the whole investigation period, especially in the years 2010, 2011 and 2012.

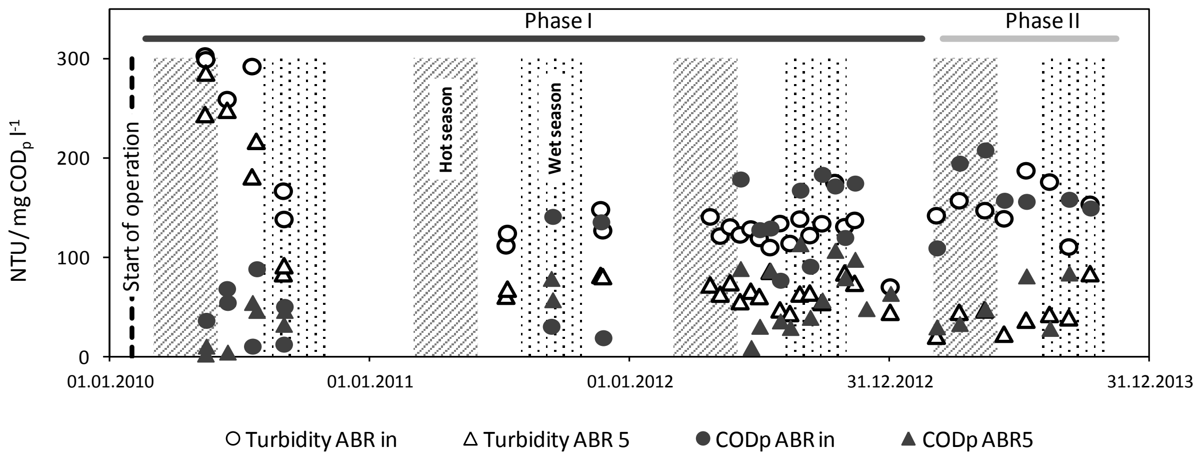

Figure 77: ABR_{in} and ABR 5 turbidity and COD_p concentrations, the light red areas indicate the warmest period of the year, the light blue areas indicate the wettest period of the year

Since turbidity is known to be a robust indicator for particulate wastewater content it was used to check the plausibility of the available COD_p data. As can be seen on Figure 77, the turbidity measurements confirm that the particulate content of the wastewater did not correlate with the seasons.

As opposed to the COD_p data, turbidity was very high in the first half year of operation which appears plausible since the digesters had just been started up. The linear reduction of the turbidity data supports its credibility since this is exactly what would have been expected to happen during reactor start-up. The COD_p data on the other hand was unexpectedly low until the end of 2010. It also showed a very high variability in 2010 and 2011 which is not consistent with the comparably constant turbidity values.

This raises questions concerning the accuracy of the COD values measured in the first two years. It was therefore decided not to include them for the further analyses of the dataset, also since the operational conditions in 2010 were obviously not comparable to the following years.

The remaining COD_p dataset was then subjected to statistical investigations in order to assess whether there were significant differences between the reduction rates of the two phases. Data was normally distributed. Paired-sample t-tests failed to reject the null hypotheses that COD_p concentrations were similar for ABR_{in} and ABR 5 across phases (see Table 27 for details). Significant increase of COD_p reduction from one phase to the next therefore appears statistically improbable.

The t-tests were however repeated with the turbidity data after asserting their normal distribution and, as opposed to the COD_p values, showed a significant increase of the ABR_{in} values and decrease of the ABR 5 values across the phases (see Table 27). This implies that also the particulate reduction from ABR_{in} to ABR 5 increased from Phase I to Phase II.

Table 27: Details of t-tests investigating the difference between COD_p and NTU values across phases

	SP	Unit	M	SD	df	t	P	Significant difference
Phase I	ABR_{in}	COD_p	142	39	14	2.1	0.3	no
Phase II	ABR_{in}	COD_p	162	35				
Phase I	ABR 5	COD_p	63	33	19	2.1	0.2	no
Phase II	ABR 5	COD_p	43	30				
Phase I	ABR_{in}	NTU	126	21	22	2.1	0.02	yes
Phase II	ABR_{in}	NTU	151	23				
Phase I	ABR 5	NTU	63	13	21	2.1	0.005	yes
Phase II	ABR 5	NTU	41	19				

Figure 78 presents ABR_{in} and effluent COD_s concentration and wastewater temperature. As can be seen, seasonal wastewater temperature fluctuation and wet seasons most probably strongly influenced the COD_s digester effluent concentration. Rain infiltrating the piping system could have lead to dilution and therefore reduction of measured wastewater concentrations. The high COD_s values during and after the warmest season of the year could be explained by an increase of particulate organics solubilisation in the digester accompanied by rising of general SMA.

The average ABR_{in} COD_s concentrations were 368 mg COD_s l^{-1} and 434 mg COD_s l^{-1} for Phase I and Phase II respectively (see Figure 79). Figure 78 however shows that the apparent increase of average feed COD_s was caused by a larger fraction of measurements taken during the warm and dry season in the Phase II dataset (57%) than in the Phase I dataset (33%). It is therefore not possible to compare the treatment of both phases only based on the available COD_s (and therefore COD_t) data.

The COD_s concentrations measured in the supernatant of ABR 5 on the other hand were significantly lower during the warm dry season of Phase II than of Phase I. This implies a significantly higher COD_s reduction in Phase II. Influence of rainwater can be excluded in this case since the relevant data points only lay within the dry season.

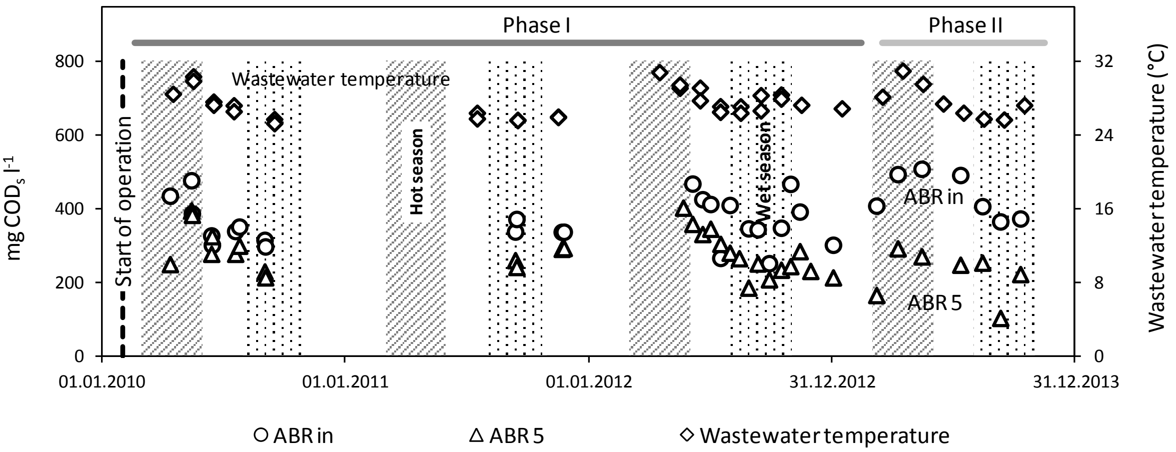

Figure 78: ABR_{in} and ABR 5 COD_s concentrations and measured wastewater temperature, the light red areas indicate the warmest period of the year, the light blue areas indicate the wettest period of the year

Figure 79 a and b present the average COD concentration values measured in the different reactor chambers in Phase I and II. The represented values were computed only with measurement results from 2012 onwards and exclude certain outliers for reasons explained above. The COD values measured on June 11th, 2013 were not considered since these were extremely high, leading to non-normality of the complete COD_p and parts of the COD_t and COD_s dataset. Plausibility checks with turbidity measurements were made which supported the decision to remove these values. No rain occurred on the day of or prior to the sampling. The reason why these values differed from the rest of the dataset could not be identified.

In order to take the above shown effect of seasonal variations on data into consideration, the following average reactor reduction rates were calculated as the average of all differences between corresponding ABR_{in} and ABR 5 values measured on the same day:

In Phase I the average reduction from ABR_{in} to ABR 5 implied by the available data is 35%, 49% and 26% for COD_t, COD_p and COD_s respectively.

The calculated average reduction in Phase II is 58%, 73% and 50% for COD_t, COD_p and COD_s respectively.

The reactor reduction rates of all COD fractions within the same phase are obviously significant (see Figure 79 b so that further statistical testing was not deemed necessary in this case.

As to the significance of the treatment increase across phases, for reasons mentioned above, comparing the treatment efficiencies simply based on the available COD data could lead to wrong conclusions.

Available turbidity data for instance is believed to be less prone to analytical error than the COD_p data. It is therefore assumed to depict reality better in terms of particulate content. Thus, based on turbidity measurements a statistically significant increase of particulate, and therefore COD_p, reduction across the phases is accepted as being the most credible scenario although COD_p data itself indicates the opposite.

COD_s concentrations on the other hand cannot be directly compared across phases because seasonal factors differently affected both datasets. Nevertheless a significant increase in reduction from Phase I to II is implied by the curve progression of the available data.

It is therefore concluded that COD reduction indeed increased significantly from Phase I to II. A quantification with the available data was however not possible.

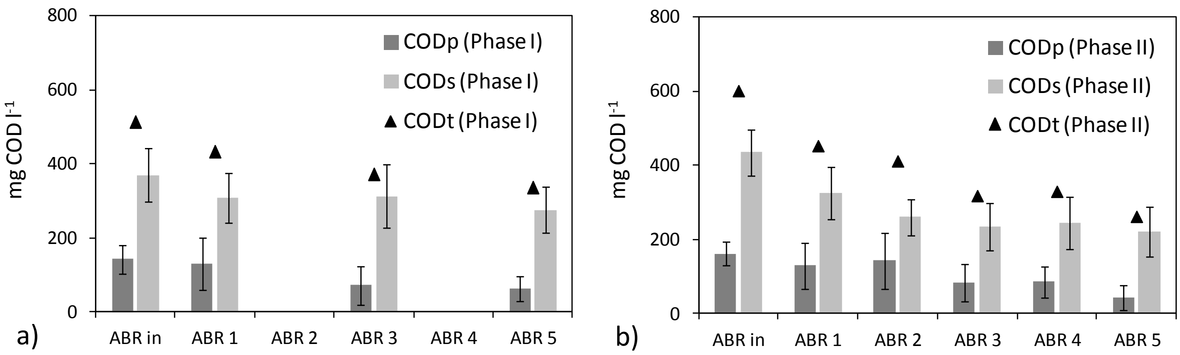

Figure 79 a and b: COD fraction concentration profiles as measured in reactor chambers, error-bars indicate standard deviations

Table 28 summarizes the outcomes of paired sample t-tests investigating the statistical significance of COD reductions measured across chambers during Phase I.

The COD_t reductions were statistically significant across all chambers. The only significant COD_p reduction however was measured between ABR 1 and 3, whereas the COD_s concentration significantly decreased in ABR 1 and between ABR 3 and 5.

Table 28: Details of t-tests investigating the statistical significance of COD reductions measured across ABR chambers, Phase I

	ABR in & 1		ABR 1 & 3		ABR 3 & 5	
	P	Significance	P	Significance	P	Significance
COD_t	0.001	yes	0.003	yes	0.02	yes
COD_p	0.5	no	0.01	yes	0.5	no
COD_s	0.0002	yes	0.6	no	0.03	yes

Table 29 summarizes the outcomes of paired sample t-tests investigating the statistical significance of COD reductions measured across chambers during Phase II.

Significant COD_t reduction occurred until the rear chamber although not throughout all chambers. Significant COD_p reduction was only measured between ABR 2 and 3. ABR_{in} COD_p data could not be used in this test since it was not normally distributed. COD_s concentrations only significantly declined in the first two ABR compartments.

Table 29: Details of t-tests investigating the statistical significance of COD reductions measured across ABR chambers, Phase II

	ABR in & 1		ABR 1 & 2		ABR 2 & 3		ABR 3 & 4		ABR 4 & 5	
	P	Significance	P	Significance	P	Significance	P	Significance	P	Significance
COD_t	0.02	yes	0.09	no	0.005	yes	0.8	no	0.045	yes
COD_p			0.9	no	0.02	yes	0.8	no	0.09	no
COD_s	$2*10^{-6}$	yes	0.01	yes	0.06	no	0.9	no	0.1	no

No effluent BOD_5 concentration was measured on this site.

Measured effluent concentrations were 336 (± 59) mg COD_t l^{-1} and 262 (± 64) mg COD_t l^{-1} in Phase I and II respectively. The effluent contained about 100 mg COD_s l^{-1} of non-biodegradable COD_s. This result is based on two investigations performed during Phase II, both of which were done with quadruple measurements. Nonbiodegradable wastewater fractions strongly depend on user habits. Since the population did not change significantly over the investigation period, the available value for Phase II is assumed to also be representative for Phase I. Consequently a large fraction of the COD leaving the reactor was still biodegradable.

6.3.8. Discussion of case study data

6.3.8.1. Plant feed characteristics

A constant reduction of community wastewater production over the entire period of investigation was observed through flow measurement campaigns. Wastewater flow to the plant was approximately 65% design flow at the start of operation in 2010 and 44% in 2013.

Apart from a slight decrease of connected people between 2010 and 2011 constant numbers of connected users can be assumed which are close to design estimations. It is therefore concluded that the organic load to the plant remained approximately constant over the entire period of investigation.

Well water measurements yielded a mean raw-water alkalinity of 468 mg $CaCO_3$ l^{-1} with a standard deviation of 59 mg $CaCO_3$ l^{-1}.

6.3.8.2. Effect of flow surges on plant reactors

It is evident that at irregular intervals storm-water entered the sewer system increasing the hydraulic load to the plant. A significant flow increase was measured during such rainfall in 2012.

The ABR sludge appeared to be remarkably unaffected by these flow surges during Phase I: the sludge-levels in all chambers accumulated regularly, apparently without being influenced by strong sludge migration within the compartments.

The matter was entirely different during Phase II, where sludge accumulation soared during the wet season, most probably due to washed out sludge from the digester. There was also a strong shift of sludge towards the rear compartments (which was supported by TS and VS sludge concentration measurements) and obvious washout from the reactor. Whether SMA, as shown by the available data, truly increased over the wet season needs to be confirmed by future investigations. A possible reason for the observed activity increase could have been washout of active sludge from the digester to the ABR.

Field observations did not indicate obvious signs of strong water level fluctuations inside the ABR.

6.3.8.3. Estimated digester load and treatment

Figure 80 a to d present the loading and treatment parameters OLR, HRT, biogas production and effluent COD concentrations for BGD 2. They compare design estimations and the results based on the investigations carried out in Phase I and II.

The OLR is in this case based on the number of connected users (shown as cap m^{-1} d^{-1}) since representative feed concentration data was not available. The load doubled in Phase II compared to Phase I and design. In terms of hydraulic load however, the adjustments made during Phase II established the load situation for which the digester had been initially designed.

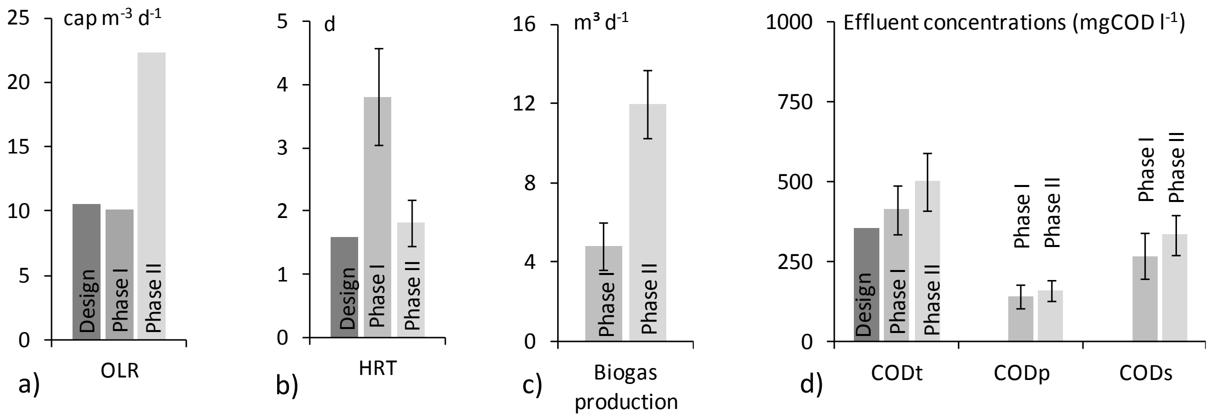

Figure 80 a, b, c and d: Loading and treatment parameters of BGD 2 in Phase I and II: OLR, HRT, biogas production and digester effluent concentrations, error-bars indicate standard deviations

Increasing the load certainly had a strong impact on the treatment of the digester: the digester effluent alkalinity increased significantly indicating stronger anaerobic activity. The biogas production more than doubled from 4.8 m³ d^{-1} to 12 m³ d^{-1}.

The slight digester effluent COD_p concentration increase in Phase II was not found to be statistically significant. The data however certainly underestimates the real particulate washout in Phase II: observations made on ABR sludge accumulation rates in Phase II before the wet season indicated an increase of more than 300% compared to Phase I. The digester also became much more sensitive to hydraulic surges in the wet season when more than fifteen times more sludge accumulated in the ABR than in Phase I. Possibly the digester had incidentally reached its maximum sludge capacity by the time the load change occurred. However, no noticeable sludge washout was recorded in the previous wet season. Also, communal biogas digesters operating under tropical climate are generally known to have excellent sludge stabilisation abilities. They certainly require desludging after much longer periods than the two years BGD 2 had been operating when the loading change occurred.

A mass balance calculation across the biogas digester was attempted with the available data using the methodology described in Section 3.8.1. The average flow measured in 2013 of 15.7 m³ d^{-1} was used as the value for Q. Repeated biogas composition measurements by an external laboratory and the BORDA research team yielded approximately 80% CH_4 content. 12 m³ d^{-1} of biogas production indicate a daily COD reduction of about 22 kg COD d^{-1}. Added to the amount of COD leaving the reactor this implies an approximate average digester feed concentration of 1.900 mg COD l^{-1} and a per capita COD production of about 52 g COD cap^{-1} d^{-1}. Both these values however surely underestimate the reality since a certain fraction of the produced methane certainly escaped the reactor, dissolved in the effluent wastewater. The design per capita production of 30 g COD cap^{-1} d^{-1} therefore certainly underestimated the real value.

The above implies an average digester treatment efficiency of 73%. Again, this value most probably does not take all produced CH_4 into consideration and therefore underestimates the real treatment efficiency.

The COD_s increase presented in Figure 80d has been shown to have been mainly induced by seasonal variations. The COD_s concentration was therefore not significantly different across the phases.

The design effluent concentration was much lower than what was measured during both phases.

It is interesting to note that significantly more biogas was produced during April, the hottest month of the year. Also, turbidity measurements indicated a start-up period which lasted approximately six months for the digester effluent particle content to reach a constant value.

6.3.8.4. Estimated ABR load and treatment

6.3.8.4.1 Reactor load and performance

A steady decline of wastewater feed flow was observed over the complete investigation period, apparently due to water scarcity.

Figure 81 a to d present the loading and treatment parameters OLR, HRT, effluent COD concentrations and average reduction rates.

The Q used for the OLR and HRT measurement calculations was the average of the flow measured in 2010 and 2011 divided by four streets and the flow measured in August divided by three streets. A variation of 20% was estimated.

In the absence of better data, the OLRs were calculated with the available COD concentration values although these were shown to not be necessarily comparable. Additional information needs to be considered in order to correctly interpret the graph.

The OLRs in both phases are comparable to the design assumption. The generally higher OLR in Phase II shown in Figure 81 a stems from an apparent COD_s increase in the Phase II feed flow, which was however shown to be insignificant. Phase II COD_p ABR_{in} data however certainly strongly underestimates the real average value since digester sludge was repeatedly washed into the ABR during storm water events in 2013 which the measured COD_p values do not account for. The actual OLR in Phase II was therefore probably much higher than design expectations.

The HRT indicates an approximately 50% design load under dry weather conditions in both phases. The average $v_{up,max}$ was 0.4 m h^{-1} without rain water influence and therefore below the design value of 0.9 m h^{-1}.

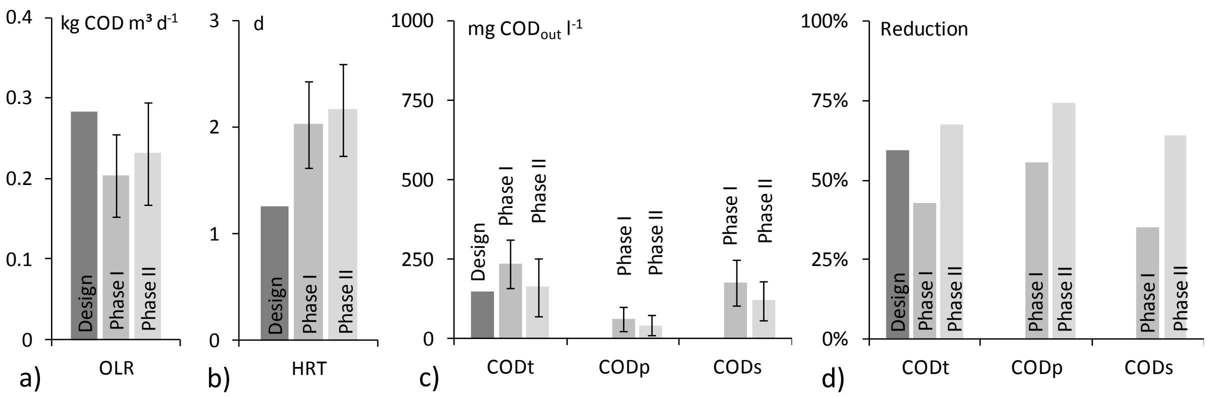

Figure 81 a, b, c and d: Loading and treatment parameters of the first five ABR chambers in Phase I and II: OLR, HRT, effluent COD concentrations and COD reduction rates, OLR error-bars indicate combination of standard error of mean of COD_t measurements and standard deviation of Q, all other error-bars indicate standard deviations

Alkalinity and pH investigations indicated stable anaerobic treatment conditions throughout the reactors and operational phases.

The COD_t treatment efficiency however was extremely low in Phase I with 35%. The comparably small COD_s and COD_p concentration reductions were found nevertheless to have been statistically significant.

A strong increase in treatment efficiency appears to have occurred from Phase I to Phase II during dry weather conditions although more measurements are needed to quantify this.

The available COD_p data did not enable meaningful statistical testing because of its associated large measurement error. A significant increase in particulate reduction is however implied by the more reliable, since less error prone, turbidity measurements. But it is important to note that this only applies to dry-weather conditions since that data does not take rainfall into consideration. Rainfall however is known to have caused considerable sludge washout from the digester into the ABR. Also, the observed increase in turbidity reduction in Phase II correlates with a marked increase of sludge bed height which might have enhanced the filter effect of the reactor.

The COD_s concentration values cannot be directly compared across phases for reasons explained above. The general trend of the data however strongly suggests improved treatment in Phase II.

A raise of COD_s reduction implies an increase of sludge bio activity in Phase II. Two hypotheses are proposed to explain this phenomenon. Firstly, increase of ABR sludge activity was linked to the wash out of active sludge from the digester into the ABR in Phase II. Secondly, increase of ABR sludge activity was due to increased organic load in Phase II.

6.3.8.4.2 COD_p massbalance

The COD_p mass balance was calculated as detailed under Section 3.8.2:

The averages of measured values for Q, COD_p concentrations of ABR_{in} and ABR 5 and VS sludge concentration led to 4.2 (min = 1.9, max = 8.9) $m^3\ y^{-1}$ sludge increase in Phase I. It is assumed that the measured sludge VS concentration is representative for Phase I although it was measured during Phase II. Minimum and maximum values take into account a feed flow variation of 20%, the standard error of means of COD_p concentrations and the standard deviation of sludge VS concentration.

Linear regression of sludge volumes measured in the first five ABR chambers led to a sludge build-up rate of 0.7 $m^3\ y^{-1}$. This is below the minimal rate calculated through mass balance. This discrepancy could be explained through unnoticed sludge washout on days on which no wastewater sampling took place. This however is improbable since no or very little sludge was found inside the chambers beyond ABR 5. The result therefore supports the hypothesis that anaerobic digestion did take place inside the ABR and significantly reduced the volume of retained biodegradable COD_p. The further testing of this hypothesis with anaerobic digestion modelling is described in Chapter 7.

Comparing sludge build-up in Phase II to mass balance results was not attempted: it is obvious that strong sludge washout from the digester into the ABR and migration out of the first five ABR compartments occurred repeatedly during the period after mid June 2013 until the end of Phase II. Since the Q and COD_p measurements do not represent these washout events they cannot be compared to sludge build-up. Attempting a comparison with the dataset gathered during Phase II before being affected by rain is unpromising since its size is very small (n = 4 for COD and sludge volume values).

6.3.8.4.3 Compartment performance ABR

Figure 82 a and b compare the average COD_t measurement data of Phase I and II with predictions given by the ABR design calculation. The inputs for these calculations are the measured average flows (as described in Section 6.3.8.4.1) and average feed concentrations. The hydraulic load was very similar in both cases. The organic feed content during dry weather however was, as discussed above, similar in relation to soluble components but had a larger particulate fraction in Phase II. The organic feed load certainly increased strongly due to rainfall since large volumes of digester sludge were washed into the ABR. The "Initial design" curve represents the treatment assumed at the design stage of the plant with a significantly lower feed concentration.

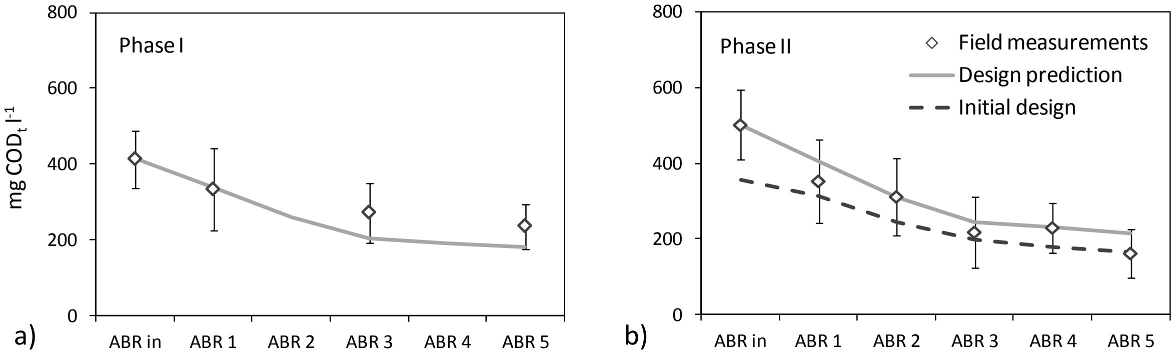

Figure 82 a and b: Measured average COD_t concentration profiles in Phase I and II, initial design predictions („Initial design") and design predictions with input variables adjusted to measured field values („Design prediction")

The COD_t reductions across single chambers (see Figure 82 a) were all shown to be statistically significant. They are however below design expectation for all chambers following ABR 1.

The COD_t reductions across single chambers as presented in Figure 82 b were shown to be statistically significant for ABR 1, ABR 3 and ABR 5, with COD_s reduction occurring primarily in the first compartments and COD_p reduction throughout all the compartments. The concentration ranges are within design predictions.

Whether the increased treatment was due mainly to increased organic load or primarily to the accumulation of active digester sludge cannot be judged at this point but a combination of both factors is probable.

Most wastewater treatment clearly occurs in the first three chambers and more so in Phase II. This correlates with SMA measurements which show higher bioactivity in the front chambers.

6.4. Case study B: Gambiran (GB)

6.4.1. The community

The people connected in Gambiran (GB) are members a low to middle income community in which most of the households live on 50 to 100 USD a month (based on estimation of head of CBO in 2011). Before the implementation of the DEWATS project, the wastewater from this community was directly discharged into the nearby river or disposed of in individual soak-pits. No home industries have been reported. The households have unrestricted access to fresh water either through private wells or municipal connections.

6.4.2. System setup and technical details

Table 30 summarises the setup, technical properties and design values of the plant. A total of fifty-five households are connected by a small sewerage system which discharges the black-water of forty households into the first treatment step, a BGD. As shown in Figure 83, the following step is a settler which is fed by the effluent of the BGD and the remaining black-water and grey-water of the connected households. The wastewater is further treated by an ABR with four compartments followed by an AF with two compartments. The desludging shafts of all AFs are lower than the water level due to a design error. In principle this would lead to an efficiency reduction since the wastewater would predominantly follow the way of least hydraulic resistance, through the empty shaft instead of the fixed bed. At beginning of operation the BGD was seeded, not so the ABR. ABR chambers were never desludged.

Table 30: Plant setup, design properties and picture of the site (manhole to BGD in front)

Plant name		**Gambiran**
Country/ Town		Indonesia/ Yogyakarta
Design	*Plant setup (reactor sizes)*	BGD (10.3 m³), Settler (9.6 m³), 4 ABRs (19.2 m³), 3 AFs (23.4 m³)
	Connected households	55
	Connected users	200
	Per cap ww production	100 l cap^{-1} d^{-1}
	Per cap organic load	114 g COD cap^{-1} d^{-1}
	Daily flow, Q_d	20 m^3 d^{-1}
	Hydraulic load ABR	1 m^3 m^{-3} d^{-1}
	Average $v_{up,max}$	0.8 m h^{-1}
	Organic load ABR	0.7 kg COD m^{-3} d^{-1}
Operation	*Start of operation*	01.12.2008

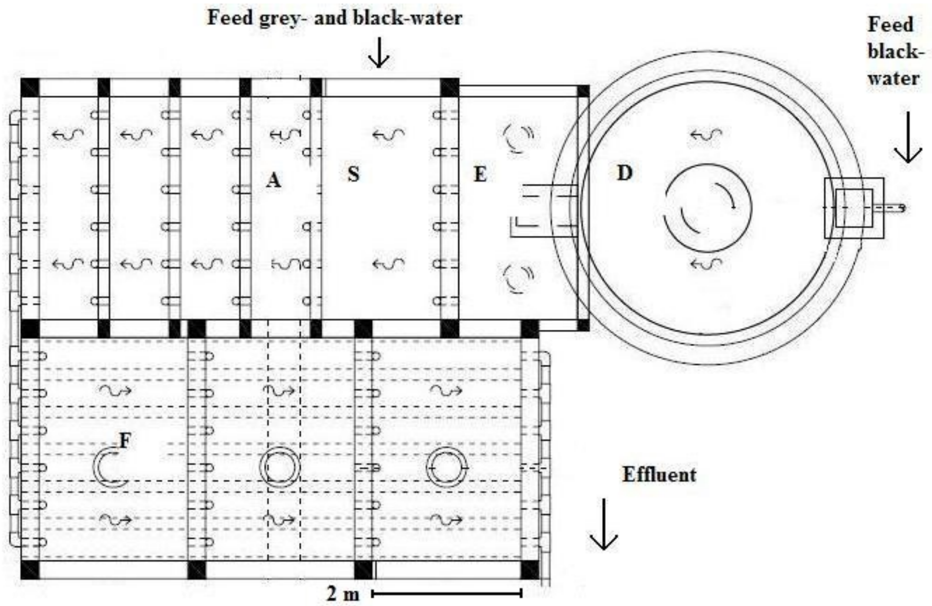

Figure 83: Schematic diagram (top-view) of the DEWATS plant built in Gambiran/ Yogyakarta with biogas-digester (D), expansion chamber (E), settler (S), ABR (A), anaerobic filter (F); water-depth of the system: 2,000 mm

6.4.3. Field observations

Chamber inspections showed that water levels had fluctuated to the point of exceeding the down-flow pipe height. Such high water levels had never been observed during times of peak flow on dry weather days. It is therefore concluded that storm water frequently intruded into the plant through the reticulation system.

As can be seen on the photographs below, several chamber supernatants were covered by a floating scum layer. The scum layer in the expansion chamber was thickest and about 2 cm strong. AF 1 had considerable amount of sludge flocks floating just below water surface.

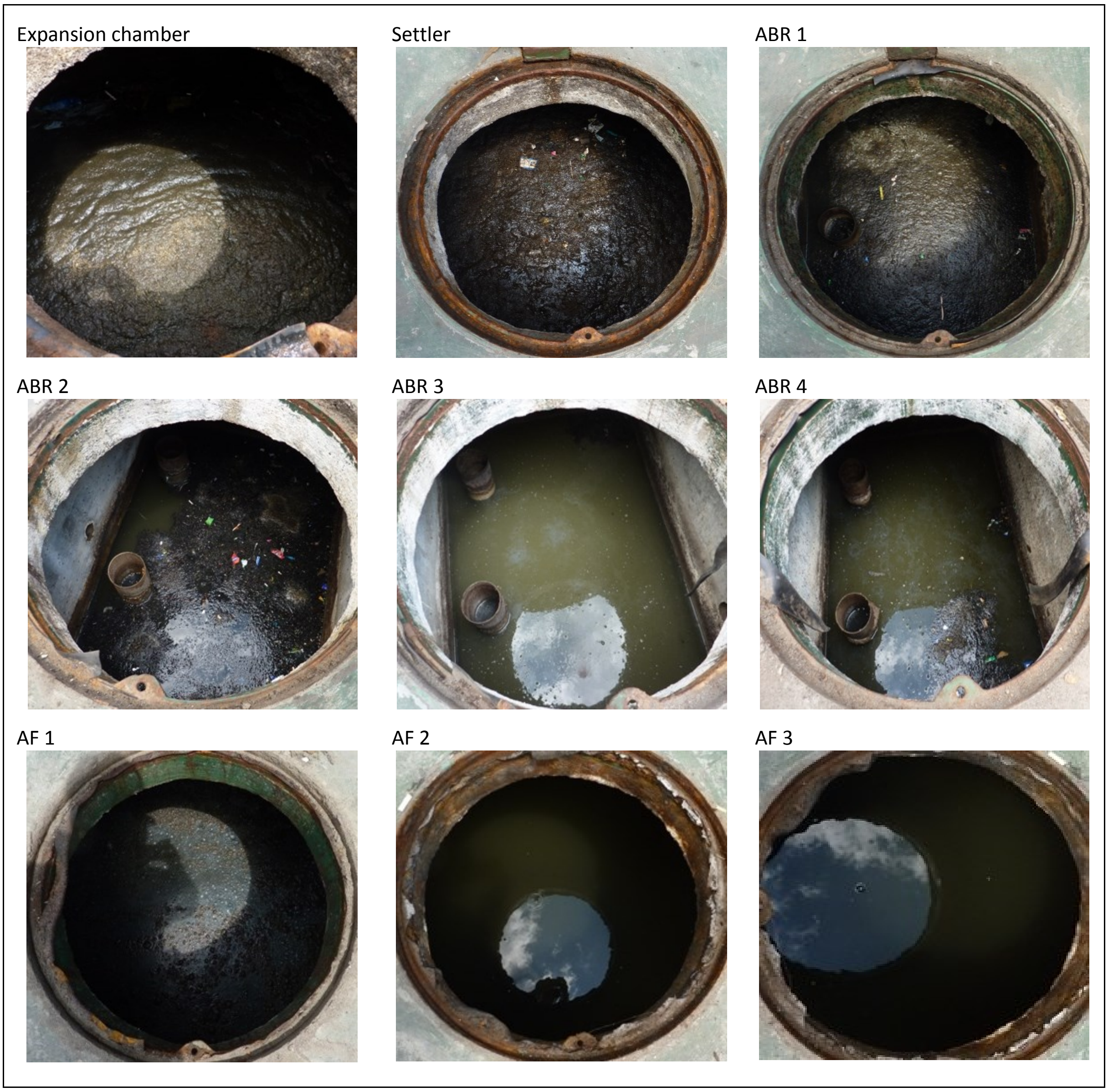

Figure 84: Settler, ABR and AF chamber supernatants as photographed on 02.09.2013

6.4.4. Monitoring results: load estimation and exposure to flow surges

6.4.4.1. Users

A detailed user-count in 2009 yielded 68 connected households with a total of 195 residents. A census in 2011 confirmed this number and the population size was reported by the head of the CBO to have remained constant until 2013. The connected population size represents 98% of the assumed design value.

6.4.4.2. Flow

Figure 85 shows the average diurnal flow patterns as calculated from measurement data produced in 2009. The flow measurements were performed during the dry season and are therefore not influenced by rain. The average daily flow was 16.6 $m^3\ d^{-1}$ implying a daily per capita wastewater production of 85 l. The wastewater production is assumed to have remained constant over the complete period of investigation. The validity of this assumption is supported by the constant number of user connections over this period (see previous section).

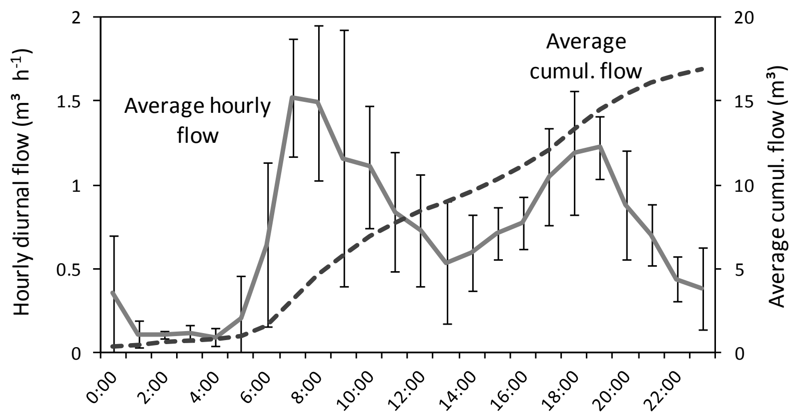

Figure 85: Average flows measured in 2009, averages were calculated with data from 07.08.2009 to 15.08.2009, error-bars indicate the standard deviation of hourly flows over this period

6.4.5. Monitoring results: sludge composition, build-up and activity

6.4.5.1. Sludge heights

Figure 86 shows a selection of the available sludge height data in each ABR chamber after the start of operation in December 2008. Most data points represent the average of triplicate measurements. The head of CBO reported that no ABR chamber was ever desludged. Chamber AF 3 never contained sludge.

Generally, sludge increase was observed in all chambers with the exception of ABR 2 and 3 in 2012, which could be due to sludge washout into the later reactor compartments ABR 4 and AF 1 that year. Settler and ABR 1 generally had the lowest sludge levels which is surprising since these chambers were logically exposed to the largest organic load. Until end 2011, during the first three years of operation, highest sludge heights were found in ABR 2 and ABR 3 followed by a shift in 2012 and 2013 to ABR 3 and ABR 4.

No sludge was found in AF 3. By far most sludge accumulated in AF 1 with a noticeable surge in 2013. This sudden increase was also observed in AF 2 where the sludge height rose from 5 cm to 40 cm in one year.

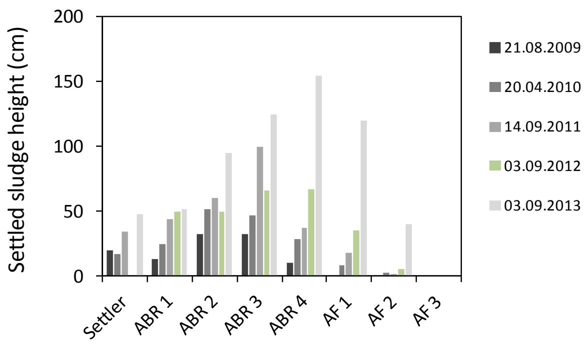

Figure 86: Selection of measured settled sludge levels in Gambiran

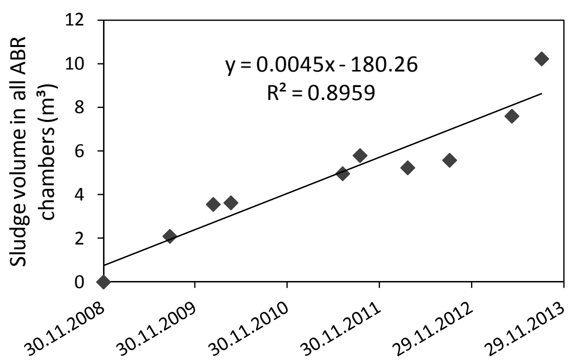

Figure 87: Total ABR sludge volume evolution in Gambiran

6.4.5.2. Sludge volume increase

Figure 87 shows the increase of total ABR sludge volume over the time of operation. Linear regression of the data indicates an approximate sludge increase of 4.5 l d^{-1} or 1.8 m^3 y^{-1}.

Little sludge accumulated inside the ABR in 2011 and 2012 after which the accumulation suddenly increased in September 2013. Sludge levels also considerably rose inside AF 1 and 2 at that time. The reason could have been sludge washed out from the digester, which by then had been operating already for 4 y without desludging. Also strong rain had been reported in July 2013. Future measurements are needed to confirm this trend.

By end 2013 approximately 7 m^3 of sludge had accumulated inside AF 1 and AF 2 which corresponds to an annual sludge increase of approximately 1.5 m^3 y^{-1}.

Most sludge increase occurred in the last chambers of the ABR reactor: 14%, 21%, 28% and 37% of the total ABR sludge build-up took place in ABR 1, ABR 2, ABR 3 and ABR 4 respectively.

6.4.5.3. Sludge Total and Volatile Solids concentrations

The composition of settled sludge was measured for most reactor chambers in 2013 as part of the SMA investigations. The results are shown in Figure 88. The TS concentration tended to be highest in the settler and lower in the following reactors while the VS concentration remained approximately constant across all reactors.

The across ABR chambers average TS and VS concentrations of settled sludge were approximately 80 g l^{-1} and 34 g l^{-1} respectively with a respective standard deviation of 13% and 17%.

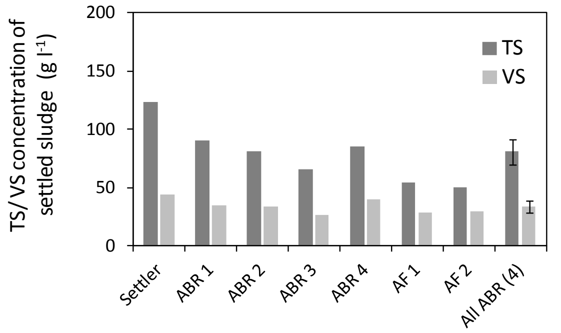

Figure 88: Settled sludge TS and VS concentration profiles, single measurements, "All ABR" bars represent averages of all ABR values, error-bars indicate the standard deviations across all ABR values

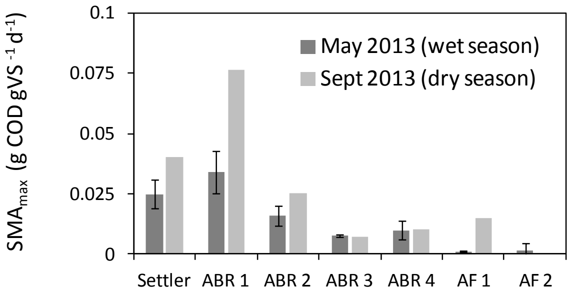

Figure 89: SMA_{max} of sludge sampled from different reactors in 2013, all samples were processed within one week after sampling, error-bars indicate the effect of the VS measurement standard deviation in May 2013 which was high (> 10%), all values are derived from single measurements

6.4.5.4. Specific methanogenic activity (SMA) of sludge

Figure 89 shows the SMA_{max} values derived from sludge activity measurements performed on sludge sampled in each reactor compartments. The May values were affected by poor TS and VS measurement results since the standard deviation of triplicate measurements of most samples was above 10%. This inaccuracy is reflected by the error-bars in the figure.

Measured SMA values were highest in the first ABR compartment followed by the settler and by the second ABR. Measurements performed during the wet and dry seasons showed a significant SMA increase in the settler, ABR 1 and ABR 3 and AF 1. Data from ABR 3 and ABR 4 however did not indicate such an increase.

6.4.6. Monitoring results: alkalinity, pH, temperature and nutrient concentrations

The alkalinity of the well water used by one household in the community was measured once in 2010 to yield 132 mg $CaCO_3$ l^{-1}.

Figure 90 and Figure 91 present the general process parameters alkalinity and pH. The average alkalinity in the expansion chamber was above 600 mg $CaCO_3$ l^{-1}, almost five times the well water value. This increase over the digester is expected since hydrolysis of urea and anaerobic digestion produce alkalinity. The alkalinity then significantly dropped after the expansion chamber to the first settler probably because the wastewater stream mixed with grey-water and black-water that bypassed the digester. It then remained constant throughout the whole treatment. The median pH values were at pH 7 in all chambers and minimum values never fell below 6.5, indicating stable anaerobic conditions. The wastewater temperature always remained between 27°C and 30°C averaging at 28.5°C.

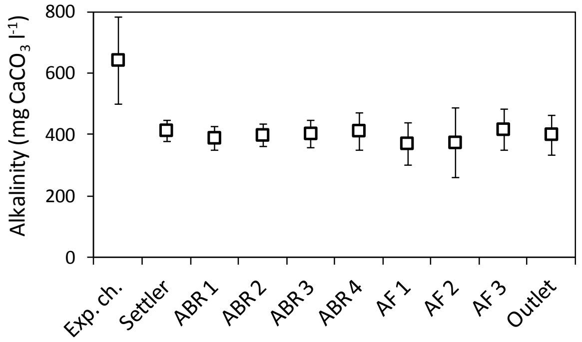

Figure 90: Average alkalinity concentration profile across reactor chambers as measured from 2010 to 2013, error-bars indicate standard deviation, 3 to 8 data points per sampling point

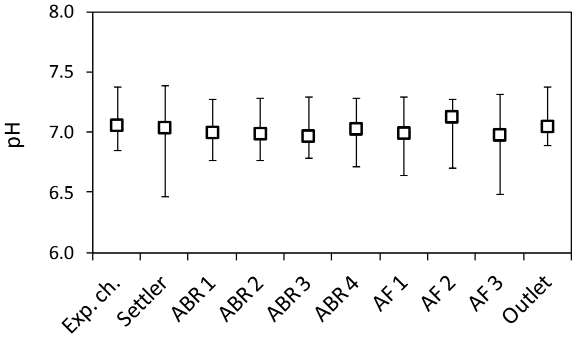

Figure 91: Maximum, median pH profiles across reactor chambers as measured from 2009 to 2013, error-bars indicate min and max values, 8 to 13 data points per sampling point

Nutrient investigations yielded 94 mg NH_4-N l^{-1} and 4.9 mg PO_4-P l^{-1} in the ABR feed and 65 mg NH_4-N l^{-1} and 6.4 mg PO_4-P l^{-1} in the effluent. All four values are averages from two sampling campaigns with duplicate measurements each.

6.4.7. Monitoring results: reactor COD concentrations and COD removal rates

No seasonal variation was found in the available COD_t dataset (see Figure 92). Time series data were therefore averaged over the complete period of investigation for further data interpretation.

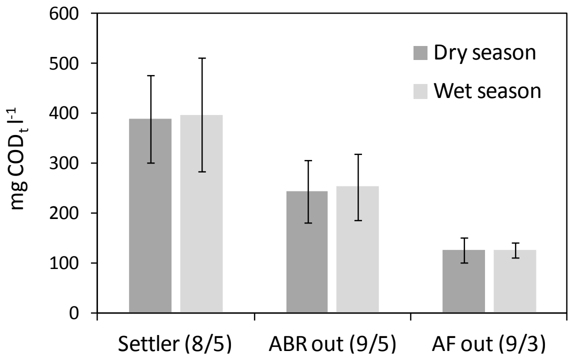

Figure 92: Average COD$_t$ concentrations measured at settler effluent, ABR effluent and AF effluent, “dry season” is defined as the months May to September, “wet season” is defined as the months October to April, the numbers in brackets indicate the number of measurements made during dry and wet season respectively

The settler COD$_t$ and COD$_p$ concentrations measured on September 2nd 2013 were identified as outliers and removed from further analyses (see Figure 93). The COD from samples taken in AF 1 were significantly higher in 2013 than in the preceding years (Figure 94).

The feed ABR concentrations including their standard deviations were 393 ± 94 mg COD$_t$ l^{-1}, 217 ± 90 mg COD$_p$ l^{-1}, 159 ± 47 mg COD$_s$ l^{-1}.

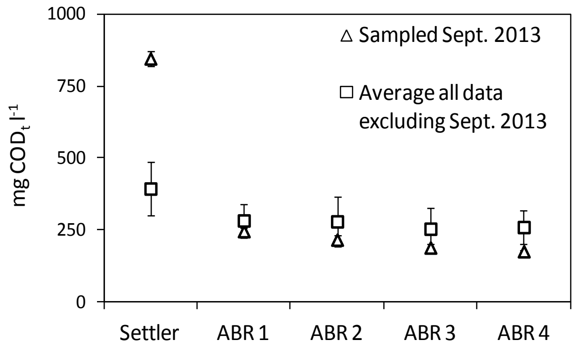

Figure 93: COD$_t$ concentration profiles across ABR chambers and outlier value measured in September 2013

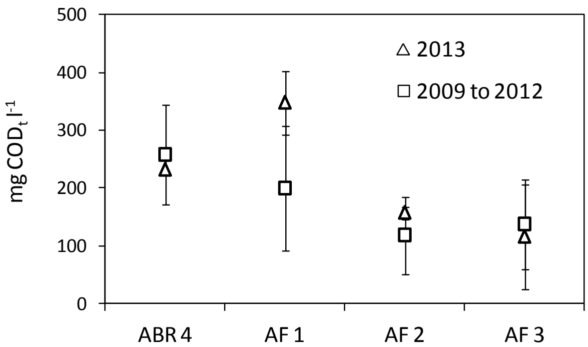

Figure 94: Comparing COD$_t$ concentration profiles across AF chambers measured in 2013 and before

Figure 95 presents the average COD fractions measured in the supernatants of the different reactor compartments. Based on this, the following hypotheses are formulated for statistical testing:

1. Significant COD$_p$ reduction occurred in ABR 1 and AF 2.
2. Significant COD$_s$ reduction occurred in AF 2.
3. No further significant COD reduction occurred in the plant.

An unpaired 2-sample t-test (significance level 5%) was used to test hypothesis 1 and showed significant reduction of COD$_p$ between 2nd settler and ABR 1 (2nd settler: M = 217, SD = 90, ABR 1: M = 136 , SD = 39); t(21) = 2.07, P = 0.01 and between AF 1 and AF 2 (AF 1: M = 149, SD = 73, AF 2: M = 45, SD = 40); t(21) = 2.08, P = 3*10^{-4}.

An unpaired 2-sample t-test (significance level 5%) was then used to test hypothesis 2 which showed significant reduction of CODs between AF 1 (M = 134, SD = 21) and AF 2 (M = 94, SD = 30); t(21) = 2.08, P = 0.001.

Hypothesis 3 was tested by using two one-way between subjects ANOVA tests to compare the average COD_p values measured in all chambers from ABR 1 to AF 1. There was no significant difference at the $p < 0.05$ level [$F(4, 54) = 0.88$, $F_{crit} = 2.54$]. The same test was used to compare the average COD_s values measured in all chambers from Settler to AF 1. There was also no significant difference at the $p < 0.05$ level [$F(5, 55) = 2.07$, $F_{crit} = 2.35$].

All 2 hypotheses are therefore supported by statistical tests: significant COD_p reduction only occurred in ABR 1 and AF 2, significant COD_s reduction only in AF 2 and none whatsoever in the ABR.

Figure 96 presents the average removal rates of the reactors. The average COD_t removal was 37% for the ABR and 49% for the AF. Most reduction in the ABR and AF was mediated through COD_p retention.

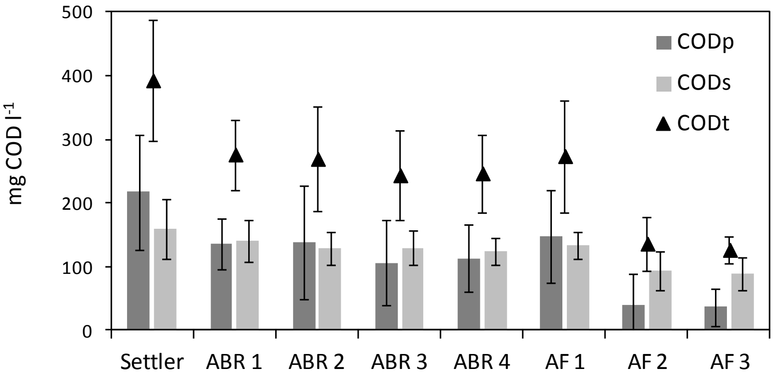

Figure 95: Average total, particulate and soluble COD profiles across reactor chambers as measured from 2009 to 2013, averages are calculated with 12 to 15 data points per sampling point, error-bars indicate standard deviations

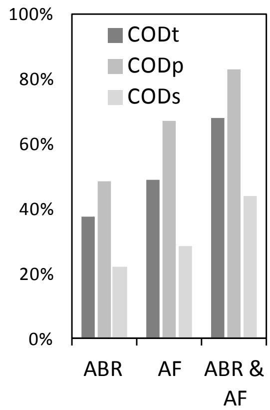

Figure 96: Average removal rates of COD fractions in ABR and AF

The COD effluent concentration measured in AF 3 was 107 (± 22) mg COD_t l^{-1} with a non biodegradable fraction of approximately 20 mg COD_s l^{-1} (see Appendix A3). Eighteen AF3 effluent BOD_5 measurements were performed between 2009 and 2013 and yielded an average effluent concentration of 69 (± 25) mg BOD_5 l^{-1}. This corresponds to 54% of the average COD_t effluent concentration.

6.4.8. Discussion of case study data

6.4.8.1. Plant feed characteristics

A flow measurement campaign in 2009 yielded an average daily flow of 16.6 m^3 d^{-1} and an average per capita flow of 85 l cap^{-1} d^{-1}. The number of people connected to the plant was approximately constant over the entire investigation period and was about 98% of the assumed design user number.

Feed concentration measurements were not undertaken at this plant and would have been complicated to implement because of the black-water and grey-water split in this system. Based on the average measured ABR feed concentration, an assumed 50% COD_t reduction through settler and digester would imply an average feed concentration of approximately 800 mg COD_t l^{-1} and a per capita COD load of approximately 70 g COD d^{-1} (this issue is further discussed in Section 6.7.1.4).

A single well water measurement indicated a raw-water alkalinity of approximately 130 mg $CaCO_3$ l^{-1} which is comparably low.

6.4.8.2. Effect of flow surges on the plant

Signs of storm water intrusion to the plant have been observed inside the reactors. Sludge accumulated fastest in the rear ABR compartments and it is hypothesised that this occurred because of sludge migration from the earlier compartments due to extreme flow peaks. Large amounts of sludge were also accumulating inside the first AF chamber, trapped below the AF growth media. Sludge washout from the AF into the receiving water body however does not seem probable at the time this investigation ended, since by then no sludge was measured in the last AF chamber.

SMA measurements indicate a significant activity increase in the settler and the first 2 ABR chambers after a 40 d period without rain. It is therefore hypothesised that sludge activity was impeded because of extreme peak flows during strong rain events.

The sludge accumulating inside the last ABR chambers and the AF had a very low level of methanogenic activity. This implies either that the active acetoclastic methanogens from the first ABRs were comparably resilient to sludge washout or that the conditions inside the rear ABR chambers and the AF were not supportive for acetoclastic methanogens.

6.4.8.3. Estimated ABR and AF load and operation

6.4.8.3.1 ABR and AF reactor load and performance

Figure 97 a, b and c place the observed OLRs and HRTs of ABR and AF into relation with design assumptions. OLRs were calculated with the average measured Q and COD_t concentrations. Error-bars take a flow variation of 20% and the COD_t concentration standard error of means into account. HRTs were computed with the same Q as the OLRs.

Because of low actual feed concentrations, observed OLRs, especially for the ABR, were lower than those which the reactors were designed for (see Figure 97 a and b). The observed hydraulic loads on the other hand were slightly lower than the design values but generally close. The average $v_{up,max}$ was 0.6 m h^{-1} and therefore slightly below the design value of 0.8 m h^{-1}.

Measured alkalinity and pH values indicate good anaerobic treatment conditions throughout the reactors.

ABR and AF treatment were statistically significant for COD_p and COD_s but also considerably below design expectations. The COD reductions in ABR and AF were shown to have been mediated mainly through COD_p retention.

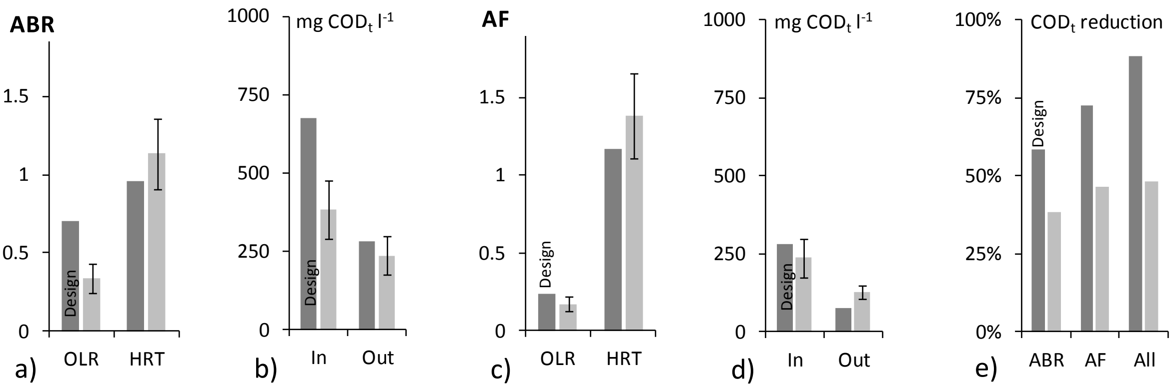

Figure 97 a, b, c, d and e: Loading and treatment parameters of ABR and AF reactors: OLR, HRT, feed and effluent COD concentrations and COD reduction rates, OLR error-bars indicate combination of standard error of mean of COD_t measurements and standard deviation of Q, all other error-bars indicate standard deviations of concentration measurement results

6.4.8.3.2 COD_p mass balance in ABR

COD_p mass balance was calculated as detailed under Section 3.8.2.

The averages of measured values for Q, ABR_{in} COD_p and ABR 5 and VS concentration of sludge yielded a 13.0 (min = 5.3, max = 26.4) $m^3\ y^{-1}$ sludge increase assuming no anaerobic digestion. Minimum and maximum values take into account a feed flow variation of 20%, the standard error of means of COD_p concentrations and the standard deviation of sludge VS concentration data.

Linear regression of sludge volumes measured in the 6 ABR chambers led to a sludge build-up rate of 1.8 $m^3\ y^{-1}$. This is below the minimal rate calculated through mass balance. This discrepancy could not be explained through unnoticed sludge washout on days on which no wastewater sampling took place, since the sludge build-up inside the AF compartments was found to only be approximately 1.5 $m^3\ y^{-1}$.

The result therefore supports the hypothesis that anaerobic digestion did take place inside the ABR and significantly reduced the volume of retained biodegradable COD_p. The further testing of this hypothesis with anaerobic digestion modelling is described in Chapter 7.

6.4.8.3.3 Compartment performance ABR and AF

Figure 98 compares the average COD_t measurement data with predictions given by the ABR and AF design calculation (curve "Design prediction"). The input values for these calculations were the average measured flows and feed concentrations.

The curve "Initial design" indicates the treatment assumed at the design stage of the plant with a significantly higher feed concentration and steeper COD reduction curve over the reactors, especially the ABRs.

The design calculation generally overestimates ABR and AF COD removal.

The ABR reduction curve "Design prediction" falls within the confidence limits of the field data until ABR 2 after which the field data shows no more treatment until AF 1.

The COD_p reductions across ABR single chambers were only statistically significant in the first ABR compartment (see Figure 95). ABR 1 was however the chamber with the least sludge build-up. This apparent contradiction documents the fact that both measurements represent different phases of reactor operation: wastewater samples for COD analyses were taken on days with minimal rain water influence on the system. Sludge height increase on the other hand represents the result of all external influences, integrated over the time between two sludge level measurements. It is therefore probable that during normal dry weather operation, particulate wastewater components are best retained in the first ABR compartment. Flow surges then lead to the migration of such retained sludge further down the treatment train.

ABR 1 is also the chamber in which the sludge yielded the highest SMA_{max} value, even at the end of the wet season. The reason for this is hypothesised to be a comparably high substrate availability in the first ABR chambers.

The main treatment mechanism across the entire AF was shown to be particle retention in only one chamber: AF 2. Why this was not also observed in other AF chambers is uncertain. The specific amount of sludge accumulated below the growth media of AF 2 may have improved its filtering characteristics. The extremely high sludge levels observed in AF 1 on the other hand lead to contamination of its effluent with floating solids (as observed in the field, see Figure 84 g). It would therefore be expected that the AF 2 COD_p treatment would worsen by the time its sludge level reaches a level similar to AF 1.

AF effluent was found be largely biodegradable (see Section 6.4.7). It is therefore hypothesised that the reactor had not reached its full treatment potential at the time this study was carried out.

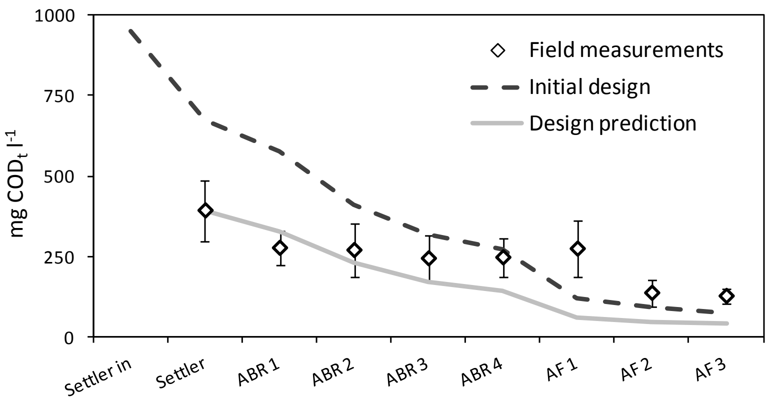

Figure 98: Measured average COD_t concentration profile, initial design prediction („Initial design“) and design prediction with input variables adjusted to measured field values („Design prediction“)

6.5. Case study C: Minomartani (MM)

6.5.1. The community

Most families in this community rely on a secure income with well-paid government and university positions, which reflects the comparatively good housing and overall clean living conditions. The formal unemployment rate is low and almost no inhabitants work in the informal sector. In 2011 the head of community estimated an average monthly household income of above 280 USD. No home industries have been reported. The households have unrestricted access to fresh water either through private wells or municipal connections.

6.5.2. System setup and technical details

Table 31 summarises the setup, technical properties and design values of this plant. The setup includes two settlers, six ABR chambers and six AF chambers (see Figure 99). The plant was not seeded before start-up. The ABR chambers were never desludged.

Table 31: Plant setup, design properties and photo of the site (last AF chamber at front)

Plant name		**Minomartani**
Country/ Town		Indonesia/ Yogyakarta
Design	*Plant setup (reactor sizes)*	2 Settlers (11.25 m³), 6 ABRs (21 m³), 6 AFs (37.3 m³)
	Connected households	67
	Connected users	350
	Per cap ww production	80 l cap^{-1} d^{-1}
	Per cap organic load	152 g COD cap^{-1} d^{-1}
	Daily flow, Q_d	28 m³ d^{-1}
	Hydraulic load ABR	1.3 m³ m^{-3} d^{-1}
	Average $v_{up,max}$	1.2 m h^{-1}
	Organic load ABR	1.4 kg COD m^{-3} d^{-1}
Operation	*Start of operation*	December, 2006

The plant is designed in an L-shape. For convenience Figure 99 represents it as a straight design.

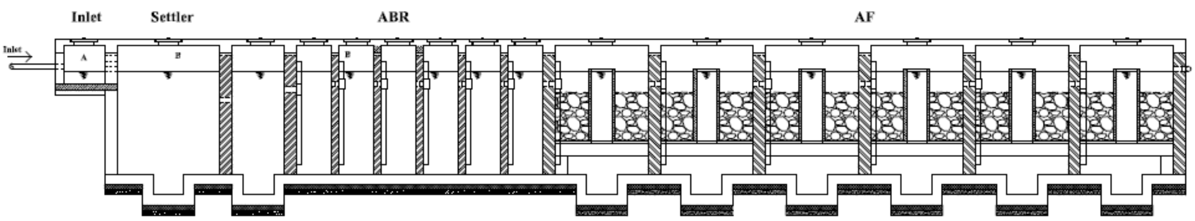

Figure 99: Schematic diagram of the DEWATS Minomartani/ Yogyakarta (side-view), depth of the system: 2,000 mm

6.5.3. Field observations

Chamber inspections showed that water levels fluctuate to the point of exceeding the down-flow pipe height. Such high water levels have never been observed during times of peak flow on dry weather days. It is therefore concluded that during tropical rains, storm water intrudes into the plant through the reticulation system.

The head of community also reported that during extremely strong rain the system would completely fill with water to a point where water would be pressed out of the closed manhole covers.

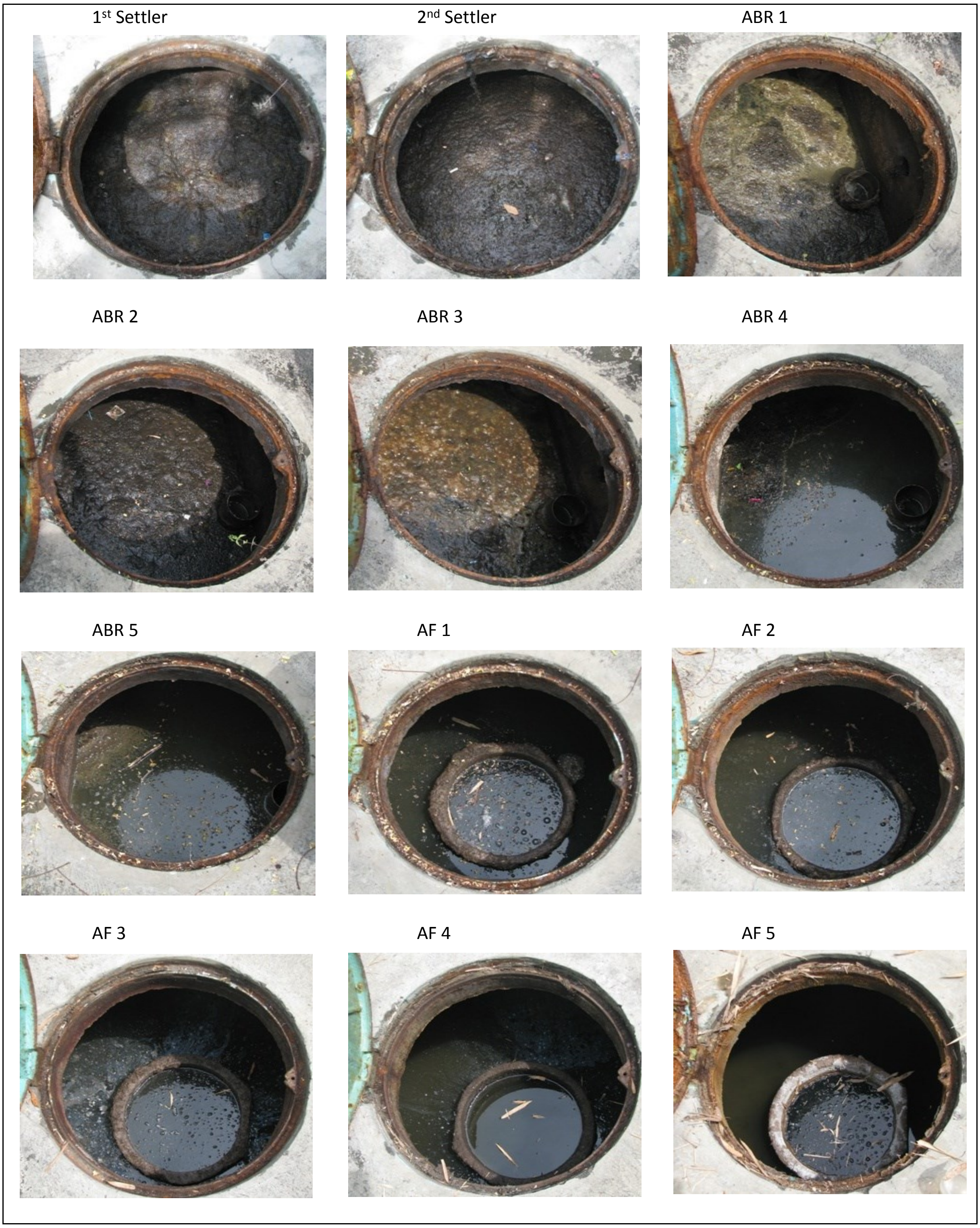

Figure 100: Settler, ABR and AF chamber supernatants as photographed on 16.08.2013

6.5.4. Monitoring results: load estimation and exposure to flow surges

6.5.4.1. Users

User-number investigations in 2009, 2010 and 2011 all showed a population of approximately 250 people which represents 71% design value (350 people). The same number was confirmed by the head of the CBO in 2013.

6.5.4.2. Flow

Figure 101 and Figure 102 show the average diurnal flow patterns as calculated with data from field investigations in 2009 and 2010. Measurements in 2009 were performed during the dry season and it did not rain for the entire campaign. The error-bars indicate the standard deviation of hourly flows over the measurement period. Measurements in 2010 were taken during the wet season. The presented averages however exclude data from days with strong rainfall above 5 mm d^{-1}. Flow data from days with precipitation of 5 mm d^{-1} or less did not seem obviously influenced by rain. The average daily flows in 2009 and 2010 were 22.9 m^3 d^{-1} and 33.5 m^3 d^{-1} respectively (for daily flow data refer to Appendix A6). The flow pattern in 2010 shows an almost consistent increase of 0.4 m^3 h^{-1} on every hour of day and night compared to the measurement one year earlier. This is remarkable since the user number was constant over both years. The values above imply a change of daily per capita ww production from 91 l to 134 l.

It cannot be excluded that a change of user habits led to a general increase of per capita wastewater production. It was noted that the ritual washing facilities at a nearby mosque were connected to the plant between both measurement campaigns. This in itself however would not account for the observed constant increase over day and night since discharges at the mosque would occur discontinuously at prayer times. Another influencing factor may be precipitation, since only data obviously influenced by strong rain (such as shown in Figure 103) was excluded from the dataset. Rain from comparably low intensity events (below 5 mm d^{-1}) or water from the saturated soil may have continuously infiltrated the reticulation system, thus contributing to the flow without obviously affecting the typical diurnal flow pattern. The overall increase at night would support this hypothesis since changes in user habits and the contribution of the mosque can be excluded as constant influencing factors at night. The data therefore indicates a probably constant rain water ingress during the wet season, even on days on which little or no precipitation occurred. The average daily flow of both measurement campaigns was used for further data analyses.

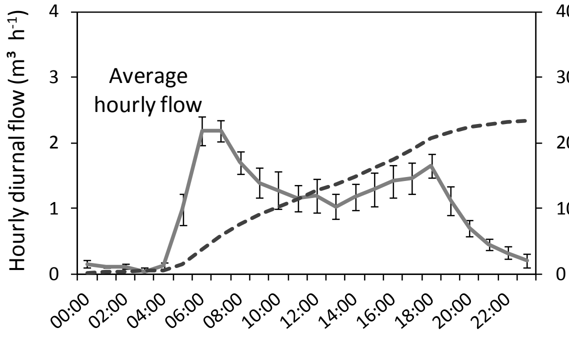

Figure 101: Average flows measured in 2009, averages were calculated with data from 10 d (16.07.2009 to 25.07.2009)

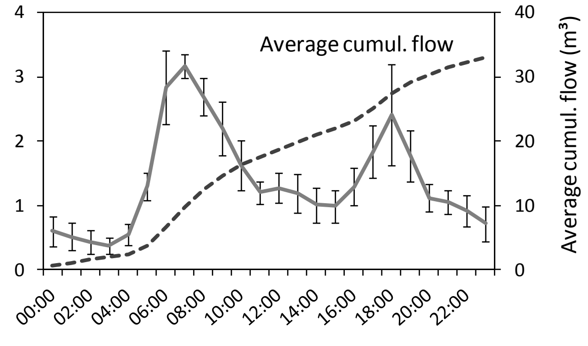

Figure 102: Average flows measured in 2010, averages were calculated with data from 6 d (11.12.2010 to 16.12.2010)

The effect of storm water on the system mentioned above is further confirmed by effluent flow measurements on rainy days. The flow was significantly influenced by precipitation and showed flow peaks clearly exceeding the standard deviation of dry weather average flow (see Figure 103).

Local precipitation measurements indicate daily rainfall of up to 200 mm d^{-1} (data not shown). DEWATS effluent flow could not be recorded on those days. Extrapolating the information in Figure 103 and assuming a linear relationship between precipitation and storm water ingress would imply peak flows of over twenty times the design value with such precipitations.

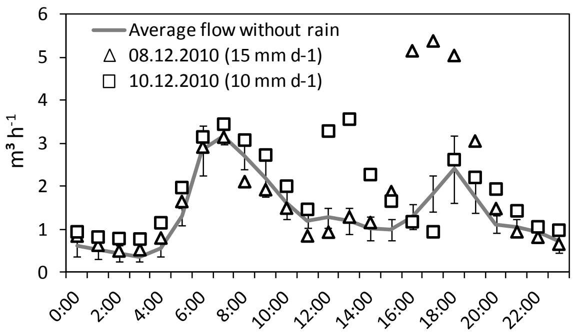

Figure 103: Effluent flows recorded on rainy days, average flow was calculated with data not obviously affected by rain recorded from 11.12.2010 to 16.12.2010, numbers in brackets behind the dates indicate the respective daily precipitations

6.5.4.3. Summary of system loading results

The plant operates at about 70% design organic and about 80% and 120% design hydraulic load, probably depending on the season.

The observed average daily flow pattern indicates a maximum up-flow velocity inside the ABR chambers of 1.8 m h^{-1} as opposed to 1.2 m h^{-1} design value.

Flow measurements during the dry season need to be repeated in order to confirm this since an increase of per capita wastewater production due to changes in user habits cannot be excluded at this point.

It became evident that the plant was exposed to large amounts of storm-water during the wet season probably leading to severe rising of reactor water levels and ABR up-flow velocities probably as much as twenty times the design value. Table 32 summarizes the available information on load parameters for this plant.

Table 32: Summary of load parameter values excluding the influence of storm-water

Time of measurement	User number	User number vs design	Average daily flow	Average daily flow vs design	HRT ABR chambers	Per capita ww prod.	Average $v_{up, max}$ in ABR*
	User	%	m³ d^{-1}	%	d	l cap^{-1} d^{-1}	m h^{-1}
Design	350	100%	28	100%	0.8	80.0	1.2
2009_07	250	71%	22.9	82%	0.9	91.6	1.3
2010_12	250	71%	33.5**	120%**	0.6**	134.0**	1.8**

* maximum average hourly flow, ** probably influenced by rain water

6.5.5. Monitoring results: sludge composition, build-up and activity

6.5.5.1. Sludge heights

Figure 104 shows a selection of the available sludge height data in each ABR chamber after the start of operation in December 2006. Representing all available data in the graph would make the chart too confusing. (A compilation of all sludge height data can be accessed as explained in Appendix A6) Most data points represent the average of triplicate measurements. No desludging of ABR chambers was reported.

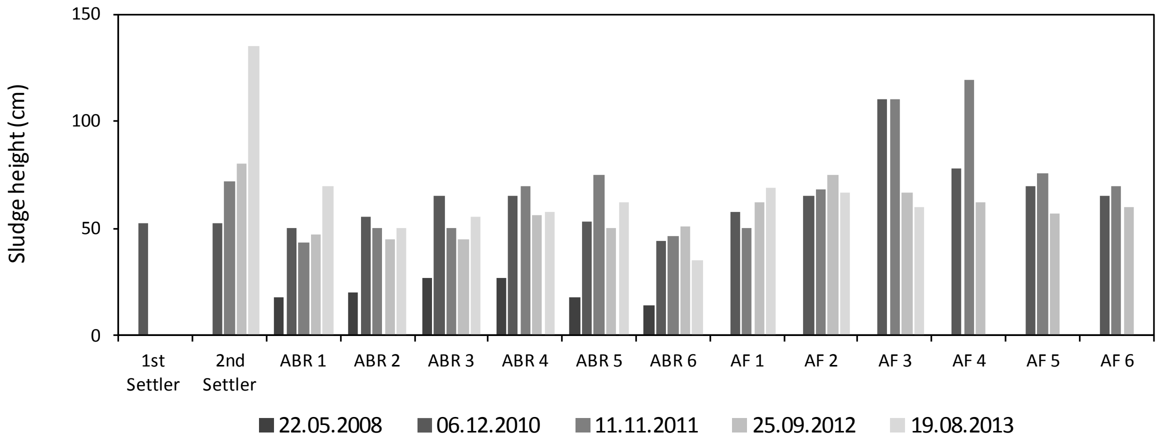

Figure 104: Selection of measured settled sludge levels in Minomartani

Most ABR chambers contained similar sludge heights with the exception of chambers 4 and 5 having generally the highest and chamber 6 having generally the lowest sludge levels. A slight shift of the highest sludge levels towards the rear chambers was observed from 2010 to 2013.

AF chamber sludge heights were measured since 2010. All measurements showed sludge heights above 50 cm indicating that washed out ABR sludge was being retained under the AF growth media. The freeboard between reactor bottom and filter material is designed with 600 mm. This space was completely filled with sludge in all AF chambers by December 2010, 4 years after plant start-up. Between 2010 and 2013, sludge height changed very little. Assuming a constant sludge washout from the ABR to the AF, the reason for this could be sludge compaction, washout or a combination of both.

6.5.5.2. Sludge volume increase

Figure 105 shows the increase of total ABR sludge volume over the time of operation. Linear regression of the data indicates an approximate sludge increase of 2.1 l d^{-1} or 0.8 m^3 y^{-1}.
The sludge heights measured inside the AFs in December 2010 correspond to an approximate sludge volume of 16.2 m^3 and an annual sludge volume increase of 3.3 m^3 y^{-1}.

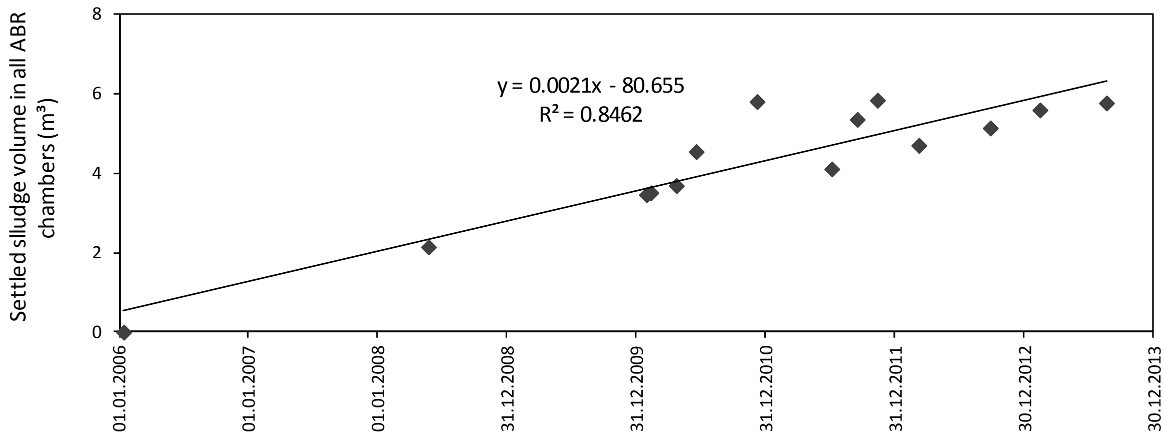

Figure 105: Total ABR sludge volume accumulation in Minomartani

The overall sludge volume accumulation was approximately constant across the chambers: 18, 15, 16, 19, 19 and 13% of the total ABR sludge build-up occurred in ABR 1, ABR 2, ABR 3, ABR 4, ABR 5 and ABR 6 respectively.

6.5.5.3. Sludge Total and Volatile Solids concentrations

The composition of settled sludge was measured for most reactor chambers on several occasions in 2013 as part of SMA investigations. Results are shown in Figure 106. There was a tendency of both TS and VS being highest in the two settler chambers and ABR 1. The values were rather constant in the following reactors.

The average settled sludge TS and VS concentrations across ABR chambers of Figure 106 were approximately 60 g l^{-1} and 33 g l^{-1} respectively. Both values had a relative standard deviation of approximately 25% across the ABR chambers.

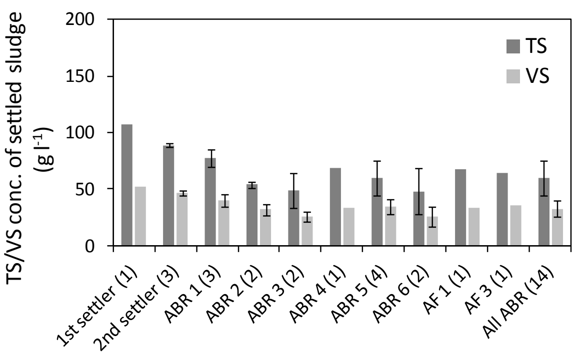

Figure 106: Settled sludge TS and VS concentration profiles, bars represent average values, "All ABR" bars represent averages of all ABR values, number of measurements is in brackets, error-bars indicate the standard deviation of multiple measurements

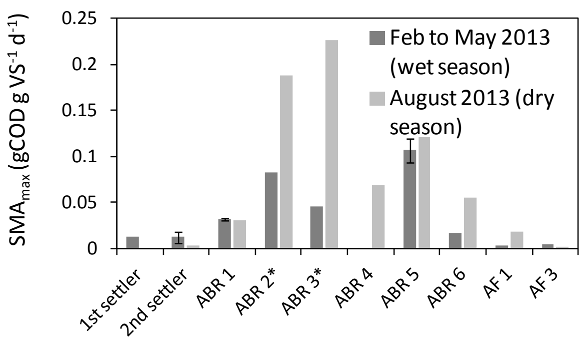

Figure 107: SMA_{max} of sludge sampled from different reactors in 2013, all sludges were processed within one week after sampling except when marked with *: time between sampling and measurement is 15 d in February, error-bars indicate the standard deviation of duplicate sequential measurements, all other values are derived from single measurements

6.5.5.4. Specific methanogenic activity (SMA) of sludge

Figure 107 shows the SMA_{max} values derived from sludge activity measurements performed on sludge sampled in each reactor compartments. The measurements for ABR 2 and ABR 3 in February 2013 underestimated the real activity since these samples were stored for 15 d before processing. Previous tests had shown that DEWATS ABR sludge activity significantly declines after a storage period of approximately one week (see Section 3.4.6).

Figure 107 indicates that general sludge activity was highest in the middle ABR compartments and very low in settlers and anaerobic filters.

A SMA_{max} of 0.23 g COD g VS^{-1} d^{-1} was measured in ABR 3 which is remarkably high. A benchmark SMA_{max} measurement campaign with highly active anaerobic sludge from a UASB reactor treating brewery process water in South Africa yielded only 0.21 g COD g VS^{-1} d^{-1} (Pietruschka, 2013).

The data shows a significant SMA_{max} increase from wet to dry season in ABR 2, ABR 3, ABR 6 and AF 1 after a period of approximately 40 d without rain. The increase exceeds the methodological variation assessed through repeated measurements during the wet season with sludge from the settler 2, ABR 1 and ABR 5. Data from ABR 1 and ABR 5 however does not indicate such an increase.

6.5.6. Monitoring results: alkalinity, pH, temperature and nutrient concentrations

The alkalinity of well water used by one household in the community was measured once in 2010 and once in 2013 yielding 115 and 138 mg $CaCO_3$ l^{-1} respectively.

Figure 108 and Figure 109 present the measured field values of the general process parameters alkalinity and pH. A slight increase of alkalinity can be observed over the first reactor compartments. It then remains generally stable across the reactors at an average concentration of about 300 mg $CaCO_3$ l^{-1}. The median pH values indicate good and stable anaerobic conditions with a general slight decrease towards the rear compartments. Minimum values however never went below 6.5. Measured wastewater temperature was always between 27°C and 30 °C averaging at 29 °C with little cross-seasonal variation.

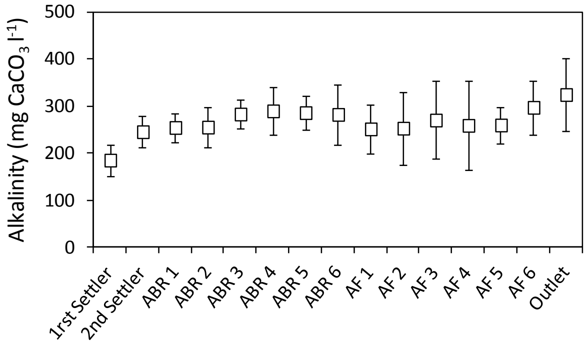

Figure 108: Average alkalinity concentration profile across reactor chambers as measured in 2010 and 2012, error-bars indicate standard deviation, 3 to 5 data points per sampling point

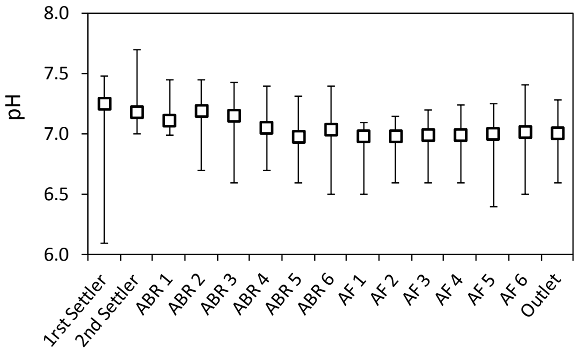

Figure 109: Median pH profiles across reactor chambers as measured from 2008 to 2013, error-bars indicate minimum and maximum measured values, 9 to 13 data points per sampling point

Nutrient investigations yielded 31 mg NH_4-N l^{-1} and 5 mg PO_4-P l^{-1} in the ABR feed and 49 mg NH_4-N l^{-1} and 5.8 mg PO_4-P l^{-1} in the AF effluent. All four values are averages from two sampling campaigns during two of which measurements were done as duplicates.

6.5.7. Monitoring results: reactor COD concentrations and COD removal rates

No seasonal variations in the ABR effluent and AF effluent were found in the available COD_t dataset (see Figure 110). Average settler effluent concentrations were lower during the wet season than during the dry season. Dilution of the incoming wastewater by rain appears unlikely since sampling was only performed on days without rain. On the other hand, groundwater infiltration caused by a higher ground water table during the wet season could be a plausible explanation for this phenomenon. However, the dataset is not large enough to make conclusive statistical testing. Time series data were therefore averaged over the complete period of investigation for further data interpretation.

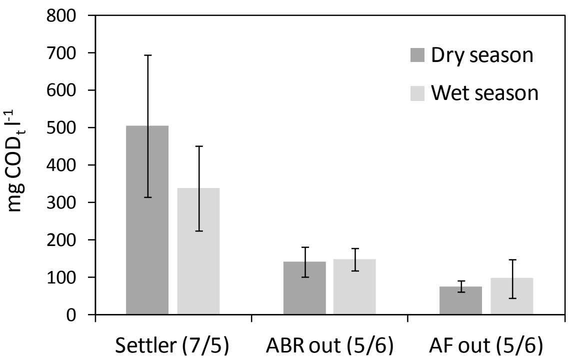

Figure 110: Average COD_t concentrations measured at settler effluent, ABR effluent and AF effluent, "dry season" is defined as the months May to September, "wet season" is defined as the months October to April, the numbers in brackets indicate the number of measurements made during dry and wet season respectively

COD_t and COD_p values measured on November 11th 2011 in ABR 2 and one COD_s value measured on September 15th 2011 in AF 2 were extremely high and by themselves made the datasets non-normally distributed. They were therefore removed from further analyses.

Figure 111 presents the resulting average COD_t, COD_p and COD_s concentrations as measured in the supernatants of the respective reactor chambers. Figure 112 shows the consequential reactor treatment efficiencies. The AF data is only analysed until the third chamber since it is known not to provide efficient treatment beyond. Also, more recent communal DEWATS AF designs never exceed two chambers.

Measured feed ABR concentrations including their standard deviations were 436 ± 178 mg COD l^{-1}, 270 ± 155 mg COD l^{-1}, 188 ± 62 mg COD l^{-1}, for COD_t, COD_p and COD_s respectively.

Generally speaking, average COD_p and COD_s values describe a nearly perfect first order reduction curve across reactor chambers. (The only exceptions are the high concentrations measured in AF 2 for which no plausible explanation could be found.) This is in line with the theoretical mechanistic and kinetic understanding of ABR treatment (higher treatment with higher organic pollution) and therefore appears plausible. However most reductions between neighbouring chambers were low compared to the standard deviations of single measurements.

Based on Figure 111 the following hypotheses were formulated for statistical testing:

- Significant COD_p and COD_s reduction occurs from the 2nd settler to ABR 3
- No further significant COD_p and COD_s reduction occurs in the following ABR chambers
- No significant COD_p and COD_s reduction occurs in the first three AF chambers

An unpaired 2-sample t-test (significance level 5%) was used to test the first hypothesis and showed significant reduction between 2nd settler and ABR 3 for COD_p (2nd settler: M = 270, SD = 155, ABR 3: M = 107, SD = 47); t(19) = 2.09, P = 0.005 and for COD_s (2nd settler: M = 188, SD = 62, ABR 3: M = 120, SD = 36); t(22) = 2.07, P = 0.004.

Two one-way between subjects ANOVA were conducted to compare the average COD_p values measured in the ABR 3, ABR 4, ABR 5 and ABR 6 and the average COD_s values measured in the same chambers. There was no significant difference at the $p < 0.05$ level for COD_p [$F(3, 36) = 2.53$, $F_{crit} = 2.87$] and COD_s [$F(3, 40) = 2.16$, $F_{crit} = 2.84$].

In other words, statistically significant COD_p and COD_s reduction only occurred in the first two ABR chambers and not in the last two ABR chambers.

An unpaired 2-sample t-test was then used to compare the combined concentrations measured in ABR 3 to ABR 6 with AF 3 values. Significant reduction between the combined last 2 ABR chambers and AF 3 was found for COD_p (ABR 3 to ABR 6: M = 82, SD = 44, AF 3: M = 40, SD = 23); t(44) = 2.02, P = 0.018 and for COD_s (ABR 3 to ABR 6: M = 104, SD = 31, AF 3: M = 66, SD = 38); t(51) = 2.01, P = 0.002. It was therefore concluded that the reduction of COD_p and COD_s in the first AF chambers was statistically significant.

Most reduction in the ABR occurs through COD_p retention. COD_p and COD_s reduction in the AF were approximately similar.

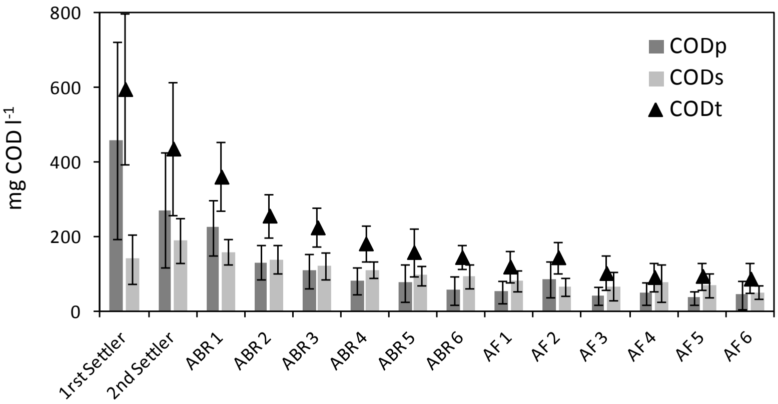

Figure 111: Average total, particulate and soluble COD profiles across reactor chambers as measured from 2010 to 2013, averages were calculated with 6 to 12 data-points, error-bars indicate standard deviations

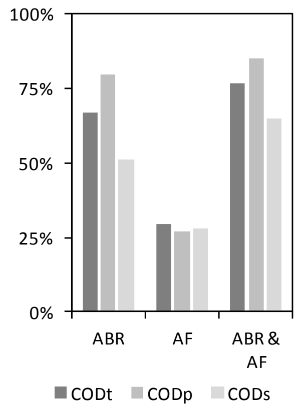

Figure 112: Average removal rates of COD fractions in ABR and AF (until AF 3)

The COD effluent concentration measured in AF 3 was 102 (± 46) mg COD_t l^{-1} with a non biodegradable fraction of approximately 20 mg COD_s l^{-1} (see Appendix A3). Nine BOD_5 measurements of the AF 6 effluent performed between 2008 and 2013 yielded an average effluent concentration of 42 (± 10) mg BOD_5 l^{-1}. This corresponds to 55% of the average total COD effluent concentration.

6.5.8. Discussion of case study data

6.5.8.1. Plant feed characteristics

Flow measurement campaigns performed in 2009 and 2010 yielded an average daily flow of 27.3 m^3 d^{-1} and an average per cap flow 109 l cap^{-1} d^{-1}. The number of people connected to the plant remained approximately constant over the entire investigation period and represented approximately 71% of the value expected at design stage.

Feed concentration measurements were not undertaken at this plant. Based on the average measured ABR feed concentration, an assumed 50% COD_t reduction through settler and digester would imply an average feed concentration of about 900 mg COD_t l^{-1} and a per capita COD load of approximately 95 g COD d^{-1} (this issue is further discussed in Section 6.7.1.4).

Well water measurements indicated a raw-water alkalinity of approximately 125 mg $CaCO_3$ l^{-1} which is comparably low.

6.5.8.2. Effect of flow surges on the plant

Storm water intrusion to the plant has been documented through flow measurements and field observations. It most probably leads to sludge migration within the ABR chambers and washout to the AF.

Sludge accumulation in all AF compartments was approximately 3.3 m^3 y^{-1} which exceeded the sludge accumulation inside the ABR by far. Sludge washout from the AF into the receiving water body appears probable since the sludge levels in all AF chambers (including the last) are high.

The measured SMA_{max} values of sludge from certain chambers significantly increased after a 40 d period without rain. A reduction of sludge activity due to extreme peak flows during strong rain events is therefore probable.

The sludge accumulating inside the AF was found to exhibit very low methanogenic activity. This implies that either the active acetoclastic methanogens were comparably resilient to sludge washout from the ABR or that the conditions inside the rear ABR chambers and the AF were not supportive for acetoclastic methanogens.

6.5.8.3. Estimated ABR and AF load and treatment

6.5.8.3.1 ABR and AF reactor load and performance

Figure 113 a and c put the observed OLRs and HRTs of ABR and AF in relation with design assumptions. The observed OLRs were calculated with the average measured Q and COD_t concentrations. The error-bars take a flow variation of 20% and the standard error of means of COD_p concentrations into account. The HRTs were computed with the same Q as the OLRs.

The observed OLRs, especially for the ABR, were lower than those for which the reactors were designed due to lower feed concentrations (see Figure 113 a and b). This can only partly be due to the low number of connected people: either the settler treatment was higher or the per capita COD production lower than assumed. The hydraulic field load on the other hand confirmed the design HRT values. The average $v_{up,max}$ was 1.5 m h^{-1} and therefore slightly above the design value of 1.2 m h^{-1}.

The measured alkalinity and pH values indicated good anaerobic treatment conditions throughout the reactors.

The reactor treatments of both ABR and AF were shown to be statistically significant for COD_p and COD_s but were slightly below design expectations. ABR COD treatment was mediated mainly through COD_p retention and AF COD treatment through COD_s reduction.

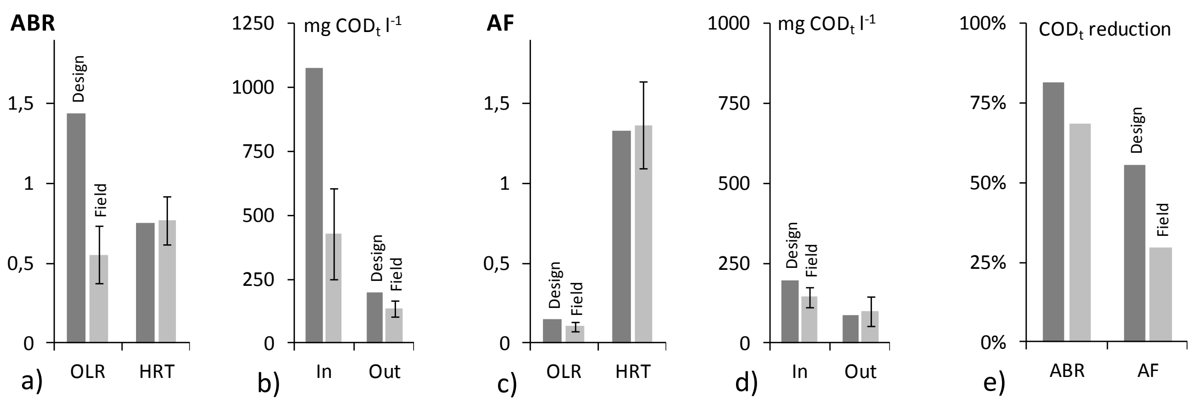

Figure 113 a, b, c, d and e: Loading and treatment parameters of ABR and AF reactors: OLR, HRT, feed and effluent COD concentrations and COD reduction rates, OLR error-bars indicate combination of standard error of mean of COD_t measurements and standard deviation of Q, all other error-bars indicate standard deviations

6.5.8.3.2 COD_p mass balance in ABR

The particulate COD mass balance was calculated as detailed in Section 3.8.2:

The averages of measured values for Q, COD_p of ABR_{in} and ABR 6 and VS concentration of sludge led to a 44.1 (min = 21.0, max = 87.1) $m^3\ y^{-1}$ sludge increase assuming no anaerobic digestion. Minimum and maximum values take into account a feed flow variation of 20%, the standard error of means of COD_p concentrations and the standard deviation of sludge VS concentration data.

Linear regression of sludge volumes measured in the six ABR chambers led to a sludge build-up rate of 0.8 $m^3\ y^{-1}$. This is far below the minimal rate calculated through mass balance. This discrepancy could not be explained through unnoticed sludge washout on days on which no wastewater sampling took place: the sludge build-up inside AF compartments was found to be only approximately 3.3 $m^3\ y^{-1}$.

The result therefore strongly supports the hypothesis that anaerobic digestion took place inside the ABR and significantly reduced the volume of retained biodegradable COD_p. The further testing of this hypothesis with anaerobic digestion modelling is described in Chapter 7.

6.5.8.3.3 Compartment performance ABR and AF

Figure 114 compares the average COD_t measurement data with predictions given by the ABR design calculation. The inputs for these calculations were the average measured flows and feed concentrations.

The curve “Initial design“ indicates the treatment assumed at the design stage of the plant with a significantly higher feed concentration and steeper COD reduction curve over the reactors, especially the ABRs.

The ABR reduction curve computed with the design calculation (“Design prediction”) however falls within the confidence limits of the field data. This is remarkable under the operational circumstances with extreme hydraulic flow surges.

The across single chamber COD_t, COD_p and COD_s reductions were shown to only be statistically significant in the first two chambers (see Figure 111). These were also the chambers in which the highest SMA_{max} values were measured. SMA_{max} values in ABR 2 and 3 even appeared to exceed SMA values measured with high rate anaerobic reactors. This is surprising due to the comparably low organic plant load and needs to be confirmed in future studies.

The design calculation slightly overestimates the AF COD removal. AF effluent was found be largely biodegradable (see Section 6.5.7). It is therefore hypothesised that the reactor had not reached its full treatment potential at the time this study was carried out.

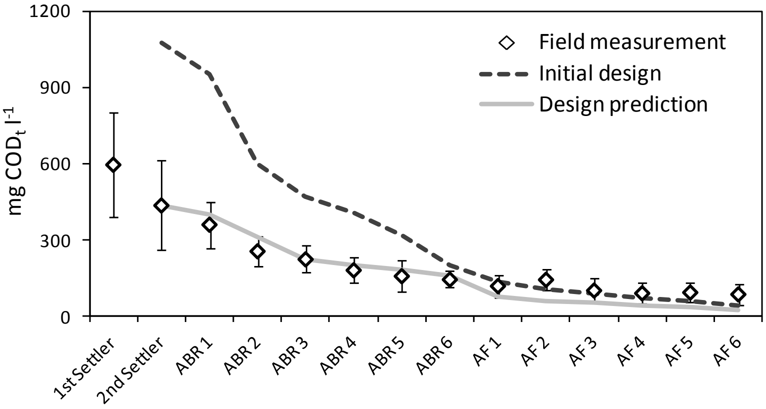

Figure 114: Measured average COD_t concentration profile, initial design prediction („Initial design“) and design prediction with input variables adjusted to measured field values („Design prediction“)

6.6. Case study D: Santan (ST)

6.6.1. The community

The households discharging their wastewater into the system are heterogeneous concerning their income but are mostly considered middle class with an average monthly household income of about 220 USD. Many connected houses are boarding homes for students.

Before the implementation of the DEWATS project, the wastewater of this community was directly discharged into the nearby river or disposed of in individual soak-pits. A number of laundry shops and small restaurants do exist in the neighbourhood. The operator however reported that these are not connected to the DEWATS. The households have unrestricted water access through private wells or municipal connections.

6.6.2. Setup and technical details

Table 33 summarises the setup, technical properties and design values of the plant. The households are connected by a small sewerage system to the first treatment step, a settler with two chambers. The next treatment step is an anaerobic baffled reactor (ABR) with five chambers followed by an anaerobic filter with two chambers (AF). No start-up material was used at the start of operation. No desludging was performed between the operational start of the plant and the end of the here presented investigation.

Table 33: Plant setup, design properties and photograph of the plant

Plant name		**Santan**
Country/ Town		Indonesia/ Yogyakarta
Design	*Plant setup (reactor sizes)*	2 Settlers (19.2 m³), 5 ABRs (32 m³), 3 AFs (31.2 m³)
	Connected households	
	Connected users	350
	Per cap ww production	100 l cap^{-1} d^{-1}
	Per cap organic load	97 g COD cap^{-1} d^{-1}
	Daily flow, Q_d	35 m³ d^{-1}
	Hydraulic load ABR	1 m³ m^{-3} d^{-1}
	Average $v_{up,max}$	0.9 m h^{-1}
	Organic load ABR	0.8 kg COD m^{-3} d^{-1}
Operation	*Start of operation*	19.04.2010

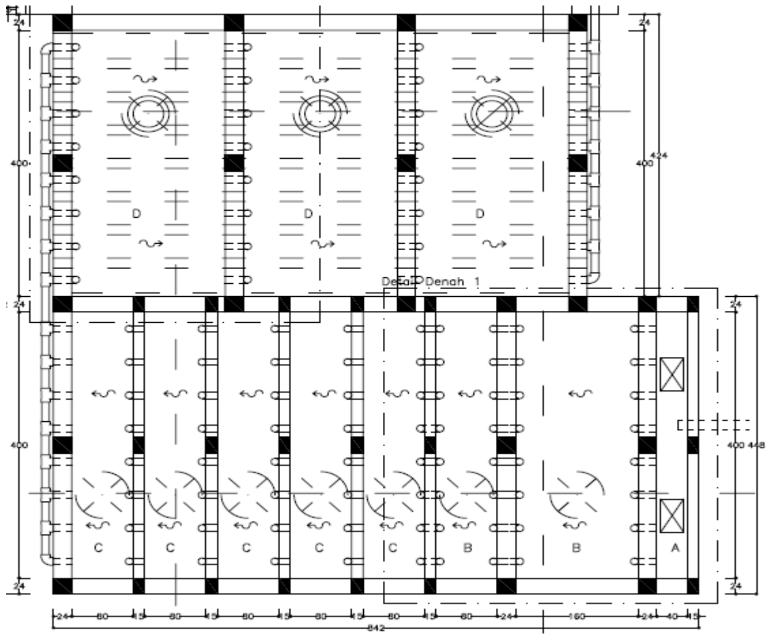

Figure 115: Schematic diagram (top-view) of the DEWATS-plants built in Santan/ Yogyakarta with feed flow distribution channel (A), settler (B), ABR (C), anaerobic filter (D), design water depth of the system: 2,000 mm

6.6.3. Field observations

Figure 116 a to e show the ABR chamber supernatants as photographed on August 26th, 2013. Signs of water level fluctuations are obvious in all chambers with scum and sludge marks on walls and the top parts of down flow pipes. Such high water levels have never been observed during times of peak flow on dry weather days. It is therefore concluded that storm water during tropical rains intrudes into the plant through the reticulation system.

Scum layers floating on the chamber supernatant was observed mainly in ABR 1 where it reached a thickness of about half a centimetre. Gas bubbles were mainly observed in ABR 3 and ABR 5.

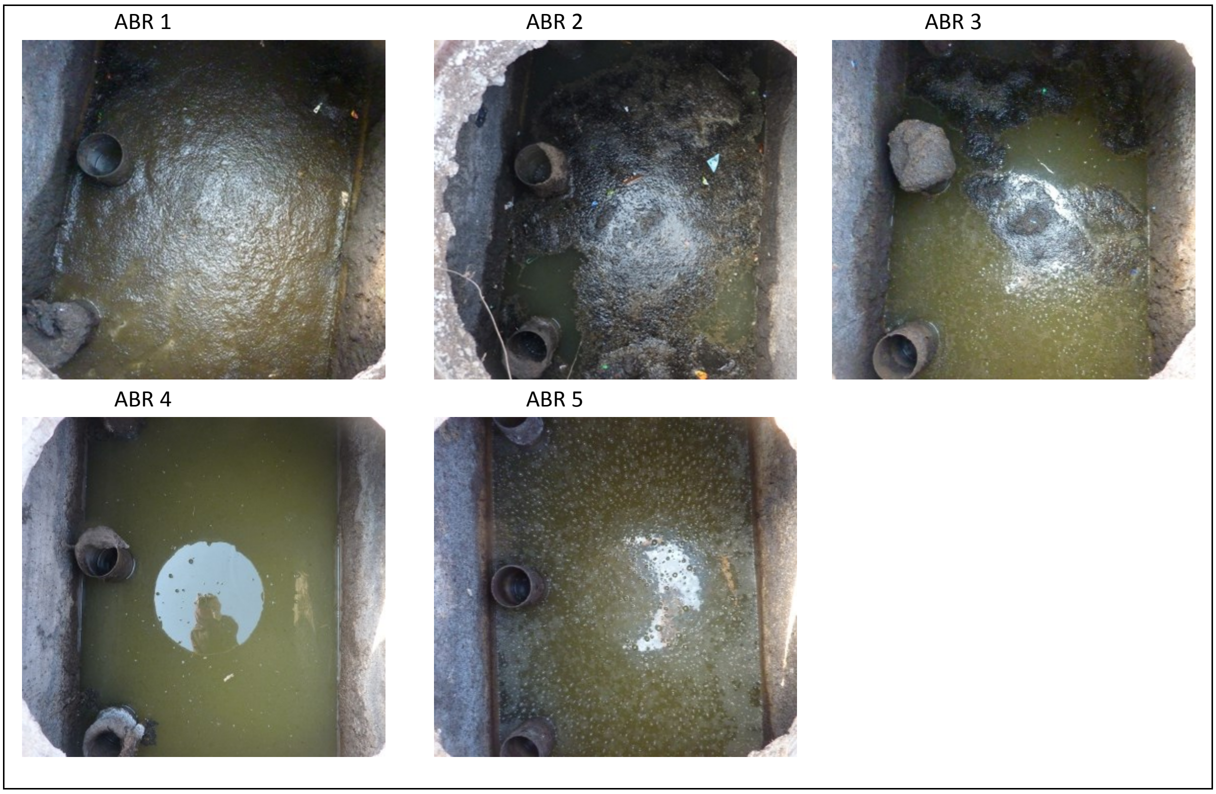

Figure 116 a, b, c, d and e: photographs taken through the ABR manholes showing chamber supernatant and down flow pipes, photographed on 26.08.2013

Figure 117 a and b are pictures of the AF 1 and 3 chamber supernatants as photographed on August 26th, 2013. Scum and small pieces of plastic waste can be seen on the desludging shaft, signs of past water level fluctuations. Small gas bubbles appear on the water surface of both chambers.

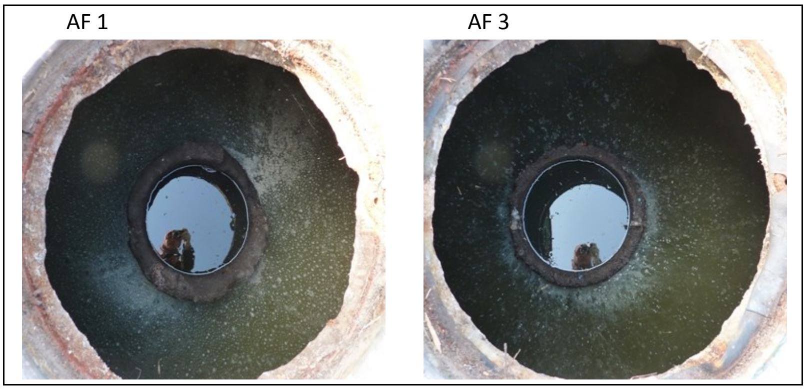

Figure 117 a and b: photographs taken through the AF manholes showing chamber supernatant and desludging shafts in the centre, AF 2 manhole cover could not be opened, photographed on 26.08.2013

6.6.4. Monitoring results: load estimation and exposure to flow surges

6.6.4.1. Users

The estimated number of connected users was recorded every year by the plant operator. As can be seen in Table 34 the number constantly increased until 2013 when it reached 467 users. Since the number however had not changed strongly from 2011 onwards, further data analysis will assume a constant value of 450 connected people.

Table 34: Number of connected users per year

Year	2010	2011	2012	2013
Total no of people connected per year	312	423	451	467

6.6.4.2. Flow

Figure 118 shows the average diurnal flow patterns as calculated from measurement data. The data was recorded manually every hour of the day from 7:00 to 21:00. The morning peak flow could therefore not completely be captured by the measurement. The cumulative flow between 21:00 and 7:00 was computed with the average hourly flow between these times. The average daily flow was 36.4 $m^3 d^{-1}$ which implies a daily per capita wastewater production of 81 l. The measurements were conducted during the dry season and were therefore not affected by rain. The wastewater production is assumed to remain constant over the complete period of investigation. The validity of this assumption is supported by the low variation of user connections as discussed in the section above.

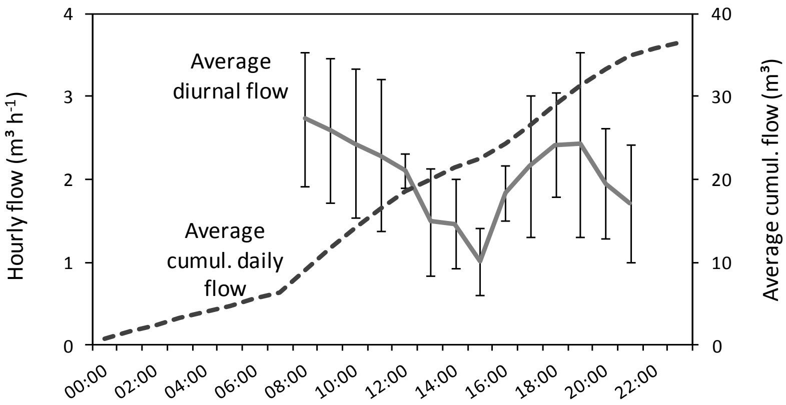

Figure 118: Average flows as measured in 2013, averages were calculated with data from 7 d (19.09.2013 to 25.09.2013), error-bars indicate the standard deviation of hourly flows over that period, no rain

6.6.5. Monitoring results: sludge composition, build-up and activity

6.6.5.1. Sludge heights

Figure 119 shows the sludge heights measured in settler 2 and all ABR chambers after the start of operation in April 2010. Most data points represent the average of triplicate measurements. ABR chambers were never desludged. AF sludge levels were only measured in 2013. The data can therefore not provide information about sludge accumulation in the previous years. The AF 2 manhole cover could not be opened which is why no data is available for that chamber.

The sludge heights do not follow a noticeable pattern across the chambers. Lowest sludge levels were generally found in ABR 1 and 3 whereas ABR 2, ABR 4 and ABR 5 generally contained a similar amount of sludge. The sludge level in ABR 5 suddenly increased strongly between March and August 2013.

The measured levels in the AF chambers remained approximately constant during 2013. By far most sludge accumulated in AF 1 and only a small amount in AF 3.

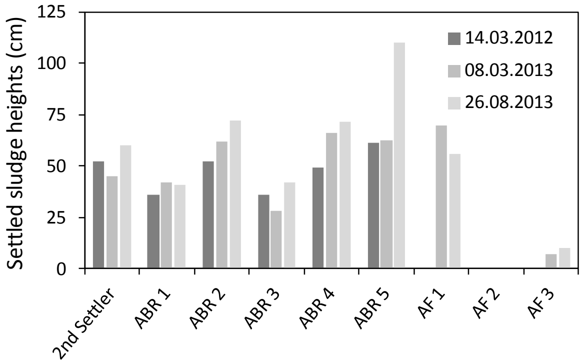

Figure 119: Measured settled sludge levels in Santan

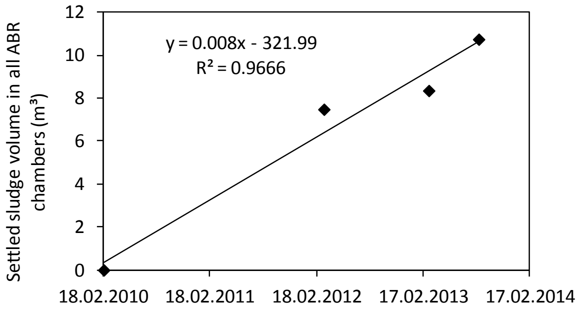

Figure 120: Total settled ABR sludge volume evolution in Santan

6.6.5.2. Sludge volume increase

Figure 120 shows the total ABR sludge accumulation over the time of operation. Linear regression of the data indicated an approximate sludge increase of 8 l d^{-1} or 2.9 m^3 y^{-1}. The two available data points indicate an approximately constant sludge volume increase over time.

The sludge heights measured inside AF chambers in August 2013 corresponds to an approximate sludge volume of 8 m^3 (estimating 30 cm sludge level inside AF 2) or an annual sludge volume increase of 2.3 m^3 y^{-1}.

The overall sludge volume increase is highest in the last chamber: 13%, 22%, 12%, 23% and 30% of the total ABR sludge build-up occurred in ABR 1, ABR 2, ABR 3, ABR 4, ABR 5 chamber respectively.

6.6.5.3. Sludge Total and Volatile Solids concentrations

The composition of settled sludge was measured for most reactor chambers in 2013 as part of the SMA investigations[14]. The results are shown in Figure 121. There is a general tendency of TS concentration being highest in the settler and the first ABR and constant or lower in the following reactors while the VS concentration remains approximately constant across the chambers.

The across ABR chambers average TS and VS concentrations of settled sludge were 94 g l^{-1} and 37 g l^{-1} respectively with a respective standard deviation of 23% and 15%.

[14] The field research team succeeded in opening the manhole cover of AF 2 only during one sludge sampling campaign.

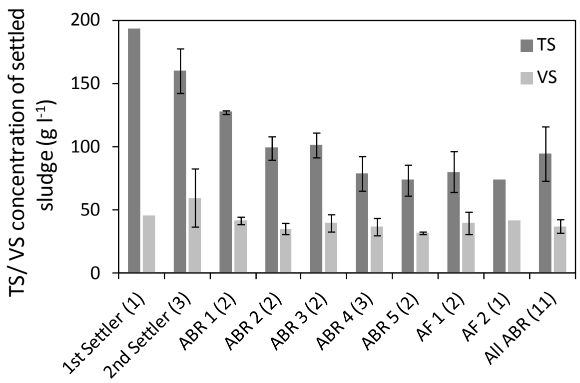

Figure 121: Settled sludge TS and VS concentration profiles, bars represent average values, "All ABR" bars represent averages of all ABR values, number of measurements is in brackets, error-bars indicate the standard deviation of multiple measurements

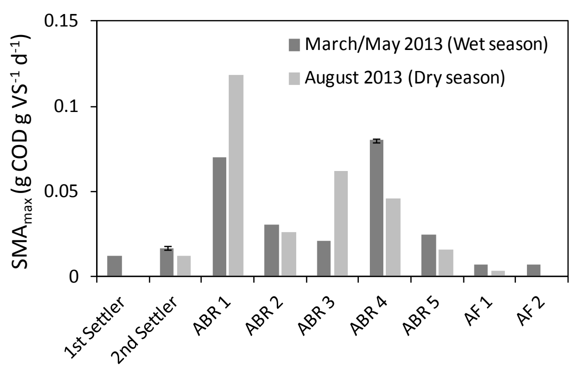

Figure 122: SMA_{max} of sludge sampled from different reactors in 2013, all sludge was processed within one week, error-bars indicate the standard deviation of duplicate measurements, all other values are derived from single measurements

6.6.5.4. Specific methanogenic activity (SMA) of sludge

Figure 122 shows the SMA_{max} values derived from sludge activity measurements performed on sludge sampled in each reactor compartment. AF 2 sludge could not be sampled during the dry season. All sludge was processed within one week after sampling. The error-bars indicate the standard deviation of duplicate sequential measurements of sludge from identical chambers during the wet season. All other values were derived from single measurements.

The measurements taken during the wet season indicated high activity in ABR 1. The SMA_{max} values of ABR 2 and ABR 3 gradually decreased and suddenly increased in ABR 4 before decreasing again in ABR 5. A similar pattern of high activity reactor chambers followed by compartments with gradually decreasing activity was observed during the wet season. Here again the activity peaked in the first and one rear chamber. The second activity peak however, previously in ABR 4, shifted one compartment towards the front of the reactor to ABR 3. The following chambers again showed gradually decreasing SMA_{max} values.

6.6.6. Monitoring results: alkalinity, pH, temperature and nutrient concentrations

The alkalinity of the well water used by one household in the community was measured once in 2013 yielding 180 mg $CaCO_3$ l^{-1}.

Figure 123 and Figure 124 present the general process parameters alkalinity and pH both of which were very constant across reactor chambers which indicates stable conditions for anaerobic treatment.

Measured wastewater temperature remained constantly between 28°C and 30°C averaging at 29°C.

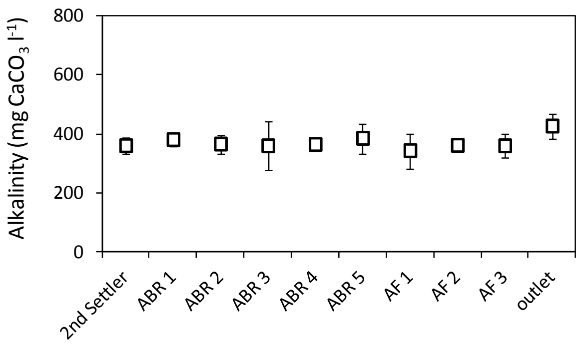

Figure 123: Average alkalinity concentration profile across reactor chambers as measured from 2012 to 2013, error-bars indicate standard deviation, 2 to 6 data points per sampling point

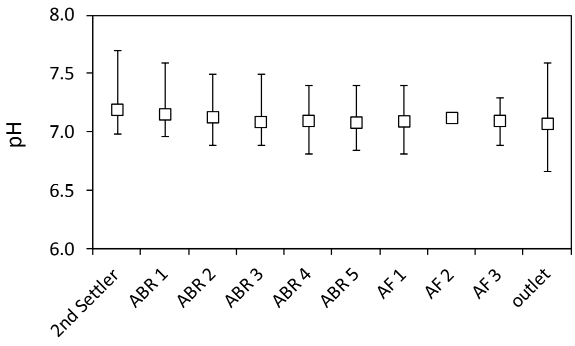

Figure 124: Maximum, median and minimum pH profiles across reactor chambers as measured from 2011 to 2013, 1 to 5 data points per sampling point

Nutrient investigations indicated 85 mg NH_4-N l^{-1} and 11.4 mg PO_4-P l^{-1} in the ABR feed and 64 mg NH_4-N l^{-1} and 12 mg PO_4-P l^{-1} in the effluent. All four values are averages from four sampling campaigns with duplicate measurements each.

6.6.7. Monitoring results: reactor COD concentrations and COD removal rates

No seasonal variation in the AF effluent was found in the available COD_t dataset (see Figure 125). Average settler effluent concentrations and ABR effluent concentrations however were lower during the wet season than during the dry season. Dilution of the incoming wastewater by rain appears unlikely since sampling was only performed on days without rain. On the other hand, groundwater infiltration caused by a higher ground water table during the wet season could be a plausible explanation for this phenomenon. However, the dataset is not large enough to make conclusive statistical testing. Time series data were therefore averaged over the complete period of investigation for further data interpretation.

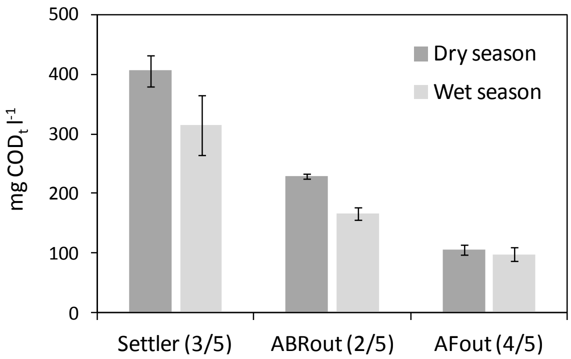

Figure 125: Average COD_t concentrations measured at settler effluent, ABR effluent and AF effluent, “dry season” is defined as the months May to September, “wet season” is defined as the months October to April, the numbers in brackets indicate the number of measurements made during dry and wet season respectively

The settler and ABR 5 COD_t and COD_p concentrations measured on September 2nd, 2013 were identified as outliers and removed from further analyses (see Figure 126).

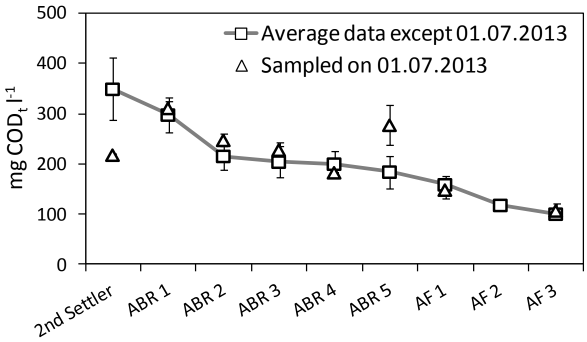

Figure 126: COD_t concentration profiles across ABR chambers and outlier values measured in July 2013

The feed ABR concentrations including their standard deviations were 349 ± 62 mg COD_t l^{-1}, 213 ± 64 mg COD_p l^{-1} and 140 ± 14 mg COD_s l^{-1}.

Figure 127 presents the average COD fractions measured in the supernatants of the different reactor compartments. Based on this the following hypotheses were formulated for statistical testing:

- Significant COD_p reduction in the ABR occurred only in ABR 2.
- Significant COD_p reduction in the AF only occurred in AF 1.
- Significant COD_s reduction did not occur throughout the treatment until AF 1.
- The COD_s reduction between AF 1 and AF 3 was significant.

An unpaired 2-sample t-test (significance level 5%) was used to test the first hypothesis which showed no significant reduction of COD_p between 2nd settler (M = 213, SD = 64) and ABR 1 (M = 168, SD = 41); t(15) = 1.72, P = 0.107. A second unpaired 2-sample t-test showed significant reduction of COD_p between ABR 1 (M = 168, SD = 41) and ABR 2 (M = 96, SD = 47); t(16) = 3.45, P = 0.003. A "one-way between subjects ANOVA" was then used to compare the average COD_p values measured in all chambers from ABR 2 to ABR 5. There was no significant difference at the $p < 0.05$ level [F(3, 32) = 0.9, Fcrit = 2.9].

Two unpaired 2-sample t-tests (significance level 5%) were used to test hypothesis 2 and showed a significant reduction of COD_p between ABR 5 and AF 1 (ABR 5: M = 99, SD = 53, AF 1: M = 49, SD = 33); t(16) = 2.12, P = 0.028 and no significant difference between AF 1 and AF 3 (AF 1: M = 49, SD = 33, AF 3: M = 24, SD = 12); t(16) = 2.12, P = 0.054.

Hypothesis 3 was tested with a "one-way between subjects ANOVA" which compared the average COD_s values measured in all chambers from 2nd settler to AF 1. There was no significant difference at the $p < 0.05$ level [$F(6, 56) = 1.54$, $F_{crit} = 2.27$].

Finally another unpaired 2-sample t-test (significance level 5%) confirmed the 4th hypothesis by showing significant reduction of all CODs measured in all chambers between 2nd settler and AF 1 (M = 122, SD = 32) and AF 3(M = 78, SD = 16) ; t(70) = 1.99, P = $9*10^{-5}$.

Statistics therefore support the four hypotheses: statistically significant COD_p reduction occurred only in ABR 2 and AF 1. No statistically significant COD_s reduction occurred from the settler until AF 1. The COD_s reduction between the upstream reactors and AF 3 however was statistically significant.

Figure 128 presents the average removal rates of the reactors. The average COD_t removal was 47% for the ABR and 45% for the AF. Most reduction in the ABR and AF was mediated through COD_p retention.

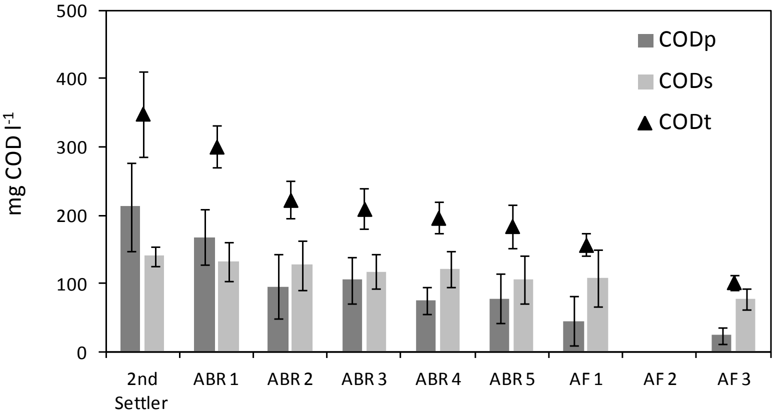

Figure 127: Average total, particulate and soluble COD profiles across reactor chambers as measured from 2011 to 2013, averages were calculated with 9 to 10 data points per sampling point, error-bars indicate standard deviation

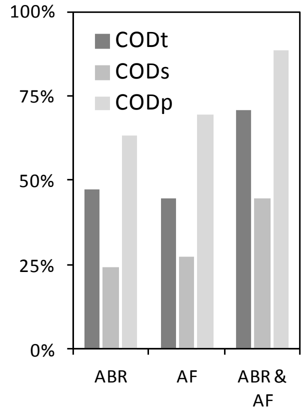

Figure 128: Average removal rates of COD fractions in ABR and AF

The COD effluent concentration measured in AF 3 was 102 (± 10) mg COD_t l^{-1} with a non biodegradable fraction of approximately 20 mg COD_s l^{-1} (see Appendix A3). Ten BOD_5 measurements of the AF3 effluent performed between 2011 and 2013 yielded an average effluent concentration of 83 (± 8) mg BOD_5 l^{-1}.

6.6.8. Discussion of case study data

6.6.8.1. Plant feed characteristics

A flow measurement campaign was performed in 2013 yielding an average daily flow of 36.4 m^3 d^{-1} and an average per capita flow of 81 l cap^{-1} d^{-1}. The number of people connected to the plant was approximately constant over the entire investigation period and was about 130% of the assumed design user number.

Feed concentration measurements were not performed at this plant. Based on the average measured settler effluent concentration, an assumed 50% COD_t reduction by the settler would imply an average plant feed concentration of about 700 mg COD_t l^{-1} and a per capita COD load of approximately 56 g COD d^{-1} (this issue is further discussed in Section 6.7.1.4).

A single well water measurement indicated a raw-water alkalinity of approximately 130 mg $CaCO_3$ l^{-1}.

6.6.8.2. Effect of flow surges on plant

Signs of storm water intrusion to the plant have been observed inside the reactor chambers. Sludge accumulated fastest in the rear ABR compartments certainly because of migration from the earlier compartments due to extreme flow peaks. Also large amounts of sludge were accumulating inside the first AF chamber, trapped below the AF growth media. Strong sludge washout from the AF into the receiving water body did not appear probable at the time this investigation ended, since only little sludge was found in the last AF chamber until then.

SMA measurements indicated significant differences between wet and dry seasons with methanogenic activity increasing in certain compartments and decreasing in others. It is therefore probable that the extreme peak flows occurring during strong rain-fall affect sludge activity.

The sludge accumulating inside the settlers and AF chambers had a very low methanogenic activity.

This implies that the conditions inside the settler were not supportive for acetoclastic methanogens. This implies further that either the active acetoclastic methanogens were comparably resilient to sludge washout from the ABR or that the conditions inside the AF chambers were not supportive for acetoclastic methanogens.

6.6.8.3. Estimated ABR and AF load and treatment

6.6.8.3.1 ABR and AF reactor load and performance

Figure 129 a and c put the observed OLRs and HRTs of ABR and AF in relation to design assumptions. The observed OLRs were calculated with the average measured Q and COD_t concentrations. The error-bars take into account a flow variation of 20% and the standard error of means of COD_p concentrations. The HRTs were computed with the same Q as the OLRs.

The observed OLRs, especially for the ABR, were far lower than those the reactors were designed for because the feeds were far less concentrated (see Figure 129 b and d) although 30% more people were connected than assumed during design. The observed hydraulic loads on the other hand were close to design values, being generally slightly higher. The average $v_{up,max}$ was 0.8 m h^{-1} and therefore also slightly below the design value of 0.9 m h^{-1}.

The measured alkalinity and pH values indicated good anaerobic treatment conditions throughout the reactors.

The reactor treatments of both ABR and AF combined were statistically significant for COD_p and COD_s however below design expectations. The ABR COD reduction was shown to be mediated only through COD_p retention. No significant ABR COD_s reduction occurred. The COD removal in the AF however took place through significant COD_p retention and COD_s reduction.

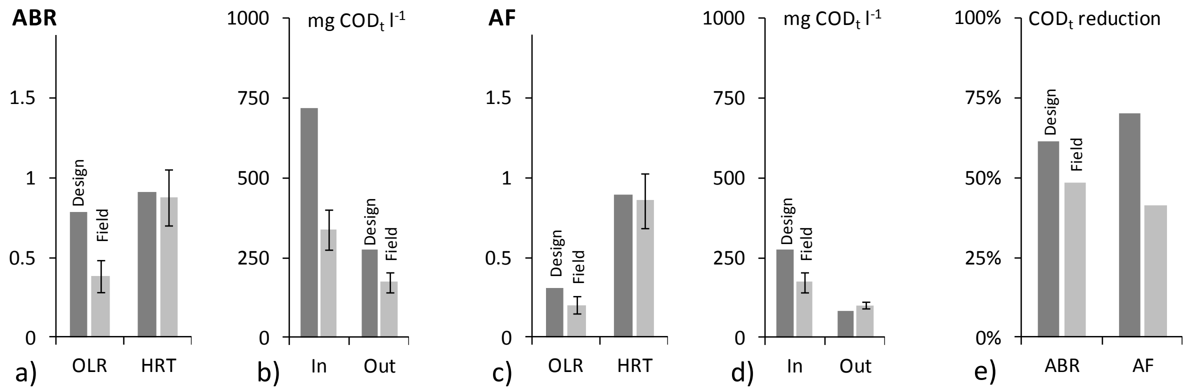

Figure 129 a, b, c, d and e: Loading and treatment parameters of ABR and AF reactors: OLR, HRT, feed and effluent COD concentrations and COD reduction rates, OLR error-bars indicate combination of standard error of mean of COD_t measurements and standard deviation of Q, all other error-bars indicate standard deviations

6.6.8.3.2 COD_p mass balance in ABR

The COD_p mass balance was calculated as described in Section 3.8.2:

The averages of measured values for Q, COD_p of ABR_{in} and ABR 5 and VS concentration of sludge led to a 32.7 (min = 16.7, max = 58.5) $m^3\ y^{-1}$ sludge increase assuming no anaerobic digestion. Minimum and maximum values take into account a feed flow variation of 20%, the standard error of means of COD_p concentrations and the standard deviation of sludge VS concentration data.

Linear regression of sludge volumes measured in the 6 ABR chambers led to a sludge build-up rate of 2.9 $m^3\ y^{-1}$. This is below the minimal rate calculated through mass balance. This discrepancy could not be explained through unnoticed sludge washout on days on which no wastewater sampling took place, since the sludge build-up inside the AF compartments was found to only be approximately 2.3 $m^3\ y^{-1}$.

The result therefore supports the hypothesis that anaerobic digestion took place inside the ABR and significantly reduced the volume of retained biodegradable COD_p. The further testing of this hypothesis with anaerobic digestion modelling is described in Chapter 7.

6.6.8.3.3 Compartment performance ABR and AF

Figure 130 compares the average COD_t measurement data with predictions given by the ABR and AF design calculation (curve "Design prediction"). The input values for these calculations were the average measured flows and feed concentrations.

The curve "Initial design" indicates the treatment assumed at the design stage of the plant with a significantly higher feed concentration and steeper COD reduction curve over the reactors, especially the ABRs.

The design calculation generally overestimates the COD removal inside the ABR and AF.

The ABR reduction curve "Design prediction" falls within the confidence limits of the field data in almost all chambers. Field measurement results in ABR 4 and ABR 5 were slightly higher than model prediction.

The only statistically significant COD removal in the whole ABR occurred in ABR 2 in the form of COD_p retention. This chamber incidentally had one of the higher sludge accumulation values. Measured COD_s reduction was not statistically significant in either of the ABR chambers although a certain methanogenic activity as well as gas bubbles were observed in several of them. It is therefore probable that COD_s removed by methanogenic activity was masked by COD_s produced through hydrolysis in the same chamber.

The AF reduction curve is similar to "Design prediction", leads however to higher effluent concentrations. AF effluent was found be largely biodegradable (see Section 6.6.7). It is therefore hypothesised that the reactor had not reached its full treatment potential at the time this study was carried out.

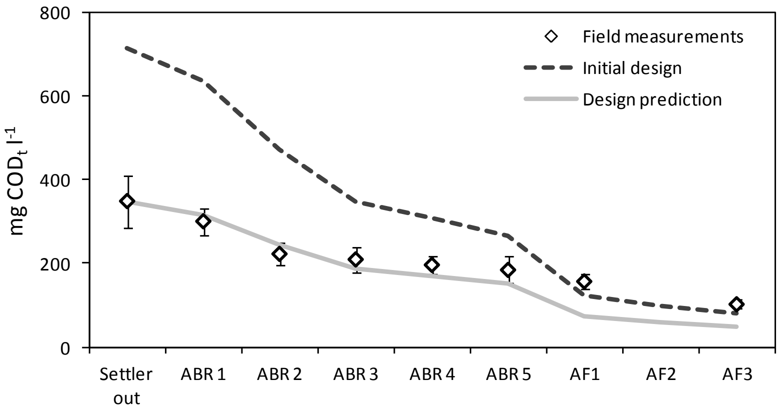

Figure 130: Measured average COD_t concentration profile, initial design prediction („Initial design“) and design prediction with input variables adjusted to measured field values („Design prediction“)

6.7. Discussion of case study data across plants

6.7.1. Plant-feed characteristics

6.7.1.1. Users

Figure 131 compiles the aforementioned plant user connection numbers and puts them into relation with design assumptions. Variations over the years and inaccuracies in the estimations can of course not be excluded. Operators and heads of communities were however generally very well informed about the dynamics and events in their communities and always showed interest in sharing information needed by the research team. Also all four communities were in residential areas with constant numbers of residing families where no large variation in population numbers is to be expected. Pipe systems were checked for major blockages and breakages by the research teams in India and Indonesia. Where broken house connections were found, the number of connected persons was adjusted accordingly.

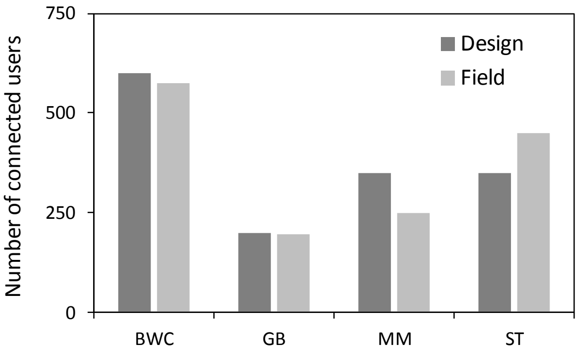

Figure 131: Number of connected users per plant

It is therefore assumed that potential variations not reflected by the data did not exceed a level that would significantly affect conclusions drawn further below.

The user numbers in BWC and GB were very close to design assumptions. The actual population size connected to the plants in MM and ST was about 70% and 130% of the values anticipated respectively at the design stage.

6.7.1.2. Flow

Figure 132 presents the average diurnal flows measured at all four sites. The averages presented for BWC were calculated with all data measured after the flow reduction in 2011 mentioned in Section 6.3.4.2. The MM curve was computed with the data from both measurement campaigns of which, for reasons unknown, the second yielded higher flows (see Section 6.5.4.2). This explains the comparably high standard deviation of measured daily flows presented in Figure 133. The GB and ST curves are based on data from one measurement campaign each.

Diurnal flows recorded in GB, MM and ST all featured morning and evening peaks typical for communal wastewater (Haestad et al., 2004). The community in BWC only received water in the morning which explains the non-typical diurnal fluctuation pattern measured there. The peak factors were 2.0, 1.8, 2.3 and 1.8 for BWC, GB, MM and ST respectively and therefore tend to confirm the assumed design peak factor of 2 (also see Section 4.3.1.2).

Average daily plant flows were similar to design assumptions in the case of GB, MM and ST (see Figure 133). The actual flow to BWC however was about 50% of what was projected during plant design. The resulting average per capita wastewater production was extremely low with only 30 l cap^{-1} d^{-1}. The highest average per capita value was measured in MM with 109 l cap^{-1} d^{-1}.

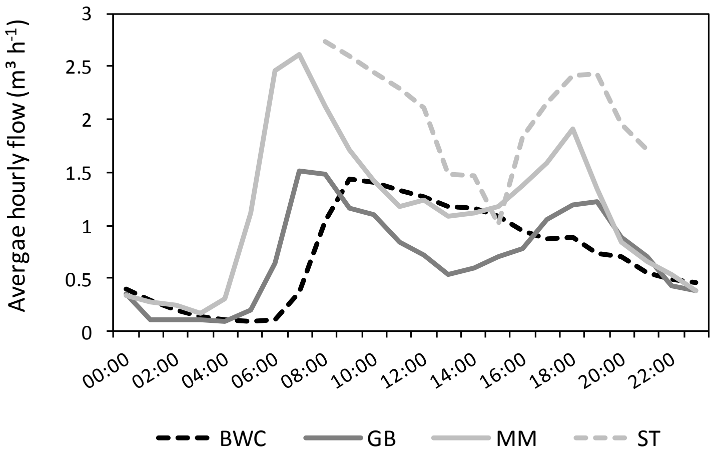

Figure 132: Average diurnal flows measured at the four case study sites

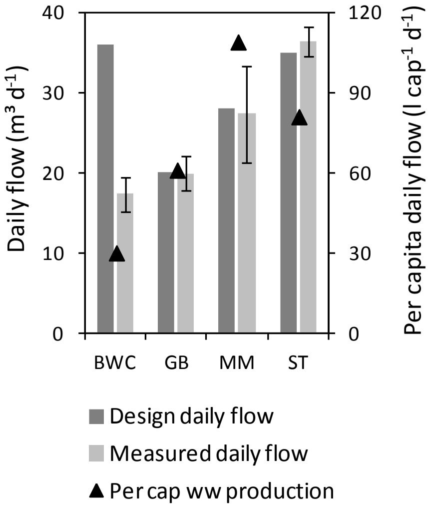

Figure 133: Daily flows at the four sites, error-bars indicate standard deviation across measurement days

6.7.1.3. Raw-water alkalinity

Well water alkalinity was measured six times, once, twice and once in BWC, GB, MM and ST respectively. The measurement results from the four sites were 468 (SD= 59), 132, 126 and 180 mg $CaCO_3$ l^{-1} respectively.

These values, although representing very few measurements, allow a valuable approximation of the average raw-water alkalinity in the four communities given that fresh water alkalinity from the same source depends primarily on geological factors and therefore varies little. All values measured in Yogyakarta were similar which appears plausible since most Indonesian households use low depth wells (WHO/UNICEF, 2013) therefore accessing similar aquifers.

Foxon (2009) hypothesised that the low ABR treatment observed during her pilot scale investigations was, amongst other reasons, caused by too low wastewater alkalinity (approximately 200 mg $CaCO_3$ l^{-1}). This led to a low reactor pH often below pH 6.5 therefore reducing biochemical conversion rates and microbial growth which in turn reduced the maximum up-flow velocity at which the system could be run. From a steady state modelling exercise, the author concluded that a feed alkalinity of 1,000 mg $CaCO_3$ l^{-1} would be needed to guarantee a process pH of at least 6.5.

Field data from the Indonesian case studies presented here however suggests that this does not apply to their situations. Raw-water alkalinity from bore wells was about 150 mg $CaCO_3$ l^{-1} at all sites and increased to 300 (MM) and 400 (GB and ST) mg $CaCO_3$ l^{-1} in the reactor feeds. The wastewater alkalinities throughout the systems remained approximately constant at these values. As opposed to Foxon's observations at slightly lower alkalinity, median pH values were approximately 7.0 in all plants and minimum values very rarely dropped below 6.5.

6.7.1.4. Estimated plant feed concentration, per capita COD production and pre-treatment efficiency

The average pre-treatment HRTs were approximately 73 h, 27 h, 10 h and 13 h for BWC, GB, MM and ST respectively. Even when taking into account that accumulated sludge reduces the active reactor volume and therefore also the HRT, these values are significantly larger than the 2 h HRT suggested by Sasse (1998).

Plant feed concentrations are difficult to measure and the available data was not considered to be sufficiently representative to extract information from. The plant feed concentrations can therefore only be estimated by either:

1. assuming the per capita COD production of the connected populations (of which the sizes and wastewater productions are known). The typical literature value is 120 g COD cap^{-1} d^{-1} (Tchobanoglous et al., 2003).
   or by
2. assuming a certain reduction rate of the first treatment step (of which the effluent concentration is known). The typical literature value is 50% COD reduction (see Section 2.1.2.2).

Table 35 summarizes the results of both these approaches. The results indicate in all cases that, when assuming correct population numbers, both assumptions cannot hold at the same time: the pre-treatment efficiency may have been higher and the per capita organic load may have been lower than the typical literature values.

Biogas production rates measured at the BGD in BWC support this since they indicate a probable COD removal of more than 76% and a per capita COD production of slightly above 60 g COD cap^{-1} d^{-1} (see Section 6.3.8.3). However, COD removal rates of simple settlers above 60% appear unrealistic when comparing with literature (see Section 2.1.2.2), especially when considering that the settlers in MM and ST had only about 12 h HRT. In these cases, lower per capita organic loads (such as suggested by Campos and vonSperling (1996)) become more plausible (see Section 2.5.4).

Extensive feed concentration measurement campaigns would be needed to further investigate this question.

Table 35: Summary of plant feed concentration assessments

	Calculation method 1			Calculation method 2		
Plant	Per capita organic load	Plant-feed concentration	Pre-treatment efficiency	Pre-treatment efficiency	Plant-feed concentration	Per capita organic load
	g COD cap^{-1} d^{-1}	mg COD l^{-1}	%	%	mg COD l^{-1}	g COD cap^{-1} d^{-1}
BWC	120	4,400	89%	50%	1,000	27
GB	120	1,400	71%	50%	800	71
MM	120	1,100	60%	50%	900	95
ST	120	1,500	76%	50%	700	56

6.7.1.5. Exposure to flow surges

Increased flow during rainfall was measured in BWC and MM (see Section 6.3.4.2 and Section 6.5.4.2). Signs of strong water level fluctuations inside the ABR chambers were observed in GB, MM and ST. Such high water levels had never been observed during times of peak flow on dry weather days.

In MM the head of community even reported that during extremely strong rain the system regularly completely filled with water to a point where water was pressed out of the closed manhole covers. This phenomenon did not occur in any of the other systems.

It is concluded that such water level fluctuations had to be caused by flows significantly greater than normal peak flows. It is argued that only water infiltration during storm water events can have led to this. Thus, all four plants were exposed to unknown but certainly considerable peak flows during wet seasons.

6.7.2. Sludge characteristics

6.7.2.1. Sludge build-up

Table 36 summarizes the sludge build-up rates observed in the four plants. The largest ABR per capita rate was observed in GB, the lowest in MM. Sasse (1998) cites Garg (unknown year) with a build-up rate of 30 l sludge cap^{-1} y^{-1} in septic tanks. Further sludge accumulation rates for septic tanks are compiled in Table 18 in Section 4.3.4. All are significantly higher than the rates observed in the ABRs, possibly because of more efficient stabilisation of organic material in the ABR and therefore less sludge build-up. Also, ABR feed is pretreated as opposed to septic tanks of which the raw wastewater certainly contains higher amounts of nonbiodegradable particulates.

Foxon (2009) normalized the sludge accumulation rates observed during her ABR study using OLR. The amount of accumulated sludge per kg COD applied was approximately 2.1 l (kg COD applied)$^{-1}$ during a loading regime described as supportive to good treatment and anaerobic digestion. Sludge build-up observed in this study was significantly lower (see Table 36).

Table 36: ABR and per capita sludge build-up rates at the four sites

		BWC	GB	MM	ST
Yearly sludge build-up	m^3 y^{-1}	0.8	1.7	0.8	2.9
Per capita sludge build-up	l cap^{-1} y^{-1}	4.2	8.5	3.1	6.5
Normalized sludge build-up	l sludge (kg COD applied)$^{-1}$	0.38	0.74	0.19	0.64

6.7.2.2. Sludge build-up distribution

Sludge build-up in GB, MM and ST occurred predominantly in the last reactor chambers (see Figure 134). This trend was observed the more the longer the plant had been operating: over time the highest observed sludge level in all 2 ABRs shifted towards the rear compartments.

In BWC during Phase I on the other hand most build-up occurred in the first compartment and gradually decreased towards the rear of the reactor (see Figure 134). The closer a chamber was to the feed, the higher its sludge level was during all 2 y of Phase I.

Foxon (2009) reports that right after start-up of a pilot ABR in South Africa sludge accumulated most in the first compartments. As the operation progressed, accumulation was also observed in later compartments so that, similar to the Indonesian case studies, sludge levels there eventually exceeded the levels of the first chambers.

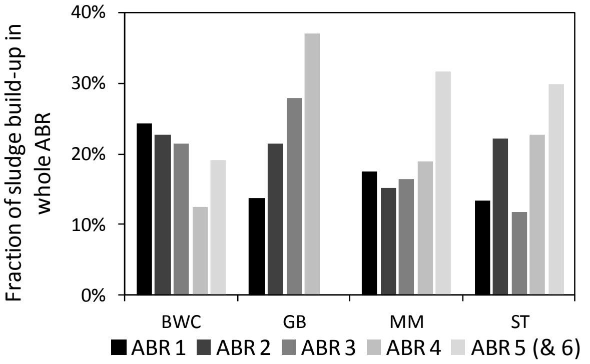

Figure 134: Fractions of ABR sludge build-up observed in the different reactor chambers

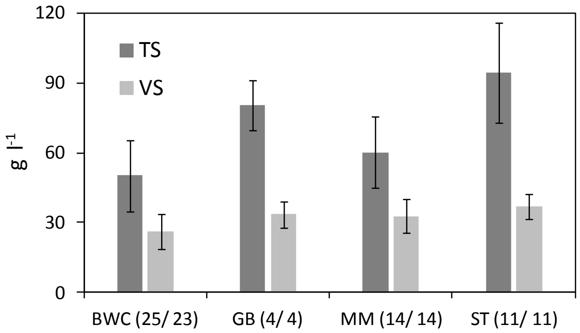

Figure 135: Average ABR settled sludge TS and VS concentrations at the four sites, error-bars indicate standard deviations of measurements across chambers

6.7.2.3. Sludge Total and Volatile Solids concentrations

Settled ABR sludge TS and VS determinations are expected to have considerable uncertainties associated to them since they involve a number of measurements that are prone to error: sludge level measurements, sludge sample volume measurements and TS and VS concentration measurements.

Nevertheless the results provide a coherent picture across all four plants. They all indicate higher sludge (and especially TS) concentrations in the first and approximately constant concentrations in all following reactor chambers (see Figure 136 and Figure 137):

- In all four plants higher TS concentrations were found in the settlers than in the ABR chambers.
- In two plants (BWC, MM, ST) the highest ABR-TS concentrations were measured in the first ABR chambers.
- In the case of BWC and MM, the highest VS concentrations were observed in the settlers and first ABR chamber. In the other plants the VS concentrations were approximately constant throughout the ABRs.

Average TS concentrations of ABR sludge varied across the systems from about 50 g TS l^{-1} to 95 g TS l^{-1}. The sludges from all four ABRs had a similar average VS concentration of about 30 g VS l^{-1} (see Figure 135). Mtembu (2005) observed settled sludge TS concentrations of 12 g TS l^{-1} to 34 g TS l^{-1} on a pilot ABR in South Africa. Foxon (2009) reported an estimated VS to TS ratio of 0.57 on that same plant which results in a settled sludge concentration of 7 g VS l^{-1} to 19 g VS l^{-1}. This is significantly lower than values observed in the four case studies, which apparently featured a much more dense sludge. Koottatep (2014) reported TS concentrations of thickened bottom sludge in onsite sanitation systems treating raw sewage of 40 to 220 g TS l^{-1}. VS content of this sludge was 60% to 70%. The significantly lower VS content of the sludge observed in the four case studies (52%, 42%, 55% and 39% in BWC, GB, MM and ST respectively) may be due to better stabilisation.

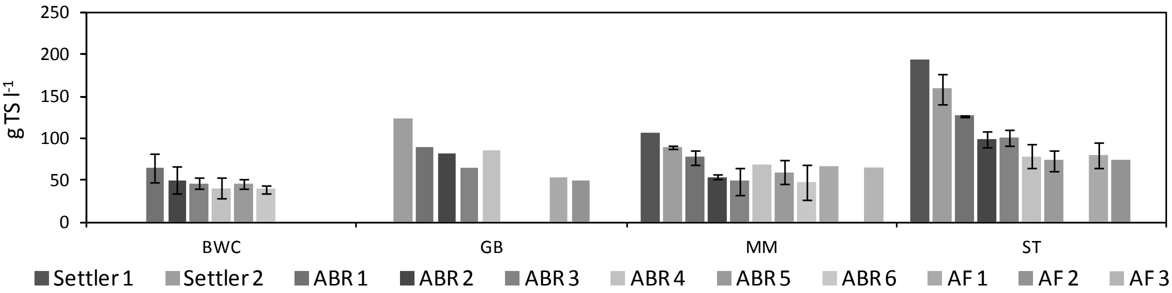

Figure 136: Settled sludge TS concentrations at the four case study sites

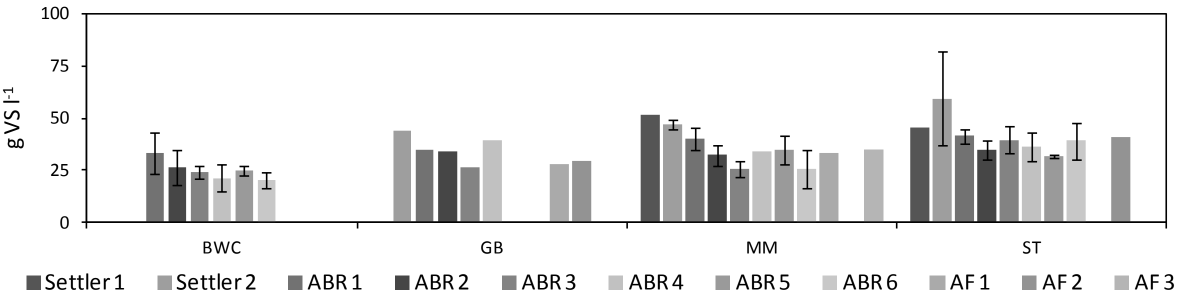

Figure 137: Settled sludge VS concentrations at the four case study sites

6.7.2.4. Methanogenic activity

Figure 138 and Figure 139 present the SMA_{max} values measured during the wet and the dry season at the four case study plants and compare them to the benchmark value proposed by Pietruschka (2013) for active anaerobic sludge.

Most methanogenic activity was found in MM where it even reached the benchmark value proposed by Pietruschka (2013). Least methanogenic activity was measured in GB. Also there does not appear to be a correlation between the amount of bubbles found on the chamber supernatant and the corresponding SMA_{max} value although more data would be needed to confirm this. SMA measurements done in BWC cannot be compared to the other plants since the sludge height measurements indicated sludge washout from the digester into the ABR. Such washout would certainly have included active sludge. The measured sludge activity in the ABR can therefore not be solely attributed to ABR operation.

The existing SMA dataset is not based on a very large number of measurements which was at first regarded as limiting its general informative value. However a number of recurring observations can be made across the plants. It is argued that this coherence justifies a certain confidence in the data although some of the following interpretations will certainly have to be verified by further investigations.

First of all, system components with low methanogenic activity are strikingly similar across case studies:

In both plants built without digester (MM and ST) settler sludge yielded very low SMA_{max} values indicating low fractions of active methanogenic MOs. Similarly low SMA_{max} values were observed with all sludges sampled from rear ABR chambers and all AF chambers.

On the other hand there is a general tendency of the highest SMA_{max} always being in one of the first three ABR chambers, especially during the dry season.

Sludge activity is generally found to increase after a period of approximately 40 d without rain influence. Although this is not the case for all reactor chambers, the occurrences of measured sludge activity increase outweigh the cases in which the sludge activity did not increase.

It is acknowledged that the unknown MO fraction of two sludges with similar VS concentrations makes it impossible to differentiate between non existing methanogens and existing but inactive methanogens (see Section 3.4.6). An observed difference in SMA_{max} values therefore only allows making a qualitative comparison on the average acetoclastic methanogenic activity, not on the amount of methanogens per se.

It is hypothesised that the two main causes for the above mentioned variations of acetoclastic methanogenic activity in ABR chambers are: substrate availability and forced migration through flow surges.

It is striking that SMA_{max} values always indicate alternating activity strength across chambers in MM, ST and BWC: chambers with high activity sludge are always followed by one or two chambers with significantly lower activity in all measurement campaigns.

Such a pattern could be explained by the varying availability of easily biodegradable substrate: high substrate availability in one chamber would lead to an activity increase of the MO-consortia feeding on this substrate (in this case the acetoclastic methanogens). The resulting high substrate uptake would lead to the reduction of available substrate for the MO population in the following chambers, therefore reducing their activity. This reduced activity of consuming organisms allows the build-up of easily biodegradable substrate which is then available for the MO in the following compartment and the process starts anew. This would imply in all four plants hydrolysis being the rate-limiting step since substrate availability for methanogens appears to be too low to sustain a high biomethane activity throughout all chambers.

Low substrate availability also appears to be a plausible explanation for the low activities of settler and AF sludge. It is possible that incomplete hydrolysis processes inside the settler may not have enabled a large methanogenic community to develop. Most released substrate is then consumed throughout the ABR, starving the populations in the sludge at the bottom of the AF chambers.

Sludge level measurements in MM, GB and ST showed the occurrence of sludge migration from the first to the last reactor chambers. This migration however does not appear to have affected the average methanogenic activities proportionally. If that had been the case most biomethane activity would have been found in the rear of the ABR and in the AF.

The fact that the front ABR chambers contained the sludges with the highest SMA_{max} values, even at the end of the wet season, leads to the conclusion that active acetoclastic methanogens succeeded in establishing a stable community even under high hydraulic loading. Acetoclastic methanogens appear therefore to be surprisingly resilient to washout. Their marked activity increase after 40 d to 60 d without rain (especially in the front chambers) indicated that they certainly were impeded during the wet season. Whether the storm water primarily affected the sludge because of reduced substrate availability or through washout of methanogens cannot be determined with the available information. The data however tends to point towards the reduced substrate availability as being the cause.

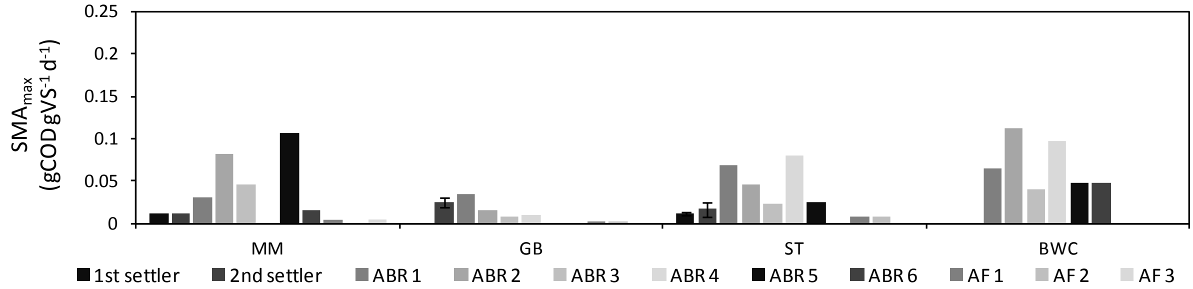

Figure 138: SMA_{max} values measured across reactor chambers of the four case study plants during wet-season

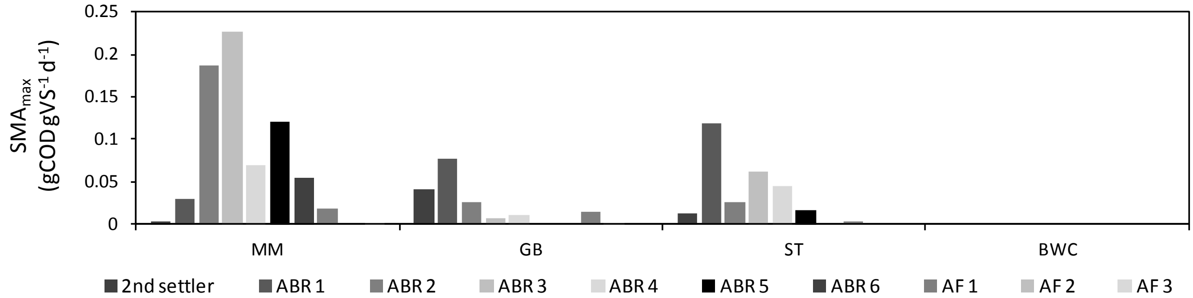

Figure 139: SMA_{max} values measured across reactor chambers of three case study plants during dry-season

6.7.3. Effect of flow surges on the systems

All Indonesian case study plants (GB, MM, ST) were affected by storm water. Strong water level fluctuations inside the ABR, sludge migration to the rear ABR and AF compartments and reduced methanogenic activity have been observed in all systems and can be traced back to extreme hydraulic surges during tropical rains. These alterations are bound to have had an impact on the system treatments.

Whether the flow surges mainly affected the methanogenic activity because of wastewater dilution (and a, therefore, low mass transfer driving force between substrate and biomass) or methanogens washout cannot firmly be concluded from available information. Results of SMA investigations however point towards the first reason. During any season (wet and dry) most activity was found in the first chambers and there was little accumulation of active sludge towards the rear of the reactors. Also all reactors except GB showed patterns of varying activity across chambers in both the wet and dry season. This appears difficult to explain through sludge migration since migration would occur more evenly across chambers. It was hypothesised that varying availability of easily biodegradable substrate are the main reason for this phenomenon.

BWC did not show signs of water level fluctuations or severe sludge washout although rain certainly infiltrated the system. It is therefore possible that BWC treatment processes were less affected by storm-water than the Indonesian plants. Because of the different climates, rain events are far less intense in Bangalore than in Yogyakarta. BWC operated with comparably low hydraulic load which might have increased its capacity to handle flow surges. The effect of rainwater on BWC's sludge activity cannot be assessed with the available data: SMA tests were only performed during the wet season and were probably influenced by sludge migrated from the digester to the ABR.

6.7.4. ABR load estimations

Figure 140, Figure 141 and Figure 142 summarize the values across plants for the hydraulic load parameters HRT, $v_{up\ mean}$ and $v_{up\ max}$. HRTs are represented as per chamber values in order to simplify the comparison across systems because the four ABRs have varying chamber numbers.

MM was designed with the highest per chamber HLR, followed by ST. BWC and GB design rates are approximately similar. All parameter field values are close to design expectations in the case of GB, MM and ST. BWC however is loaded with less than half the design hydraulic load.

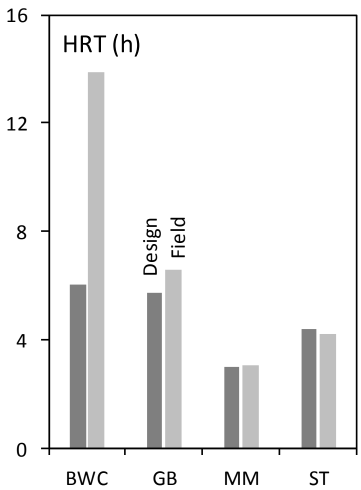

Figure 140: HRTs of single ABR chambers of the four plants

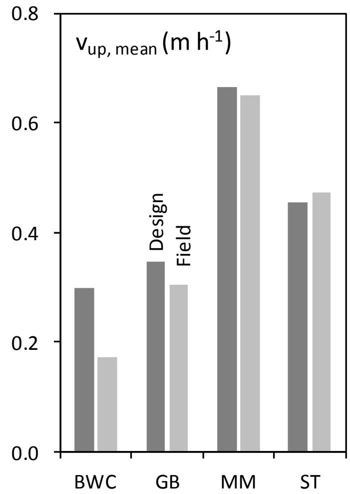

Figure 141: $v_{up,mean}$ values of the four ABRs

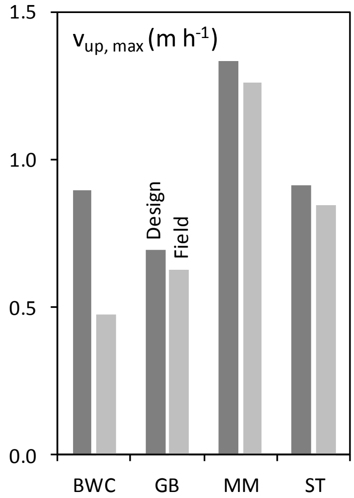

Figure 142: $v_{up,max}$ values of the four ABRs

Figure 143 summarizes the ABR_{in} COD_t concentrations as measured at the four sites and compares them with design assumptions.

MM was designed with by far the highest feed concentration of all four reactors, BWC having the lowest. Similar feed concentrations were assumed at design stage for GB and ST.

Measured feed concentrations do not strongly differ from one site to another, especially in the case of GB, MM and ST (see Figure 143) where they were found to be about 350 mg COD_t l^{-1} to 450 mg COD_t l^{-1}. The ABR feed sampled at BWC was in average slightly higher concentrated (about 500 mg COD_t l^{-1}). This wastewater however contained a considerable fraction of nonbiodegradable COD, approximately 100 mg COD_s l^{-1}. Nonbiodegradable COD concentrations at the other plants were only about 20 mg COD_s l^{-1}. The general biodegradable organics feed concentration can therefore be regarded as approximately similar in all cases. Hence it slightly exceeds the initial design assumption in BWC but is far lower than assumed in GB, MM and ST. As discussed in Section 6.7.1.4 the reason for the latter could either be a higher pre-treatment efficiency or a generally lower per capita COD production than assumed.

The resulting OLRs are presented in Figure 144 and all take the respective nonbiodegradable COD fractions into account. The error-bars represent the standard error of COD_t measurements and a flow variation of 20%. BWC had by far the least and MM the organically strongest loaded ABR reactors. However all plants, especially GB, MM and ST, were loaded way below design expectations.

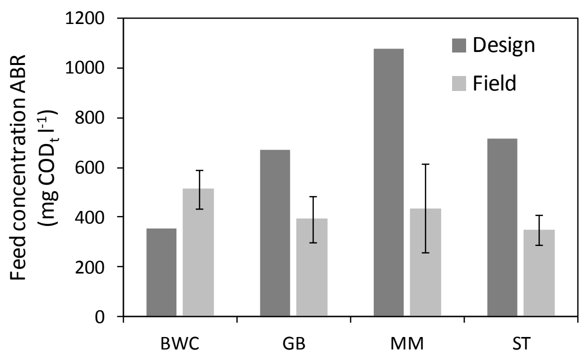

Figure 143: Average ABR feed concentrations of the four plants, error-bars indicate the standard deviations

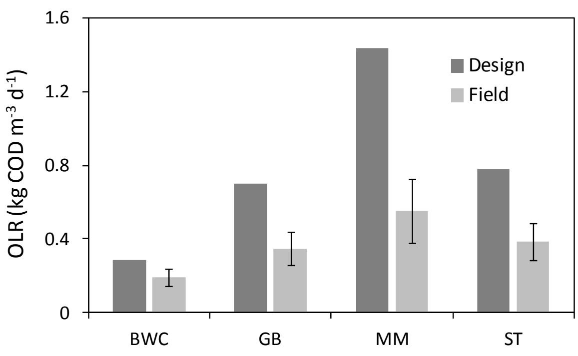

Figure 144: Average ABR OLRs of the four plants, error-bars indicate the standard error of COD$_t$ concentration measurements and a 20% variation of Q

The comparison across plants is complicated by the varying number of reactor chambers. Figure 145 therefore presents the OLRs per reactor chambers. Also, in an effort to better represent the organic fraction relevant to methanogens only the measured biodegradable COD$_s$ concentrations were used to compute the OLR.

The figure shows that MM across all chambers had by far the highest soluble OLR of all plants. BWC, GB and ST ABR chambers were approximately similar in that respect. The soluble OLR cannot be compared to design assumptions since these do not differentiate between particulate and soluble compounds.

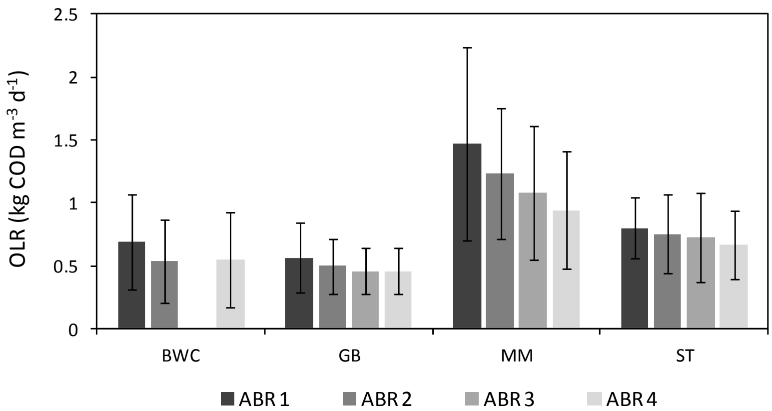

Figure 145: Biodegradable soluble OLRs of single ABR chambers of the four plants, error-bars indicate the standard deviation of COD$_s$ concentration measurements and a 20% variation of Q

6.7.5. ABR anaerobic activity

ABRs treat communal wastewater essentially by retaining and digesting particulate organics and their soluble decomposition products. The two main treatment-influencing parameters are therefore solid retention and anaerobic sludge activity. The assessment methods of system activity normally used for anaerobic systems, methane production measurements and COD mass balances, are very difficult to implement and often not feasible in the case of BORDA DEWATS (Reynaud et al., 2011). ABRs are for instance not designed with biogas catchment which makes in situ measurements impossible. Also, small scale communal wastewater treatment facilities are intrinsically exposed to large variations of both organic concentration and hydraulic loading. Accurate stream load assessments are therefore very resource intensive and difficult to implement especially in the context of developing countries.

The COD fraction COD_s is a lump indicator including all dissolved short chain organic compounds whose further reduction to methane ultimately leads to the removal of COD from the wastewater. COD_s is comparably easy to measure however difficult to interpret in terms of system activity. A stable COD_s concentration could for instance indicate inactivity of the system. But it could also stand for an active anaerobic environment in which the depleted soluble organics through methane production are compensated by the hydrolysis of particulate organics.

SMA investigations give some qualitative information on the sludge activity and have shown that the ABR sludge in all case studies had a certain methanogenic activity. However these tests have only recently been introduced to the research program presented in this thesis. They are not yet sufficiently well understood in order to draw strong conclusions based only on them and in any case can account only for the last year of plant history since this is when they have been applied.

Sludge build-up on the other hand has the huge advantage of representing the complete cumulated plant loading history as opposed to point in time stream concentration measurements or sludge activity measurements. An active anaerobic system would feature much less sludge production than a system in which organic material simply accumulates without getting digested. The observed sludge accumulation rates are therefore compared to simple particulate organics mass balances based on the available flow data, sludge VS concentration data and COD_p data (for details on the methodology see Section 3.8.2).

Figure 146 summarizes the outcomes of this exercise. The four observed sludge accumulation rates are all below the confidence interval of the mass balances, in the case of MM strongly so.

Accumulation rates could be underestimated because of sludge washout. The sludge accumulation observed inside the AF chambers however indicates that, although sludge washout from the ABR did occur, it was comparably little and cannot account for the difference observed between the measured and the theoretical build-up.

It is further conceivable that the mass balance results yield too high results due to exaggerated feed- or too low effluent concentrations. All COD measurements were performed on days not influenced by rain water events. The existing ABR feed concentrations therefore rather underestimate the real average since they do not take possible sludge washout from the settler into account. The extent to which the ABR effluent data underestimates the real average value can be approximated through the sludge build-up in the AF chambers as explained above. It does not appear to be considerably high. Future investigations however are needed to confirm this point.

The results therefore support the hypothesis that the case study systems were active and significantly degraded the accumulated solids inside the ABR. The further testing of this hypothesis with anaerobic digestion modelling is described in Chapter 7.

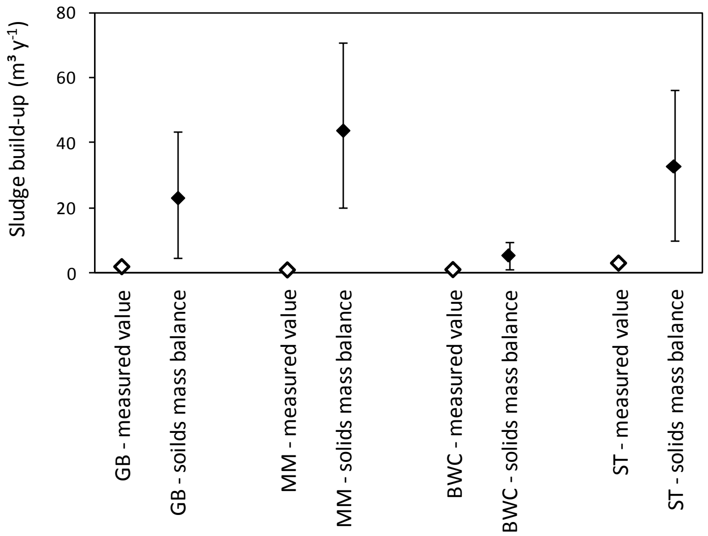

Figure 146: Average sludge build-up rates observed in reactors and estimated through mass balance of particulate organics, error-bars represent confidence intervals taking a feed flow variation of 20%, the standard error of means of COD_p concentrations and the standard deviation of sludge VS concentration data into account

6.7.6. ABR COD removal rates

The pH values measured at all plants indicated good and stable anaerobic conditions with median values at about pH 7 and the minimum values never sinking below 6.5. The wastewater temperature at the Indonesian plants GB, MM and ST always remained between 27°C and 30°C and did not vary noticeably across the seasons. In BWC, wastewater temperature fluctuations between the dry and wet season were however significant and have to be considered during data interpretation.

Figure 147 compiles the COD_t concentration profiles across the ABR chambers of the four plants. All average values were adjusted by subtracting the nonbiodegradable COD_s. The latter was significant in the case of BWC with approximately 100 mg COD_s l^{-1}. Investigations at all other systems yielded much lower values of approximately 10 mg COD_s l^{-1}. All resulting feed concentrations in BWC (Phase I), GB, MM and ST are similar. The average of the measured ABR_{in} concentrations in BWC Phase II is significantly higher than in Phase I. It was however shown that this difference is mainly due to both datasets having been differently influenced by seasonal fluctuations. The actual dry weather feed was therefore probably more similar in both phases than suggested by Figure 147. Effluent concentrations of the ABRs were 236 (± 42), 162 (± 40), 236 (± 61), 135 (± 33) and 174 (± 32) mg COD_t l^{-1} for BWC Phase I, BWC Phase II, GB, MM and ST respectively[15]. All values exclude the plant specific non biodegradable CODs concentration.

Figure 148 summarizes the average dry weather treatment efficiencies of the four different ABRs in terms of COD_t removal and compares them to design assumptions. Also the removal rates were calculated with COD_t concentrations reduced by the estimated nonbiodegradable COD_s concentrations.

All ABRs were designed for treatment efficiencies of about 60% with the exception of MM which was planned to remove about 82% of the incoming COD_t load. In the case of BWC (Phase I), GB and ST, the

[15] The numbers in brackets indicate standard deviations.

average actual treatment was however only about 40%, way below design expectations. Field data from BWC (Phase II) and MM suggest 68% COD_t removal, although the value for BWC may have to be corrected slightly upwards since the average feed concentration probably underestimated the real value. It was however shown that a significant treatment improvement occurred from Phase I to Phase II without being able to correctly quantify it with the current dataset. These treatment efficiencies are surprisingly low and certainly lower than the 70% to 90% reported in literature (see Section 2.3.7).

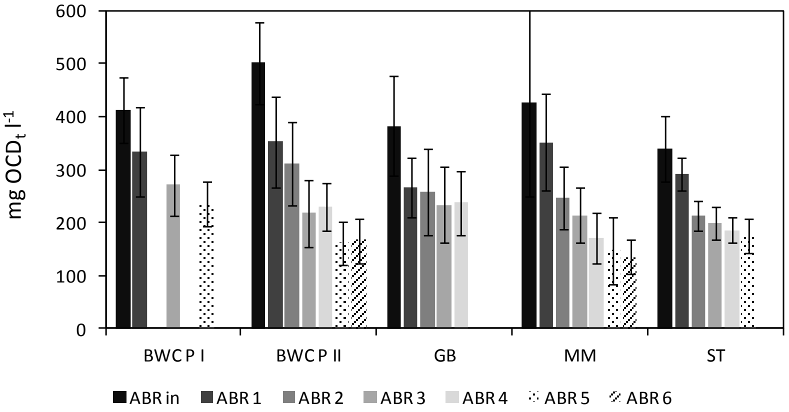

Figure 147: Average COD_t concentration profiles of the four plants and two phases in BWC, error-bars indicate standard deviations, all values exclude the plant specific nonbiodegradable COD_s concentration

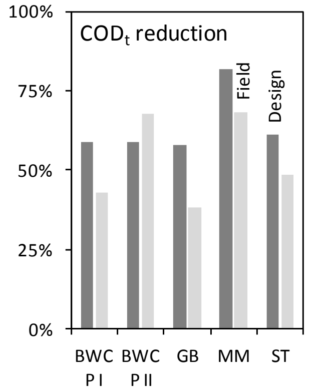

Figure 148: Average COD_t reduction rates of the four plants

Table 37 summarises the chambers until which significant COD_p and COD_s reduction occurred in the four case study ABRs. Statistically significant COD_p reduction was found to occur in all ABRs however generally only in the first three chambers. The reduction in GB only took place in the first and in Santan only in the first two chambers. Significant COD_s reduction was only statistically proven in BWC and MM, with MM having the most number of chambers (three) taking part in this. The ABRs in GB and ST did not significantly reduce the COD_s concentration at all.

MM therefore appears to be the plant in which the most ABR chambers contribute to significant organic wastewater concentration reduction followed by BWC (Phase II), ST and finally GB. Literature states that treatment is generally observed in the first three chambers, similar to the observations made in MM.

Table 37: Furthermost downstream ABR chambers significantly contributing to COD_p and COD_s reduction

	BWC PI	BWC PII	Gambiran	Minomartani	Santan
COD_p	ABR 3	ABR 3	ABR 1	ABR 3	ABR 2
COD_s	ABR 1*	ABR 2	None	ABR 3	None

*significant but small reduction in the rear chambers

6.7.7. Effect of dry weather loading rates on case study ABR treatment

Table 38 summarizes treatment indicator values of the four case studies and Phases I and II in BWC. Table columns from left to right are arranged depending on plant treatment quality. The treatment indicators are *average COD_t, COD_p and COD_s removal* in relative terms as well as in absolute numbers normalized over the reactor volume, *furthermost downstream reactor chamber partaking in significant COD_p and COD_s removal*, *average SMA_{max}* and normalized *sludge accumulation*. Average COD_t and COD_s removal rates were adjusted by subtracting the nonbiodegradable COD_s concentrations from the averages. The "Average SMA_{max}" represents the mean of all SMA_{max} values measured during the dry season in all ABR compartments of one plant. The values for BWC were not included because they were probably significantly influenced by the washout of active sludge from the digester. The presented sludge accumulation values are normalized over the organic load.

MM and BWC (Phase II) feature by far the best COD_t removal efficiencies of all ABRs. These are however still significantly below rates reported from investigations run under ideal laboratory scale (about 90%) and field conditions (about 80%) (see Section 2.3.7). All other case studies had comparably poor efficiencies below 50% COD_t removal.

Table 38: Summary of average treatment indicator values

		MM	BWC (Phase II)	ST	BWC (Phase I)	GB
Average% COD_t removal ABR		68%	68%	49%	43%	38%
Average% COD_p removal ABR		80%	74%	63%	56%	48%
Average% COD_s removal ABR		54%	64%	26%	35%	23%
Average COD_p-removal	g COD_p d^{-1} m^{-3}	280		154	36	95
Average COD_s-removal	g COD_s d^{-1} m^{-3}	125		40	15	32
Significant COD_p removal until		ABR 3	ABR 3	ABR 2	ABR 3	ABR 1
Significant COD_s removal until		ABR 3	ABR 2	None	ABR 1*	None
Average SMA_{max}**	g COD g VS^{-1} d^{-1}	0.11		0.05		0.03
Sludge accumulation	l kg COD_{appl}^{-1}	0.19		0.64	0.38	0.74

* significant but small reduction in the rear chambers, ** average of values from first five chambers

COD_p retention represented in all case studies the COD fractions with the highest reduction. As a result, effluent COD_p concentrations in MM and BWC (Phase II) were approximately 50 mg COD_p l^{-1} and 80 to 100 mg COD_p l^{-1} in BWC (Phase I), GB and ST. The average COD_p reduction in absolute numbers was highest in MM. The BWC COD_p retention improvement from Phase I to II cannot be explained with certainty. It is however interesting to notice that it coincided with a significant sludge level increase inside the reactor. Higher sludge levels could have enhanced the filtering characteristics of the sludge. No similar observations have been found described in ABR literature.

It is likewise uncertain why MM exhibited superior solid retention compared to BWC (Phase I), Gambiran and Santan and why in GB and ST significant COD_p reduction occurred only in one and two chambers respectively. Sludge heights and settled sludge TS and VS concentrations were comparable across plants.

BWC (Phase II) is not further considered in this discussion since it is doubtful whether its operation (especially concerning sludge activity) can be compared to the other case studies. The effect of sludge washout from the digester to the ABR on the increase of ABR treatment is uncertain at the time this thesis is written. Further investigations will have to show whether the improvement can be traced back to a change in organic loading or not.

MM exhibited by far the highest COD_s reduction of all systems, both in percentage as well as in absolute terms normalized over reactor volume. Three ABR chambers were involved in the statistically significant COD_s reduction in MM compared to only one in BWC and none in GB and ST.

The same trend appears with SMA_{max} values with MM having sludge with far greater average methanogenic activity than GB and ST which were approximately similar.

Observed normalized sludge increase values fit into the picture since also here MM distinguishes itself from the other case studies by exhibiting by far the least sludge build-up which indicates improved sludge stabilisation. GB and ST are again similar and BWC has a value about half way between them and MM.

Each indicator has an uncertainty associated with it which is amplified by the comparably small dataset sizes available for this study. This makes conclusions based on only one or two of these indicators difficult. The agreement of the six independent indicator datasets presented in Table 38 however lends strong credibility to the general conclusions that can be drawn from those datasets:

In summary, MM performs (at times by far) best in all the above presented performance indicators with the remaining three systems being similar. The tendency would be to attribute a slightly better performance to BWC and ST and the worst performance to GB.

A clear correlation can be observed between increased COD_p and COD_s removal, the partaking number of reactors in treatment, general methanogenic activity of the sludge and reduced sludge accumulation.

It therefore appears evident that the operating conditions prevailing in MM lead to a more efficient ABR treatment, both in terms of particle reduction as well as in terms of anaerobic digestion.

The main treatment rate determining factors for communal ABRs operating under roughly similar climatic conditions are the hydraulic and the organic load (see Section 2.3.7). Wastewater alkalinity was reported to play an important role in the pilot scale investigations conducted by Foxon (2009) but this seemed not to be the case in this study (see Section 6.7.1.3).

All case studies presented here were regularly exposed to flow surges of unknown but most probably considerable intensity during wet seasons. These flow surges were shown to have significantly affected the sludge dynamics and methanogenic activities in the two Indonesian plants GB, MM and ST. There is no reason to believe that one plant might have been less affected than the other. Their exposure to sudden hydraulic extreme loads is therefore assumed to be comparable. BWC in India was possibly less affected since sludge level investigations indicated no severe sludge migration over the first three years of operation. Flow measurements in BWC however clearly documented increased flow during rain events.

The wastewater treated in all four systems can be considered of similar type: it is in every case communal wastewater without industrial discharge. For unknown reasons the nonbiodegradable fraction of the wastewater was surprisingly high in BWC. Its mathematical effect was taken into account by subtracting it from all relevant values and rates presented in this chapter.

The loading of the four plants is now further discussed in order to identify why MM was performing better than the other investigated case studies. This is done assuming that no unknown external factor had an influence on the systems superimposing the effect of plant loading.

Table 39 summarizes average hydraulic and organic loading parameter values of the four case studies. Table columns from left to right are arranged depending on plant treatment quality following Table

38. The hydraulic load is expressed as HRT normalized over the number of ABR chambers, the mean up-flow velocity $v_{up,mean}$ and the measured maximum up-flow velocity $v_{up,max}$. The organic load is expressed as COD_t and COD_s feed concentrations and the OLR and soluble OLR. HRT and the two OLRs were all normalized in ways to represent the first reactor chamber only. This was done in order to simplify the comparison across systems which were built with a varying number of chambers. Also, treatment was in all cases only significant until ABR 3 which is in line with observations made in literature (see Section 2.3.7). The number of ABR chambers existing beyond the first three was therefore not considered relevant for the following comparisons.

Table 39: Summary of average loading indicator values

		MM	ST	BWC (Phase I)	GB
HRT (one chamber)	h	3	4	10	7
$\mathbf{v_{up,mean}}$	m h^{-1}	0.7	0.5	0.2	0.3
$\mathbf{v_{up,max}}$	m h^{-1}	1.3	0.8	0.5	0.6
Feed COD_t conc.	mg COD_t l^{-1}	426	339	413	383
Feed COD_s conc.	mg COD_s l^{-1}	178	131	298	149
OLR 1st chamber	g COD_t m^{-3} d^{-1}	3.5	2.0	1.3	1.4
soluble OLR 1st chamber	g COD_s m^{-3} d^{-1}	1.5	0.8	0.7	0.6

Bischofsberger et al. (2005) mentioned the lowest desirable anaerobic reactor feed concentration to be 400 mg COD_t l^{-1}. Shen et al. (2004) confirmed this specifically for ABRs by reporting a significant ABR performance decrease with feed concentrations below 350 mg COD_s l^{-1}. The investigation was performed on a laboratory scale system fed with purely soluble substrate. Average concentrations observed at the four case studies presented here were very much in the vicinity of these values and concerning COD_s even considerably below. All systems were therefore, at best, operating at the lowest concentration range limit desirable for stable anaerobic digestion.

The numbers presented in Table 39 indicate clearly that MM was exposed to the highest hydraulic and especially OLR of all four ABRs.

This observation implies that with such low feed concentrations and under the influence of seasonal extreme flow surges, the OLR becomes the determining factor for stable digestion, before the dry-weather up-flow velocity and HRT. OLR increase appears to improve the treatment more than HRT increase reduces it (see Figure 149, Figure 150 and Figure 151).

The ABR sludge activity assessment discussed in Section 6.7.5 indicates that active anaerobic MO populations were able to establish inside the case study ABRs in spite of the seasonal storm water influence. These MO consortia therefore were intrinsically resilient to washout and high up-flow velocities. The limiting factor for the activity of the sludges containing these populations would consequently have been substrate availability and OLR, not HRT and reactor up-flow velocity.

This goes principally against the notion generally held that communal ABRs should be designed mainly dependent on the HRT and up-flow velocity (because of sludge and MO retention):

Figure 149 to Figure 151 compare the COD removal rates reported in literature on communal ABRs treating composite wastewater with results from the case studies presented here. The investigations that are most comparable to the case study conditions are from Feng et al. (2008a), Bodkhe (2009) and Foxon (2009) (highlighted in the graph). Feng et al. (2008a) and Bodkhe (2009) describe systems run with COD_t concentrations below 500 mg l^{-1} whereas other studies were carried out with feed concentrations of 500 to 1400 mg COD_t l^{-1}. The values reported by Foxon (2009) represent the only non-laboratory study. Her pilot scale ABR was however different in some design aspects and fed with constant feed flow unlike the here presented real scale case studies which were exposed to daily flow fluctuations. The feed was degridded municipal wastewater with an average concentration of 750 mg COD_t l^{-1}.

As mentioned before, removal rates are always reported in literature to improve with reduced hydraulic loading (in the case of HRT only up to a certain point, approximately 10 h) and therefore reduced organic loading[16] (See Figure 149 to Figure 151). Also Foxon (2009) observed better treatment with lower $v_{up,mean}$.

The hypothesis however that all of these investigations deal with reactors operating with sludges of fundamentally different characteristics than in the here presented case studies and can therefore not be used for comparison appears plausible. Apart from the obvious fact the all systems described in literature were not exposed to extreme hydraulic loads, measured sludge characteristics support this: the sludge in the system described by Foxon (2009) for instance was about half as dense (see Section 6.7.2.3) and accumulated approximately four times as fast as case study sludge (see Section 6.7.2.1).

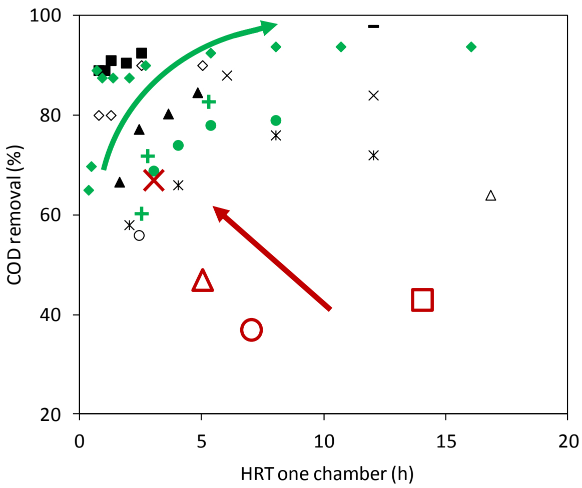

Figure 149: COD removal efficiency against one chamber HRT, data from literature and case studies

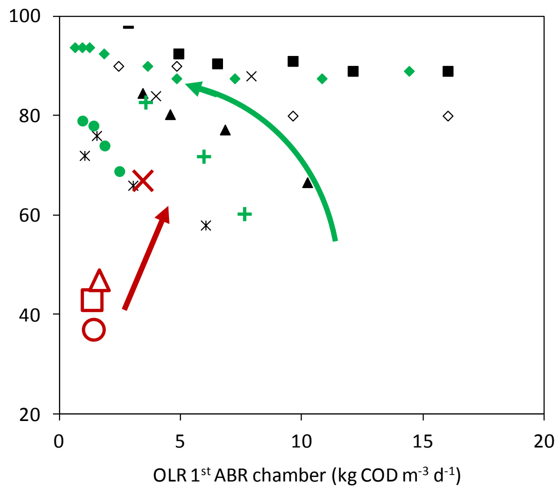

Figure 150: COD removal against 1st chamber OLR, data from literature and case studies

[16] All reactors presented in literature are shallower compared to reactor volume than the BORDA ABR design, leading to lower $v_{up,mean}$ values at similar HRTs.

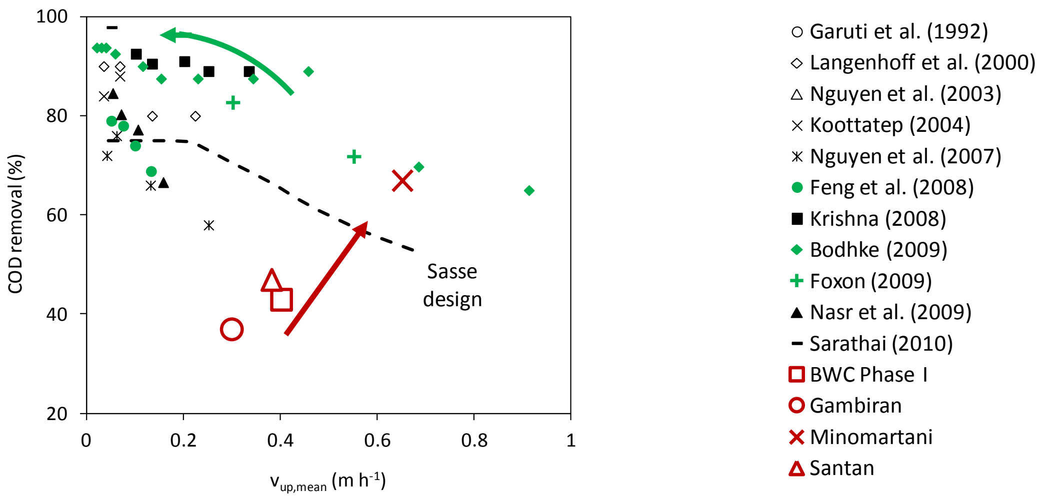

Figure 151: COD removal efficiency against average up-flow velocity $v_{up,mean}$, data from literature and case studies Curved arrows illustrate the general tendency of literature, straight arrows illustrate the general tendency of case studies

6.7.8. AF load estimations and COD removal rates

Figure 152 to Figure 154 summarize the values across plants for the load parameters HRT, average AF feed concentration and average OLR. All HRTs and OLRs were computed representing three chamber AFs since this is the most commonly adopted design and since the comparison across systems had to be simplified, MM being built with six AF chambers.

The AF of MM was designed with the lowest HRT, followed by ST and GB. Parameter field values are close to design expectations in all cases with GB being slightly higher hydraulically loaded than assumed during design.

Measured AF feed concentrations and OLR of all three systems are below design expectations (Figure 153). All three AF reactors however operated (during dry-weather flow) within hydraulic and organic loading ranges described in literature as being ideal for AF treatment (15 h to 25 h HRT; 0.1 to 0.3 kg COD m^{-3} d^{-1} OLR; see Table 8 in Section 2.4).

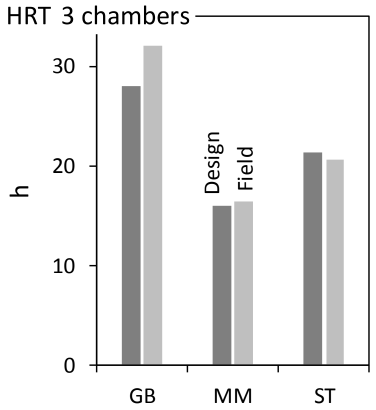

Figure 152: HRTs of three AF chambers at three plants

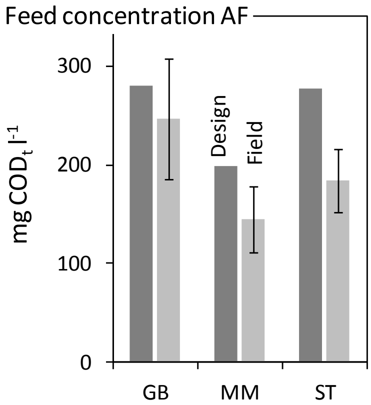

Figure 153: Average AF feed concentrations of three plants, error-bars indicate standard deviations

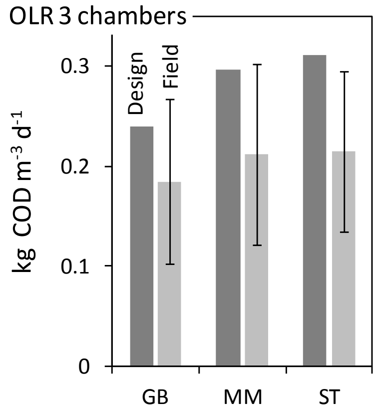

Figure 154: Average AF OLRs (three chambers) of three plants, error-bars indicate standard error of COD_t conc. measurements and a 20% variation of Q

The pH measured in all AF chambers indicate good and stable anaerobic conditions with median values at about pH 7 and the minimum values never sinking below 6.5. The wastewater temperature at all three plants was always between 27°C and 30°C and did not vary noticeably across seasons.

Figure 155 compiles the COD_t concentration profiles as measured across the first three AF chambers of the three plants GB, MM and ST. COD_s and COD_p reductions across the first three chambers were found to be statistically significant in all cases (see sections 6.4.8.3.3, 6.5.8.3.3 and 6.6.8.3.3). The average COD_s reduction was slightly above 25% in all three cases. COD_p removal at the AFs of GB and ST were both similar and slightly below 70% but significantly lower in MM with approximately 30% reduction. Low COD_p reduction in MM may have been due to the longer period of plant operation with high sludge levels saturating the AF growth media with sludge particles over time.

The resulting AF COD_t removal rates were about 50% at GB and ST and only 25% in MM. The AFs performed in all instances significantly below design expectations (see Figure 156). Their treatment was also poor compared to most published studies which report treatment efficiencies of approximately 60% to 80% COD_t removal for similar loading conditions (Bodik et al., 2002, Inamori et al., 1986, Reyes et al., 1999, Manariotis and Grigoropoulos, 2006, Young, 1991).

The effluent COD_s concentrations represented 70%, 65% and 76% of the effluent AF COD_t concentrations at GB, MM and ST respectively.

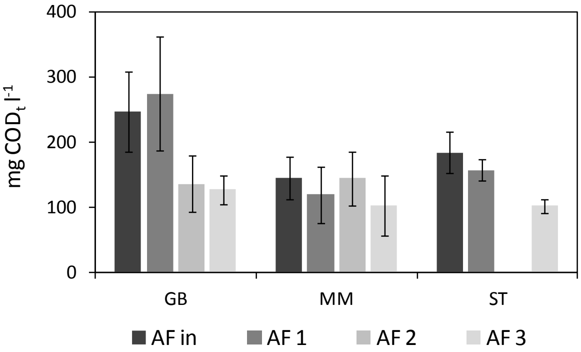

Figure 155: Average COD$_t$ concentration profiles of the three plants, error-bars indicate standard deviations

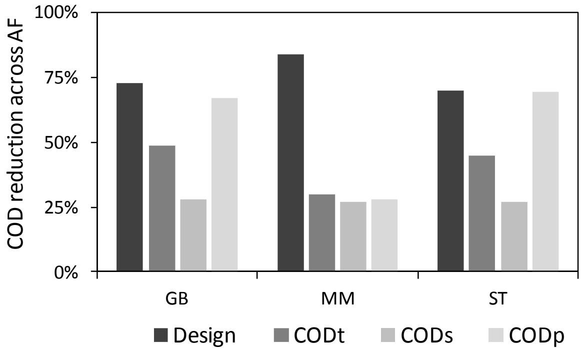

Figure 156: Average COD$_t$, COD$_s$ and COD$_p$ AF reduction rates of three plants

6.7.9. Effect of dry weather loading rates on case study AF treatment

This study did not concentrate on AF treatment and therefore did not consider a number of factors crucial for the understanding of the reactor operation such as the types of AF growth media, their specific surface area, filter void ratio or the methanogenic activity of biomass retained on the growth media surface. Available COD concentration and hydraulic load data however allow a number of meaningful observations to be made:

The sludges accumulating below the AF growth media had very low methanogenic activities and appeared to be mainly sludge washed out of the ABRs. These sludges exhibited very low methanogenic activity. Organic removal therefore probably mainly occurred within the growth media voids.

The AF reactors played an important role in the overall DEWATS COD reductions of all three case studies. They significantly reduced the ABR effluent COD$_s$ and COD$_p$ concentrations for which no further treatment could have been expected by the ABRs since no statistically significant COD reductions were observed in the rear ABR compartments. In GB and ST the AFs were even the only DEWATS reactors reducing COD$_s$ concentrations at statistically significant levels.

The observed AF COD removal was however in all three case studies considerably below literature and design expectations. In two out of three cases the main COD reduction was achieved through retention of COD$_p$. All effluents were largely biodegradable with high BOD$_5$ to COD ratios of 0.58, 0.68 and 0.77 for GB, MM and ST respectively (see Figure 157) which indicates that higher treatment efficiencies should have been achievable. It is hypothesised that the AFs would have achieved better removal rates if they had not been exposed to extreme hydraulic surges during wet seasons.

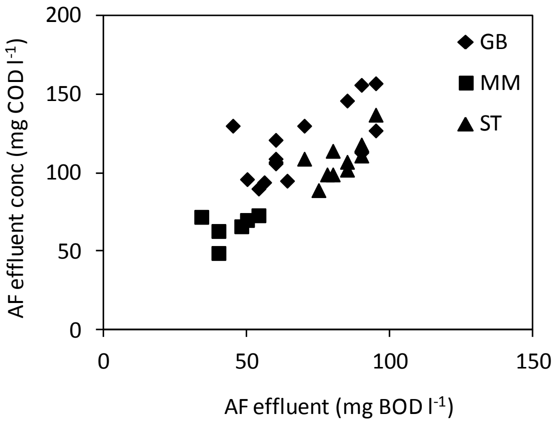

Figure 157: BOD_5 against COD AF effluent concentrations

6.8. Conclusions

6.8.1. Conclusions on case study pre-treatment steps

The average HRTs of all case study pre-treatment steps were significantly larger than the value of 2 h proposed by Sasse (1998).

Plant feed concentration measurements were not part of this study. It is therefore not possible to directly assess the treatment efficiencies of the pre-treatment steps with the available data. The surprisingly low concentrations measured in settler effluents indicate however that the pre-treatment design assumptions need to be revised. It appears that either the per capita organic loads were far lower or the pre-treatment efficiencies far greater than assumed. Available literature supports the first reason but future field investigations are needed to clarify this point.

Operation of BGDs was not the primary focus of this study. One BGD however was monitored during the course of the investigations. Available data on effluent concentration and biogas-production indicated a COD removal efficiency of at least 73%. Further outcomes are summarized in Section 6.3.8.3.

6.8.2. Conclusions on the case study ABRs

The sludge activities and therefore treatment efficiencies of at least three case study ABRs were impeded by storm water intrusion. ABR feed concentrations were generally very low and close to the minimum limit stated in literature generally acceptable for good treatment. The variation of measured SMA_{max} values across reactor chambers supports the hypothesis that the investigated systems were organically under-loaded.

All case study ABRs however featured active sludge and significantly stabilised retained particulate organics. This implies that stable anaerobic consortia were able to establish inside the reactors. This is remarkable under the extreme hydraulic conditions these systems were operating under and confirms their reputation as being very robust and resilient to operative fluctuations.

Three out of four plants however performed poorly in terms of COD_t reduction rates which were below 50%. The fourth system performed considerably better (68% COD_t reduction) however without reaching treatment efficiencies as reported in literature of 80% to 90% COD_t removal.

Field observations confirmed published laboratory investigations that most treatment occurs in the first three ABR chambers and little, if any, beyond.

A comparison of the available case study data suggests that under the operational conditions they experienced (very low feed concentrations and exposure to storm water) the ABRs performed better when operated with comparably high hydraulic load in order to reach a minimum OLR. Below this minimum OLR sludge activity appeared to be seriously impeded. This however goes against the generally published view in literature that HLR is the decisive design parameter over OLR. Further investigations are needed to confirm this.

6.8.3. Conclusions on the case study AFs

The AFs of all three case studies significantly reduced COD_p and COD_s concentrations to levels the ABRs appeared unable to. In two cases the AFs were the only DEWATS reactors reducing COD_s concentrations at statistically significant levels. With 25% to 50% COD_t reduction none of the reactors however reached design and literature treatment expectations. The comparably high biodegradable organic content of their effluents indicates that better removal rates, especially regarding COD_s, may be possible.

6.8.4. Conclusions on general DEWATS and ABR design and operation

Existing design procedures cannot be directly tested with the available data since all case study data is influenced by hydraulic surges. A number of conclusions however can be drawn concerning future ABR and DEWATS design and operation:

- The flow to the system needs to be controlled by deflecting peak flows caused by storm water. One aspect would be improved run off management which would certainly reduce the probability of storm water infiltrating the reticulation system. It would however not exclude the infiltration of possible run-off coming upstream from the community, groundwater infiltration or having non-authorized storm water connections and broken manholes or pipes. It is therefore imperative to limit the feed flow at the DEWATS inflow level. A conceptual proposition resolving some of the technical difficulties this involves is presented in Appendix A5.

- Measures should be taken to achieve higher feed concentrations which would certainly improve the general DEWATS treatment efficiency. This could for example be achieved by separately treating or percolating a fraction of the grey-water produced by the connected community. Also, the pre-treatment should possibly be designed smaller than how it was done for the case studies which would certainly increase the organic load to the ABR.

- Methanogens succeeded in establishing a stable population under extreme hydraulic conditions in all 4 case study reactors. $v_{up,max}$ values exceeding the existing design value of 1 m h^{-1} therefore appear possible. It is proposed to build and test an ABR prototype designed with 2 m h^{-1} $v_{up,max}$ (which corresponds to 1 m h^{-1} $v_{up,mean}$).

- Significant COD removal occurs only in the first three ABR chambers. Field data therefore confirms the recommended ABR design with four to five chambers in order to guarantee good long term operation.

- ABR particulate retention should be improved by increasing the area of the last ABR chamber and connecting a simple lamella clarifier to its effluent.

- Front ABR chambers should never be desludged since they contain the sludge with the highest methanogenic activity.

- It is suggested to use the performance indicator values identified for the case study MM as benchmark for good reactor operation in future investigations. SMA and per capita sludge accumulation rates can be investigated with comparably little effort.

- The best performing case study (MM) data confirmed the design calculations by Sasse (1998). Since this plant was not operating under optimal conditions, better treatment than predicted by Sasse (1998) for organically low loaded systems may be possible.

6.8.5. Future research needs at the four case study sites

It is suggested to continue the monitoring of all four case study sites in order to consolidate the existing data and the conclusions presented here and in order to document future operational changes. A focus should be laid on testing reticulation system integrity.

The existing datasets on hydraulic plant loadings were particularly small for the case studies GB, MM and ST and should be consolidated through future measurements.

The AF chambers of the case study system MM should be completely desludged in order to measure the subsequent sludge washout from the ABR and validate the here presented sludge accumulation values.

SMA measurements such as performed in the course of this study were found to be very useful for the assessment of general sludge activity. However still little experience exists with this methodology and a larger data base covering a longer measurement period should be produced in order to further assess the variability of the measurement and of the sludge activity across systems, reactor chambers and seasons.

It is further suggested to conduct detailed plant feed concentration measurement campaigns at at least two sites in order to quantify the per capita COD production and to confirm the here presented estimations.

Turbidity and EC measurements across reactor chambers were found to be helpful parameters and should be included in all future monitoring activities.

Precipitation currently strongly influences DEWATS treatment. Due to recent changes in seasonal weather patterns and the extremely local character of tropical rain events, official precipitation data is often not accurate enough to relate it to DEWATS performance. It is suggested that daily precipitation measurements be performed in order to fill this information gap.

The data gathered at BWC during operational Phase II did not allow strong conclusions to be drawn at the time this thesis was written. It did however indicate an increase in overall ABR treatment with increased organic loading. Confirmation on this could be gained by upholding the operational conditions and continuing system-monitoring. The suggested monitoring schedule at BWC is:

Monitoring the effect of seasonal changes with simple inexpensive methods

- Biogas production (monthly)
- Turbidity measurements in ABR feed and ABR 5 (weekly)
- Precipitation measurements (daily)
- Wastewater temperature measurements at ABR feed (weekly)

Monitoring the ABR treatment

- SMA (monthly)
- Sludge heights (monthly)
- COD_p and COD_s at ABR_{in}, ABR 1 to ABR 5 (monthly)

Maintenance

- The rear ABR chambers (ABR 6 to ABR 12) should be desludged in order to prevent the PGF from clogging

7. MODELLING

7.1. Background

The case study ABRs presented in Chapter 6 are known to have operated under adverse conditions and have performed below design expectations. The interpretation of the comparably scarce field data in existence was hampered by the fact that very little or no experimental data was available on the treatment performance of other full-scale ABRs operating under undisturbed conditions. It was therefore difficult to relate the treatment parameter values (notably the observed sludge accumulation rates and the effluent COD_s concentrations) observed during the case studies to other research.

With regard to modelling, this signifies that a full representation of the ABR treatment relevant processes in a model was not possible at the time this thesis was written due to lack of data. The following modelling exercises were instead driven by the necessity to obtain benchmark value estimations for the operational parameters *sludge build-up* and *effluent* COD_s *concentration* in order to assist the interpretation of available scarce field-data.

This was done by adapting an existing state of the art anaerobic model so as to represent the closest possible approximation of an ABR treating communal wastewater in order to provide first estimates for the required benchmark values.

7.2. Objectives

7.2.1. Objective 1: Assessing sludge activity with modelled sludge build-up

In its communal application ABR technology can be regarded as a retention and digestion device for particulate organic wastewater components.

The two main treatment influencing parameters are therefore solid retention and anaerobic sludge activity. The assessment methods of system activity normally used for anaerobic systems (methane production measurements and COD mass balances) are not feasible in the case of DEWATS-ABR due to technical and investigative factors. Also, small scale communal wastewater treatment facilities are intrinsically exposed to large variations of both organic loading and hydraulic loading. Accurate stream concentration measurements and load measurements are therefore very resource intensive and, in the case of large numbers of systems with wide geographical spread, impossible to carry out. Results presented in Chapter 6 however indicated that sludge build-up could be a good qualitative indicator for ABR sludge activity. This indicator would have to be measured very infrequently since it has the advantage of representing the complete cumulated plant loading history as opposed to point in time stream measurements.

In Chapter 6 the measured sludge build-up rates in four ABR systems were compared to the theoretical accumulation inferred from flow and COD_p concentration measurements assuming no anaerobic activity. Observed accumulation rates were in all four cases significantly inferior to what they would have been, had there been no anaerobic activity.

Anaerobic modelling is presented in this chapter as a positive control of this assessment by predicting the amount of sludge expected to accumulate under active and inactive sludge conditions. The hypothesis to be tested is:

Anaerobic digestion modelling confirms that observed sludge accumulation rates indicate active anaerobic treatment in all four case studies.

7.2.2. Objective 2: Assessing treatment efficiency with model benchmark values for COD_s

System efficiency is generally expressed as COD_t decrease which is composed of COD_p retention and COD_s reduction.

COD_p reduction depends on the ability of the ABR to retain particulate material. During normal hydraulic loading conditions and after an initial start-up period this is mainly a function of the filtering characteristics of the accumulated sludge. During start-up, the sludge settling velocity governs the amount of sludge being washed out of the reactor during peak flows. After a certain time of operation however, sludge characteristics adapt to the operating conditions. Only particles with settling velocities fast enough to withstand the peak flow under which a reactor is operated remain inside the system, forming the sludge blanket of which the filtering characteristics will influence COD_p reduction.

COD_s reduction depends on the anaerobic activity inside the system. The hydrolytic activity of the sludge will on the one hand tend to increase the measured COD_s since retained biodegradable solid organics, previously measured as COD_p reduction, will be converted to soluble organics. On the other hand methanogenic activity leads to COD_s decrease since it degrades the VFAs to CH_4 and CO_2 which then leave the aqueous phase.

Measured COD_s concentrations are thus difficult to interpret and cannot be used as a direct indicator for system treatment condition.

Anaerobic digestion modelling was therefore carried out with the objective of interpreting the existing COD_s measurement data by providing benchmark COD_s effluent concentration estimates for systems operating with active sludge. The hypothesis to be tested is:

Field COD_s measurements are similar to anaerobic digestion modelling results with active sludge which therefore supports the assumption that all four case studies operate under active anaerobic conditions and reach satisfactory treatment efficiency.

7.2.3. Objective 3: Assessing effect of loading rate on treatment

The data gathered at four case study sites and presented in Chapter 6 suggests that under the prevailing operational conditions (seasonal exposure to extreme hydraulic loads and very low organic load) and with all systems having a similar feed concentration, anaerobic digestion was best in the systems with highest loading rate.

All case study systems are known to have been exposed to extreme flow surges during the tropical wet seasons when great amounts of rain water infiltrated the piping systems. Nevertheless, methanogenic MO populations were able to establish under such adverse conditions, therefore displaying great resilience to hydraulic peaks.

In literature the main limiting factor for communal ABR performance is generally accepted to be the hydraulic load because of its effect on sludge and MO retention. It also governs the length of time solubilised organics are exposed to the degrading metabolisms of anaerobic MOs as substrate. The OLR on the other hand is known to have less influence as long as a minimum level is guaranteed. The ability of the ABR to efficiently treat highly concentrated wastewater has been demonstrated repeatedly.

However, very low feed COD concentrations were measured in all four case studies. Comparisons with literature values make it appear plausible that in all case studies general sludge activity was limited by low feed concentrations and the resulting low OLR.

One system however was observed to operate significantly better than the others while having been exposed to significantly higher hydraulic loading but to similar feed concentrations.

It was therefore hypothesised that for the case studies, the normal weather hydraulic load became non-limiting due to the preselection of sludge for extremely high flows during the wet season. An increase in feed flow would therefore not have been problematic from a hydraulic point of view. On the contrary, at constant feed concentration it would have represented an increase of the organic load improving the sludge activity. The positive effect of the OLR increase would have outweighed the negative effects of reduced HRT and v_{up} increase.

This hypothesis could not be verified using the available model since it implies non-typical system dynamics. The trivial outcome of a modelling exercise in which the feed flow is increased at constant concentrations would have been a decrease in treatment since reduced HRT gives the liquid phase reactions less time to occur.

In the light of what has been said about the low feed concentrations measured in the field and exploring possibilities of how to improve the field treatment it was however hypothesised that increased feed concentration would improve the methanogenic activity and COD_s treatment efficiency. The hypothesis to be tested with the model therefore is:

Anaerobic digestion modelling supports the hypothesis that increased ABR feed concentration would lead to general treatment improvement. Improvement indicators are COD_s effluent reduction and increase of acetoclastic methanogens (Xam) sludge content.

7.3. Conceptual overview of the model

7.3.1. General

The available experimental data was not sufficiently detailed for a specific ABR model to be developed. The modelling was therefore done using the existing ADM-3P model (Ikumi, 2011) which represented the most scientifically advanced and best calibrated option in the field of anaerobic digestion modelling of communal wastewater by a completely stirred tank reactor (CSTR) at the time this study was undertaken. The modelled ABR thus had to be represented as one CSTR without differentiating between reactor chambers.

The modelling exercise is based on the following basic modelling assumptions:

1. The ADM-3P model summarizes all existing and relevant knowledge of AD processes occurring in communal wastewater treatment at the time this thesis is written.
2. The current model-calibration done with communal wastewater (Ikumi, 2011) represents the best available approximation for communal AD treatment.
3. Uncertainties concerning the applicability of the available calibration of kinetic rate constants for an ABR context are outweighed by the variability of the available field data and can therefore be neglected.
4. A fully mixed digester is approximately comparable to an ABR in terms of COD_s reduction and sludge accumulation.

Assumption 3, being crucial for modelling results interpretation, is tested by investigating the sensitivity of the model towards the hydrolysis rate constants and the maximum growth rates of methanogens.

7.3.2. The ADM-3P Model

The modelling was done using a state of the art dynamic anaerobic digestion model (ADM-3P) representing a CSTR and calibrated for communal wastewater by Ikumi (2011). The ADM-3P model had been developed by extending the UCTADM1 model from Sotemann (2005) as part of an effort to develop a plant-wide dynamic modelling setup. The model therefore had to cater for the processes involved in the digestion of waste activated sludge (WAS) containing phosphorous accumulating organisms. It consequently included three phase (aqueous-gas-solid) mixed weak acid/base chemistry. This certainly exceeded the requirements and complexity of a model that would have been needed to test the above mentioned hypotheses.

The hydrolysis kinetic constants of the ADM-3P model were calibrated with data from laboratory-scale CSTRs operated at 35°C with 10 d, 18 d, 25 d, 40 d and 60 d sludge age fed with primary sludge (Ikumi, 2011). Sludge ages are significantly higher in ABRs which certainly affects the hydrolysis rates actually relevant for ABR processes. The calibration by Ikumi (2011) was nevertheless chosen for this study since it represented the best available approximation for the anaerobic treatment of communal wastewater at the time.

Model parameters as calibrated in Ikumi (2011) (see Appendix A4), with the exception of the kinetic rate constants for hydrolysis *K_bp* and *K_bps* and all maximum specific growth rates *mu*, were kept constant throughout all modelling runs.

Using the ADM-3P model introduced a number of uncertainties which, in addition to the existing uncertainties associated with the experimental data, had to be addressed during the modelling exercise. The ADM-3P model for instance requires information on the feed and initial sludge fractions that were not directly available from field data. It therefore had to be applied in a larger context, the *process model*.

7.3.3. The process model

Modelling was performed in the WEST® modelling environment which allows the combination of multiple single models to a larger *process model*.

The *process model* used in this study included two ADM-3P models (further referred to as "sub-models 1 and 2") connected in series (see Figure 158). The first represented the pre-treatment step (sub-model 1), the second the ABR (sub-model 2).

The effects of uncertainties resulting from the available field-data were explored by varying all parameter values representing wastewater streams and initial sludge characteristics (daily flow, concentrations of COD fractions, initial sludge fractions) in the course of Monte-Carlo type analyses, using the *Uncertainty Analysis* (UA) function provided by WEST®. One analysis consisted of 100 modelling runs during which parameter values were varied within their defined respective probability distributions.

The probability distributions of concentrations had to be estimated based on spot measurements of the same streams made at different times. Their means represented the best available estimates for long-term system operations. Consequently, the distributions used in the Monte-Carlo procedure were distributions of the means, rather than distributions of the spot measurements. The measure of dispersion chosen for measured concentrations was therefore the standard error of mean σ_m[17].

The volumetric flow rate Q was assumed to be uniformly distributed. The chosen confidence limits were always 20% of the measured average flow which is in line with observed variations in the field (see Section 4.3.1.2).

Each run during the uncertainty analysis was conducted for a modelling period of 600 d in order to allow a pseudo-steady state to establish.

The considered output variables of the process model were the *Sub-model 2* CODs effluent concentration, VS fractions and total mass of VS accumulated inside *Sub-model 2* after each modelling run.

All modelling runs were performed at a temperature of 28°C.

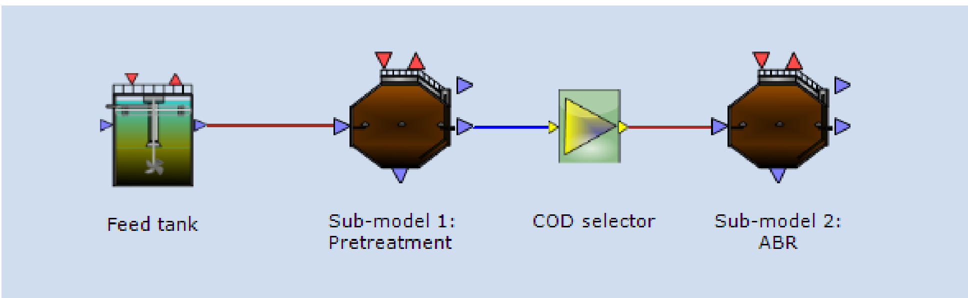

Figure 158: Process model setup in WEST®

[17] The standard error of mean σ_m is calculated as $\sigma_m = \sigma/\sqrt{n}$ with σ being the standard deviation of the dataset and n the sample size. For three and more samples the mean may be considered normally distributed (Davis and Goldsmith, 1977).

7.3.4. Process model component Sub-model 1: pre-treatment

The pre-treatment is modelled as one completely stirred tank reactor: Sub-model 1 predictions do not account for particulate retention of the reactor. The effluent particulate concentration (VSSeffl) therefore had to be set as a model input parameter with its input values based on field data.

Table 40 compiles the Sub-model 1 related input parameters varied during each uncertainty analysis.

Sub-model 1 feed characteristics are defined as “Feed tank” parameters. All parameters set for sub-model 1, with the exception of VSS effluent concentration, define the initial sludge mass and composition inside the reactor at the beginning of each uncertainty analysis run. Xoh and Xpa play no decisive role in the model for the application described here. The parameters are however listed for the sake of completeness.

Table 40: Sub-model 1 input parameters which had to be adjusted for each case study dataset

Sub-model	Parameter	Unit	Description	Distribution during UA
Feed tank	BPO_PS	g m^{-3}	Biodegradable particulate organics effluent concentration	Normal
Feed tank	FSO	g m^{-3}	Fermentable soluble organics effluent concentration	Normal
Feed tank	NH4	g NH_4 m^{-3}	Ammonium effluent concentration	Normal
Feed tank	PO4	g PO_4 m^{-3}	Phosphorous effluent concentration	Normal
Feed tank	UPO	g m^{-3}	Unbiodegradable particulate organics effluent concentration	Normal
Feed tank	USO	g m^{-3}	Unbiodegradable soluble organics effluent concentration	Normal
Feed tank	Q Pump	m^3 d^{-1}	Daily effluent flow	Uniform
Pre-treatment	BPO	g VS	Biodegradable particulate organics (resulting from MO decay), initial sludge content	Uniform
Pre-treatment	BPO_PS	g VS	Initial sludge content	Uniform
Pre-treatment	ER	g VS	Endogenous residue, initial sludge content	Uniform
Pre-treatment	ISS	g ISS	Inert settable solids, initial sludge content	Uniform
Pre-treatment	UPO	g VS	Initial sludge content	Uniform
Pre-treatment	Xac	g VS	Acetogens, initial sludge content	Uniform
Pre-treatment	Xad	g VS	Acidogens, initial sludge content	Uniform
Pre-treatment	Xam	g VS	Acetoclastic methanogens, initial sludge content	Uniform
Pre-treatment	Xhm	g VS	Heterotrophic methanogens, initial sludge content	Uniform
Pre-treatment	Xoh	g VS	Ordinary heterotrophic organisms	Uniform
Pre-treatment	Xpa	g VS	Phoshorous accumulating organisms, initial sludge content	Uniform
Pre-treatment	VSSeffl	g VS m^{-3}	VSS effluent concentration	Normal

No pre-treatment feed concentration measurements were performed in the field. The average feed organic concentrations therefore had to be extrapolated from the available pre-treatment effluent data. The literature value of 50% pre-treatment COD_t efficiency (see Section 2.1.2.2) was used. The feed standard errors of mean (σ_m) were assumed to be three times the effluent σ_m to cater for the known high feed concentration variations.

The estimated pre-treatment COD_s and COD_p concentrations are converted to "Sub-model 1" input values (see Table 40) based on the following equations:

$$COD_p = BPO\_PS + UPO \qquad \text{Equation 6}$$
$$UPO = f_{UPO} * COD_p \qquad \text{Equation 7}$$
$$COD_s = FSO + USO \qquad \text{Equation 8}$$

f_{UPO} is reported to be 0.13 to 0.22 (Ekama et al., 1986) of which the average value (0.175) was adopted.

USO concentration measurements were taken from the ABR effluents. Melcer (2003) reports that USO does not significantly change over the anaerobic process. The available effluent concentrations were therefore used for all streams.

The conversion factors to express the fractions as g COD are given in Table 41 and based on Ikumi (2011).

Table 41: Conversion factors for the pre-treatment input data (Ikumi, 2011)

Fraction	BPO_PS	UPO	FSO	USO	VSS
Conversion factor (g COD g^{-1})	1.466	1.504	1.418	1.418	1.48

Input hydrogen and carbonate concentrations were adjusted by trial and error in order to match the measured feed alkalinity concentrations and pH values.

7.3.5. Process model component: COD selector

The process model incorporates the modelling of the ABR feed fractions as the output parameters of sub-model 1. However, no field data was available on the pre-treatment feed concentrations which had to be estimated using literature. Consequently, the intermediate stream between sub-model 1 and 2 is expected to have the appropriate fractionation of components but not the correct absolute concentrations. A *COD selector* (see Figure 158) was therefore introduced to calculate the absolute concentrations of the ABR feed fractions based on available ABR feed COD_p and COD_s concentration measurements (which did not reflect the fractionation required by the model).

The *COD selector* calculates the concentrations of the different ABR feed COD_p and COD_s fractions (see Equations 9 and 10) based on their ratios given by the sub-model 2 output. Ac, Pr, H_2 and Glu stand for acetate, propionate, hydrogen and glucose respectively. The input parameters for the *COD selector* are the COD_p and COD_s ABR feed concentrations with uniform probability distribution.

$$COD_p = VS + ISS \qquad \text{Equation 9}$$
$$COD_s = Ac + Pr + H_2 + USO + FSO + Glu \qquad \text{Equation 10}$$

7.3.6. Process model component Sub-model 2: ABR

A modelled ABR is represented by one completely stirred tank reactor without differentiating between reactor chambers. Model predictions thus cannot account for the hydraulic and microbiological particularities of ABR compartmentalisation or particulate retention. The effluent particulate concentration therefore had to be set as a model input parameter with its input values based on field data.

The initial ABR sludge fractions (see Table 42) had to be defined in order to start the model. Since no field data was available on them, the uncertainty analysis was first run with random initial sludge values for a long modelling time period which would lead to pseudo steady state of the system (600 d). The 95%-tiles of the resulting sludge fraction-ratios (percentages of total sludge VS) were then determined and used to calculate the confidence limits for the seed sludge fraction masses for further modelling. The seeding masses were calculated such as to represent an approximate 40 cm sludge blanket inside the reactor using the available settled sludge VS concentration field data. 40 cm of sludge is considered the minimum sludge height conducive to good operation. Such a sludge blanket would cover the down flow pipes which end 20 cm above the reactor base, therefore supposedly allowing good mixing of sludge and feed wastewater. Xoh and Xpa play no decisive role in the model in this particular application. The parameters are however listed for the sake of completeness.

The sludge VS fractions considered by the model are detailed in Equation 11. UPO represents the complete non-biodegradable fraction of VS.

Sludge VS = BPO + BPO_PS + ER + UPO + Xac + Xad + Xam + Xhm + Xoh + Xpa *Equation 11*

Table 42: Sub-model 2 input parameters which had to be adjusted for each case study

Sub-model	Parameter	Unit	Description	Distribution
ABR	BPO	g VS	Initial sludge content	Uniform
ABR	BPO_PS	g VS	Initial sludge content	Uniform
ABR	ER	g VS	Initial sludge content	Uniform
ABR	ISS	g VS	Initial sludge content	Uniform
ABR	UPO	g VS	Initial sludge content	Uniform
ABR	Xac	g VS	Initial sludge content	Uniform
ABR	Xad	g VS	Initial sludge content	Uniform
ABR	Xam	g VS	Initial sludge content	Uniform
ABR	Xhm	g VS	Initial sludge content	Uniform
ABR	Xoh	g VS	Initial sludge content	Uniform
ABR	Xpa	g VS	Initial sludge content	Uniform
ABR	VSSeffl	g VS m^{-3}	VSS effluent concentration	Normal

7.3.7. Comparing active and inactive systems

The further modelling procedure included running two complete uncertainty analyses with the same model setup: one representing a system with active sludge (with all parameter values adopted from Ikumi (2011), see Appendix A4) and one representing a system with inactive sludge. In the latter case the hydrolysis kinetic rate constants K_bp and K_bps and the maximum specific growth constants (mu) of all organism groups were set to zero inactivating the hydrolysis and all consecutive degradation processes in the model except decay of micro-organisms. Complete "inactivity" of sludge is understood to represent an extreme, idealized scenario unlikely to ever occur in the field but was nevertheless used since knowledge concerning the correct kinetics of a more realistic "sludge inhibition" was not available. The saturation kinetics equation on which the rate of hydrolysis is based as well as the Monod equations governing the organism growth rates are detailed in Sotemann (2005).

7.4. Input data for the four case studies

Table 43 specifies the input data used for the four modelling runs. The data was computed based on the field concentration values presented in Chapter 6. Measured feed alkalinity had to be raised to 750 mg $CaCO_3$ l^{-1} for model input in order to avoid souring of settler and ABR. The initial data averages were 414, 180 and 360 mg $CaCO_3$ l^{-1} for Gambiran, Minomartani and Santan respectively. The reason for the model souring at these alkalinities could not be found but is probably related to the fact that the ADM-3P model was not an entirely appropriate representation of the ABR. Reactor souring is certainly not in line with field data since souring of ABR has never been observed with process wastewater which was always above pH 6.5, generally close to pH 7. It was therefore decided to artificially raise the alkalinity in the model in order to maintain a reactor pH close to field observations.

The BWC ABR model was sized so as to correspond to five reactor chambers (of the twelve installed). All other case study ABR model sizes represent the reactors as they have been built.

Table 43: Model input values based on field data presented in Chapter 6

Parameter	Unit	BWC	GB	MM	ST
Reactor volume settler	m³	19.3	19.9	11.25	19.2
Reactor volume ABR	m³	11.3	19.2	21	32
Q	$m^3\ d^{-1}$	6.3	17.5	27.3	36.4
Alkalinity $_{in,\ pretr.}$	g $CaCO_3$ m^{-3}	1240	750	750	750
pH $_{in,pretr.}$		7.1	7	7.2	7.2
EC	µS cm^{-1}	913	500	500	500
BPO_PS $_{in,\ pretr.}$	g m^{-3}	260	485	446	351
UPO $_{in,\ pretr.}$	g m^{-3}	55	103	95	74
FSO $_{in,\ pretr.}$	g m^{-3}	623	299	357	261
USO $_{in,\ pretr.}$	g m^{-3}	114	20	20	20
VSS $_{out,pretr.}$	g m^{-3}	107	199	183	144
COD_s $_{in,\ ABR}$	g m^{-3}	368	159	188	141
COD_p $_{in,\ ABR}$	g m^{-3}	158	294	270	213
VSS $_{out,ABR}$	g m^{-3}	42	76	37	53

7.5. Modelling results and discussion

7.5.1. Objective 1: Assessing sludge activity with modelled sludge build-up

Observed sludge build-up rates in four ABR systems are compared to model outcomes representing systems with inactive and active sludge (see Figure 159). The model input parameters were each varied based on available information from field data using a Monte-Carlo type uncertainty analysis. The resulting 95% confidence interval for sludge-build up is represented in Figure 159 by the error-bars. In all four cases the measured sludge build-up rates fall within the ranges modelled with active digestion or below. Sludge washout as the sole mechanism leading to the observed build-up rates appears unrealistic for all four case studies since comparably little sludge accumulation is observed in the AFs which follow the ABRs (see Figure 159). Future field measurements on long-term particulate COD washout will however be needed to confirm this. The BWC setup does not include an AF which is why the corresponding data point is not shown in Figure 159. In any case no or very low sludge levels were measured in the seven chambers following the five chambers modelled here.

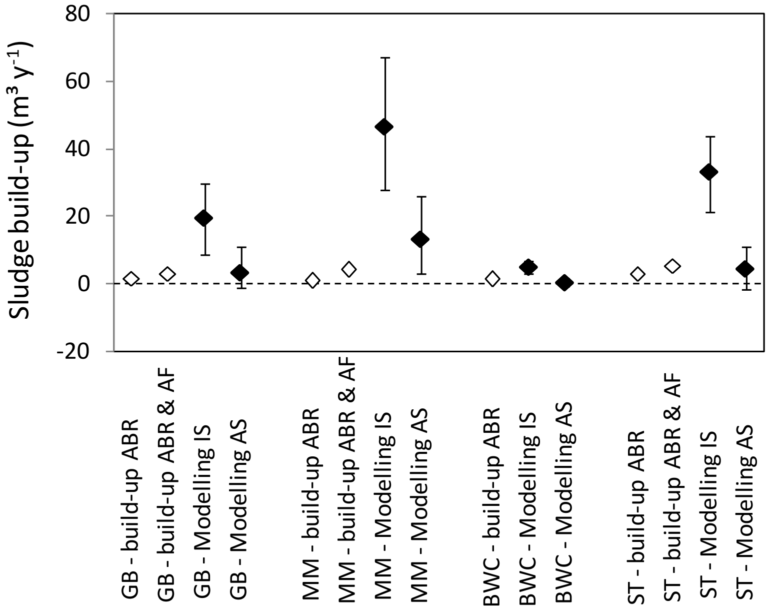

Figure 159: Average sludge build-up rates in m³ y^{-1}, field data (not full), modelled data (full), error-bars of full data points represent 95% confidence intervals of modelled outcomes after Monte-Carlo type uncertainty analysis taking into account the measured uncertainties of model input data

The sensitivity of the modelled sludge build-up towards kinetic rate constants was explored by varying the hydrolysis rate constant of the GB model setup such as shown in Figure 160. The model predictions were found to vary little even when the hydrolysis rate was reduced to only 20% of its initial value. This strengthens confidence in the basic modelling assumption number 3 postulated in Section 7.3.1 that uncertainties concerning the applicability of the available calibration of kinetic rate constants may be neglected when drawing conclusions from the modelling results.

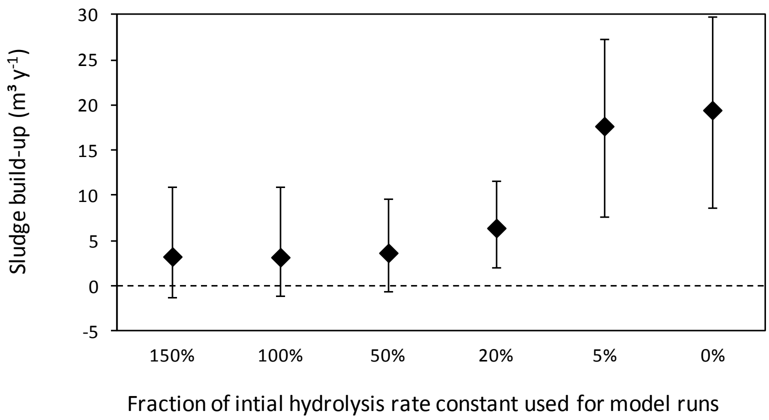

Figure 160: Sensitivity of the modelled sludge build-up rate towards the hydrolysis rate constant, error-bars represent 95% confidence intervals of modelled outcomes after Monte-Carlo type uncertainty analysis taking into account the measured uncertainties of model input data, modelling runs done with GB data

The modelling exercise therefore supports the hypothesis that the four investigated ABR systems contain active sludge. This suggests that sludge build-up rate measurements may in future be used to assess ABR system activity, in cases in which major sludge washout can be excluded (e.g. when sludge levels inside AFs allow this conclusion). The easiest way to normalize the build-up rate in order to obtain values that can be compared across systems would certainly be to divide it by the number of connected users. The alternative would be costly and time consuming hydraulic and organic load measurement campaigns. A user number assessment on the other hand can be done with comparably little effort assuming the cooperativeness of community leaders and approximately similar per capita loading rates across communities. Such an assessment would have to happen simultaneously with an investigation of the plant history in order to take possible desludging events and user connection changes into account. This should also be achievable with little effort by communicating with community leaders. It is further important to at least test the main piping sections (e.g. with food-dye tests) in order to exclude the possibility of severe blockages or breakages. The latter could possibly lead to an overestimation of the system load therefore under-estimating the normalized sludge production.

Table 44 summarizes the per capita sludge build-up rates measured in the four case studies (for a discussion of these values see Section 6.7.7). It is proposed to use these as benchmark values for further field investigations.

Table 44: Per capita annual sludge build-up rates measured at the case study sites

		BWC	GB	MM	ST
Sludge increase ABR	$l\ cap^{-1}\ y^{-1}$	4.7	9.2	3.2	6.4
Sludge increase ABR & AF	$l\ cap^{-1}\ y^{-1}$		10.8	16.3	11.6

Another possibly robust indicator of the system's hydrolytic activity could be the biodegradable fraction of sludge VS. Sludge volume reduction occurs through the hydrolysis of organic particles represented by the biodegradable fraction of sludge VS. Since the non biodegradable VS is not affected by this a small biodegradable VS fraction could be used as an indicator for active hydrolysis[18]. This could be an interesting alternative to the measurement of sludge build-up rates to assess system activity in cases in which sludge washout cannot be excluded through field observations.

The validity of this method is supported by the model as shown in Figure 161: most model runs of a UA representing active sludge led to a biodegradable VS fraction below 50% whereas most model runs representing inactive sludge resulted in a biodegradable VS fraction above 50%. Future field investigations will be needed to confirm this relationship.

[18] This point is valid under the assumption that biodegradable and non-biodegradable VS have similar settling characteristics and one is not more prone than the other to being washed out.

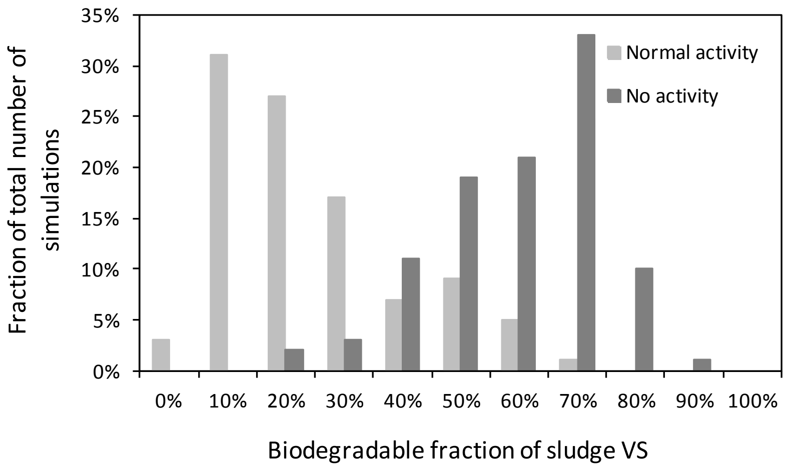

Figure 161: Biodegradable sludge VS fraction vs sludge activity, probability distribution as given by model uncertainty analysis, modelling runs done with GB data

7.5.2. Objective 2: Assessing treatment efficiency with model benchmark values for COD_s

This section discusses the model use to estimate benchmark COD_s effluent concentration values representing active anaerobic systems to compare field measurements against.

Monte-Carlo type uncertainty analyses were used to account for uncertainties in parameter and operating conditions. Figure 162 a to d present modelling results of the four case studies. Each data-point represents the result of one out of 100 modelling iterations during a Monte-Carlo type uncertainty analysis. The figures relate the modelled sludge increase assuming active anaerobic conditions to modelled effluent COD_s. The figures also indicate the measured sludge build-up rates (black dotted lines)[19] and the 95% confidence intervals of measured feed COD_s concentration means and effluent COD_s concentration means respectively (dark and dotted horizontal bands). The confidence intervals were computed with the standard errors of mean.

The uncertainty analyses always produced a number of implausible outcomes such as negative sludge accumulation rates resulting from unrealistic parameter value combinations. Sludge build-up field measurements have comparably little uncertainty associated to them and were therefore used to identify the relevant model outcomes representing field situations. Therefore those COD_s effluent concentration uncertainty analysis results were selected as benchmark values, which were associated to sludge build-up rates comparable to field measurements. In other words, model benchmark COD_s concentration ranges are shown on Figure 162 a to d where the line representing the measured sludge build-up intersects with the modelled build-up values. They are represented in Figure 162 a to d by the sparsely dotted horizontal bands.

All four case study benchmark value ranges for biodegradable COD_s effluent concentrations are therefore approximately 40 to 80 mg COD_s l^{-1}. The plant specific nonbiodegradable fractions

[19] The represented sludge build-up rates are the sums of the build-up rates observed in the ABRs and AFs. This is done under the assumption that the entire sludge accumulation occurring in an AF is due to ABR sludge washout during strong rains. These washout events are not reflected in the field concentration measurements which were always performed on dry weather days. Since the model predictions are based on dry weather data, they would therefore have to be compared to the combined build-up rates.

(100 mg COD_s l^{-1} for BWC, 20 mg COD_s l^{-1} for GB, MM and ST) inflate this value which explains the comparably high concentrations shown for BWC.

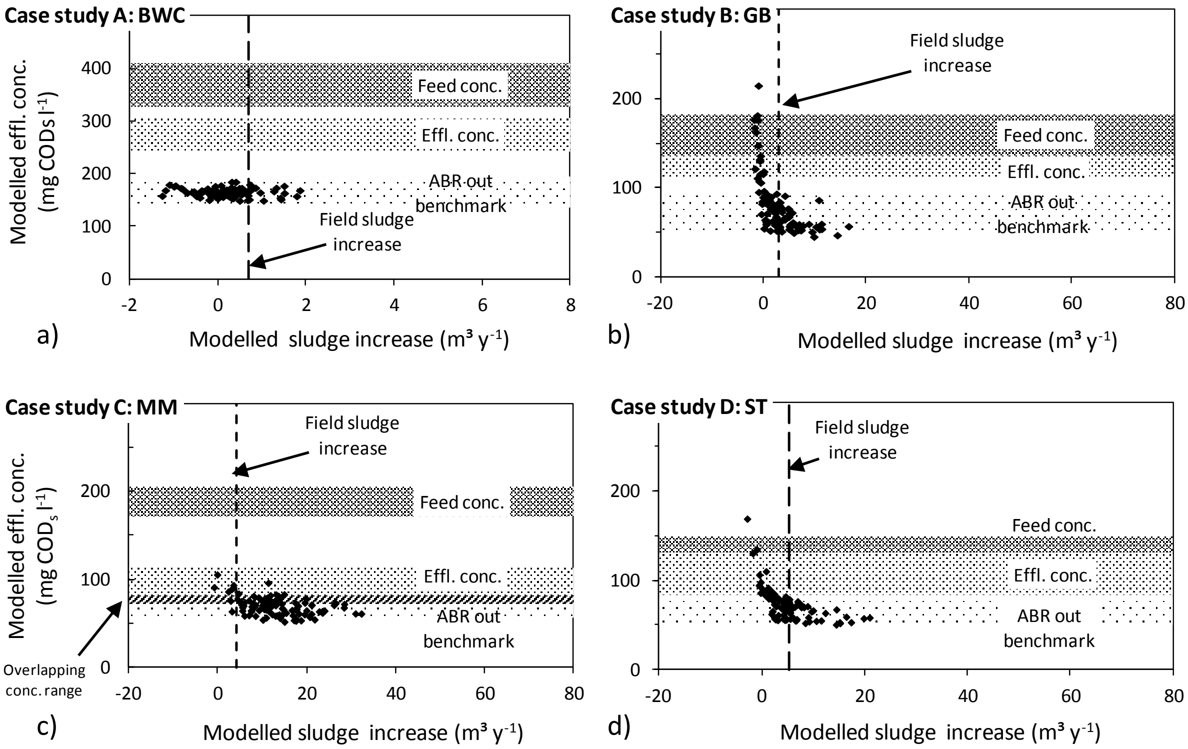

Figure 162 a, b, c and d: Modelled sludge increase representing active anaerobic treatment vs. effluent COD_s concentration. The red and blue horizontal bands represent the 95% confidence intervals of measured feed and effluent COD_s concentration means respectively, the grey horizontal band highlights the benchmark effluent COD_s concentration given by the model

The range of measured field concentrations in BWC, GB and ST are higher than the benchmarks provided by the model for systems operating with active sludge. This indicates that although the COD degrading processes in these systems were active to a certain extent, they did not reach the degree of activity as predicted by the model.

As opposed to the other three systems, the range of measured field concentrations in MM overlaps with the model COD_s concentration benchmark (Figure 162c) indicating treatment efficiency similar as that predicted by the model on that site. This is consistent with previous observations since MM operated significantly better than the other three systems with consistently best results for all considered treatment indicators (see Section 6.7.7). It was hypothesised that higher OLR at MM lead to more active biomass.

The sensitivity of the modelled effluent COD_s concentration towards kinetic rate constants was explored by varying the hydrolysis rate constant and the methanogen growth constant of the GB model setup such as shown in Figure 163. The model predictions were found to vary strongly when reducing the constants to 50% of their initial values. This questions the basic modelling assumption number 3 postulated in Section 7.3.1 that uncertainties concerning the applicability of the available calibration of kinetic rate constants may be neglected when drawing conclusions from the modelling results.

The hypothesis underlying this modelling exercise, that predictions from the current model calibration could be used as benchmarks for comparing effluent concentration measurements, was therefore refuted.

The current model calibration is based on data gathered at systems with far higher HRT and lower SRT than the ABR presented in this study in which anaerobic processes may differ significantly.

It is concluded that in order to produce truly meaningful predictions concerning COD_s reduction, the model, especially concerning methanogenesis rate constants, needs to be calibrated and validated with experimental data from systems with operation characteristics more comparable to an ABR.

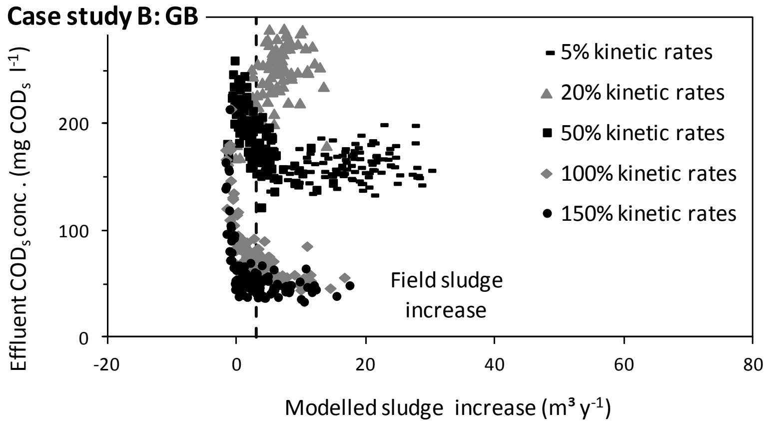

Figure 163: Sensitivity of the modelled effluent COD_s concentration towards the hydrolysis rate and methanogenesis growth rate constant, modelling runs done with GB data

7.5.3. Objective 3: Assessing effect of loading rate on treatment

Data from case study GB was used[20] for testing the hypothesis formulated for modelling objective 3 by assessing the effect of organic loading rate increase on reactor treatment efficiency.

Monte-Carlo type uncertainty analyses were conducted with varying ABR COD_s feed concentrations but otherwise identical parameter values. Each data point represents the outcome of one modelling iteration of which one hundred were performed per uncertainty analysis.

Figure 164 compares COD_s effluent concentrations and the mass of acetoclastic methanogens when setting the ABR_{in} COD_s concentration to 100%, 200% and 300% of the field value. This represents 159, 318 and 477 mg COD_s l^{-1} respectively. All other settings remain constant.

Feed concentration increase leads to a general rise of the acetoclastic methanogenic activity since the Xam mass in the system as well as the Xam VS fraction increases. The increase appears to be especially marked when doubling the feed COD concentration, less so when tripling it.

The high effluent COD_s concentrations at low mass of Xam in Figure 164 are certainly due to the methanogen concentrations being too low to process the available substrate. This effect appears lower for runs with higher feed concentrations since generally values for Xam increase.

The model further predicts a worsening of effluent quality with increased feed concentrations for runs in which case similar masses of Xam accumulate (see Figure 164). The modelling exercise representing

[20] Using case study GB appeared especially appropriate since it performed poorly in terms of COD_s reduction.

normal load yields effluent concentrations of about 60 mg COD_s l^{-1} when Xam masses accumulate to at least 8,000 g (see black dotted line in Figure). The two higher organic loading rates lead to effluent concentrations of approximately 80 mg COD_s l^{-1} and 110 mg COD_s l^{-1} respectively.

Published research results however document the resilience of the ABR treatment with effluent concentrations remaining constant even after considerable OLR increase (see Section 2.3.7). All authors attribute this operational stability to the compartmentalisation of the reactor. Since the model used in this study represents the complete ABR as one CSTR it does not reflect this characteristic. The current model therefore very probably underestimates the resilience of the ABR towards OLR variations.

Although the model predicts a worsening of the effluent concentration, higher feed concentrations do show a positive effect in terms of treatment efficiency (see Figure 168). Doubling the feed COD_s concentration improves the COD_s reduction considerably. Further feed concentration increase confirms the trend but leads to little further improvement.

The model therefore supports the hypothesis that an increase of ABR_{in} COD_s concentrations would generally lead to a more stable acetoclastic methanogen population and higher treatment efficiency.

Figure 165 compares COD_s effluent concentrations and the amount of acetoclastic methanogens when setting ABR_{in} COD_s and COD_p concentrations to 100%, 200% and 300% of the field values. This represents 159, 318 and 477 mg COD_s l^{-1} and 294, 588 and 882 mg COD_p l^{-1} respectively. All other settings remain constant.

The main difference to the runs in which only COD_s was increased appears to be that a higher Xam population establishes at higher loads, certainly due to the increased amount of MOs in the feed. Since this goes along with increased sludge and therefore VS build-up, increased load result in the decrease of the Xam VS fraction (Figure 167). The COD_s treatment efficiency significantly increases when doubling the initial COD_t load (Figure 169).

The model therefore supports the hypothesis that an increase of ABR_{in} COD_t concentrations would generally lead to a more stable acetoclastic methanogen population and higher treatment efficiency.

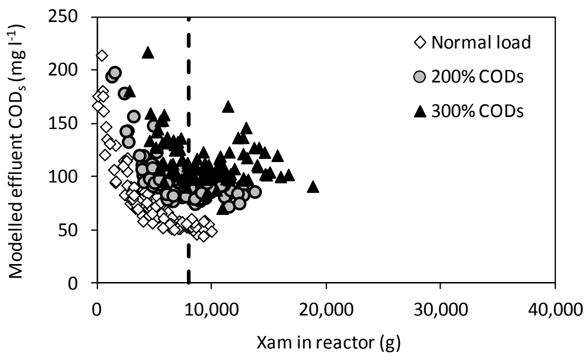

Figure 164: Xam in reactor at the end of each modelling iteration vs modelled effluent COD_s concentration depending on feed concentration

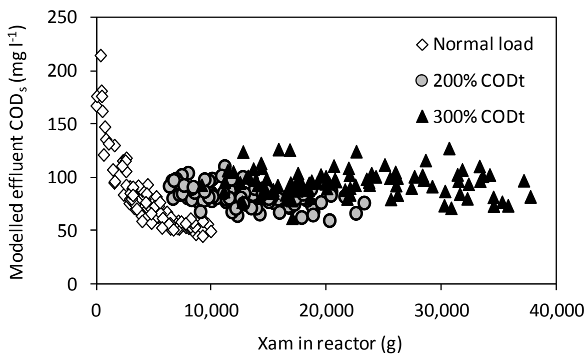

Figure 165: Xam in reactor at the end of each modelling iteration vs modelled effluent COD_t concentration depending on feed concentration

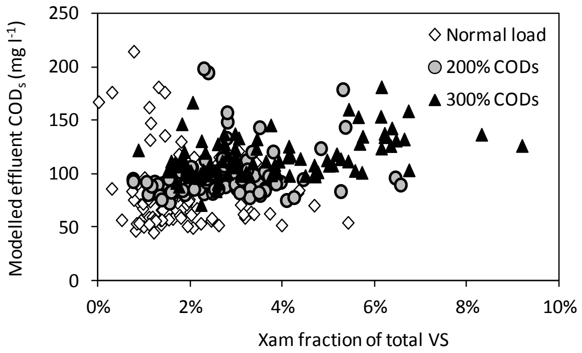

Figure 166: Xam fraction of total VS in reactor at the end of each modelling iteration vs modelled effluent COD_s concentration depending on feed concentration

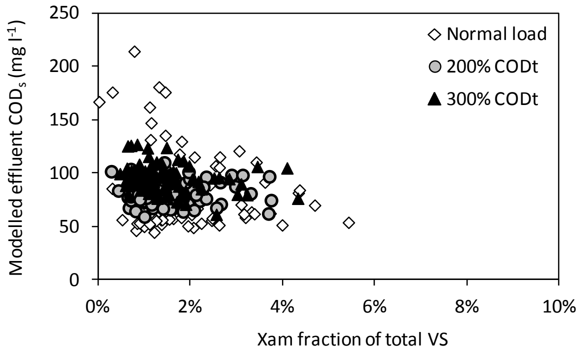

Figure 167: Xam fraction of total VS in reactor at the end of each modelling iteration vs modelled effluent COD_t concentration depending on feed concentration

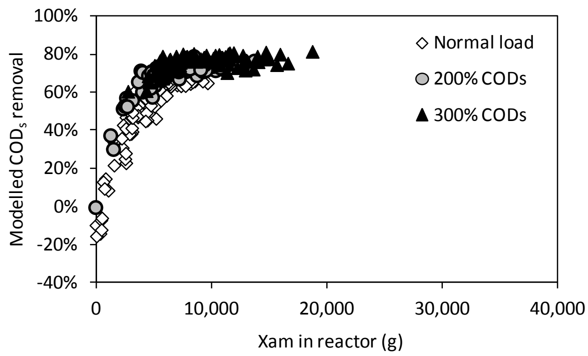

Figure 168: Xam in reactor at the end of each modelling iteration vs modelled COD_s removal depending on feed concentration

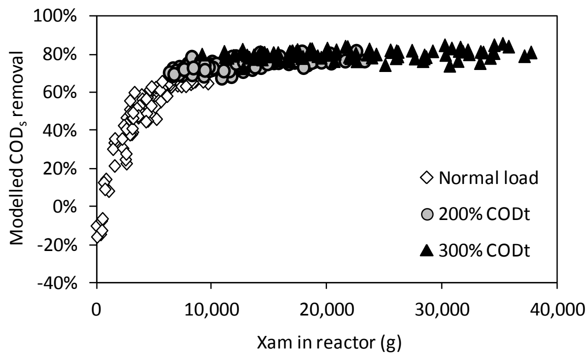

Figure 169: Xam in reactor at the end of each modelling iteration vs modelled COD_t removal depending on feed concentration

7.6. Conclusions

7.6.1. General ADM-3P model characteristics relevant to its use in this study

The ADM-3P model is used as the summarized representation of knowledge at the time of writing on the anaerobic digestion (AD) of communal wastewater. The strength of this approach lies in that it considers in great detail the kinetic and chemical aspects of AD of communal wastewater combined with the influence of the retention time. However, process-influencing factors more specific to ABR operation such as hydraulic particularities (effect of up-flow rate, mixing of wastewater with sludge), sludge characteristics (sludge settling speed, sludge accumulation and wash-out) and the reportedly strongly influencing compartmentalisation are not considered. In that respect the ADM-3P model represents a simplification. In addition the kinetic parameters were obtained from experiments conducted under very different conditions.

7.6.2. Using the model to help interpreting case study field data

A process model was developed integrating the existing ADM-3P model and its calibration for communal wastewater. This was done in order to approximate as far as possible benchmark values for the operational ABR parameters “sludge accumulation” and “effluent COD_s concentration”. Using the model in this manner supported the interpretation of scarce field data in the case of sludge accumulation but not in the case of effluent COD_s concentration.

7.6.2.1. Objective 1: Sludge accumulation

Modelled sludge accumulation rates were compared to field measurements in order to assess whether the latter indicate active or inactive hydrolysis of anaerobic systems. All modelling exercises supported the assessment that the observed sludge accumulation rates indicate active systems. This is valid under the assumption that sludge washout from the ABR was minor in all cases. This assumption is supported by field observations, since in three out of four cases little sludge was found in the rear reactor chambers of the DEWATS. Model sludge build-up predictions were found to be comparably insensitive to variations in hydrolysis rate values which increases confidence in the model benchmark.

7.6.2.2. Objective 2: Effluent COD_s concentration

Modelled effluent COD_s concentration value ranges were used as benchmarks for which to compare the measured field values. In general the model indicated that for the loading rates considered and when an active anaerobic environment establishes in the ABR, effluent biodegradable COD_s concentrations should never exceed 60 to 80 mg COD_s l^{-1}. However, model effluent concentration benchmark ranges were found to be sensitive to variations in hydrolysis rate and methanogen growth rate which strongly questions the validity of the used model benchmark predictions. The used model calibration is based on data gathered at systems with far higher HRT and lower SRT than the ABR presented in this study. In the systems used for calibration the hydrolysis was considered the main rate-limiting step. Since ABRs are operated at considerably lower HRTs and accumulate sludge leading to very long sludge retention times, processes may differ significantly.

It was noted that the benchmark range of the current model calibration corresponded reasonably well to the effluent COD_s concentrations measured at the best performing system (MM). However, since this system was known to have operated under extreme hydraulic conditions and with most probably impeded performance this does not represent a credible validation of the used calibration.

Future steps to improve the existing model by taking into account the ABR specific operation characteristics of low HRT and high SRT would certainly include the recalibration of the dissolved phase reaction rate constants. Estimating the required experimental efforts and prospect of success of such an endeavour were not part of this thesis but would certainly represent the next step for future model development.

7.6.2.3. Objective 3: Effect of OLR on treatment

The model does support the hypothesis that at constant hydraulic load, increase of the observed feed COD_s concentration and more so feed COD_t concentrations would lead to a greater mass of Xam and higher COD_s reduction. Conversely this means that the treatment efficiencies of the case study ABRs are limited by their low organic loading conditions.

The model also predicts the trend of the effluent COD_s concentration to increase with increased feed concentration. This result may not be accurate for communal ABR treatment and is contradicted by literature which reports stable effluent concentrations with increased OLR. The reason for this discrepancy may be that the model used here represents a CSTR and does not take into account the influence of ABR compartmentalisation.

7.6.2.4. Further conclusions drawn from the modelling exercises

The current model calibration was done with data from fully mixed digesters. The SRTs in these systems were considerably shorter and the HRTs considerably longer than in the case study ABRs presented in this study. The observed low sensitivity of the model output "sludge build-up" towards hydrolysis can therefore be explained by the high SRT of the ABRs allowing hydrolysis to run to completion. In the same way, the observed high sensitivity of the model output "COD_s concentration" towards methanogenesis is due to the comparably low HRT of the ABRs.

The implication of this is that, due to the long SRT, the hydrolysis may not represent the main rate-limiting anaerobic degradation step inside a communal ABR as opposed to the conventional view on anaerobic systems treating wastewater with high solid content. Future work will have to investigate which of the dissolved phase reactions is to be considered mainly rate-limiting.

7.6.3. Further applications of the process model concerning design and operation of ABR

Design engineers need to know the relationship between organic and hydraulic loading (including up-flow velocity) and the main ABR operation parameters sludge accumulation, effluent COD_s and COD_p concentration. The current model partially supports the understanding of these relationships.

7.6.3.1. Sludge accumulation and characteristics

The ranges of the modelled sludge accumulation rate for active anaerobic systems were large due to the considerable uncertainties associated with the input parameter values. By themselves these ranges were too inaccurate to provide estimates helpful for actual operation. The model output was however comparably insensitive to variations of the hydrolysis rate constant and was successfully used to validate existing field observations. As a further result, benchmark values for normalized sludge build-up representing at least partly active ABRs have been given.

Using sludge accumulation normalized over the number of connected users as a proxy for future sludge activity assessments certainly represents a very robust method applicable at a larger number of plants. A number of factors that need to be considered during such an assessment have been presented.

Another possibly robust indicator for system activity could be the biodegradable fraction of sludge VS. Modelling results indicated that active sludge should contain a significantly smaller biodegradable VS fraction (< 50%) than inactive sludge.

7.6.3.2. COD_s reduction

The current model calibration does not enable the prediction of COD_s effluent concentrations due to its high sensitivity towards methanogenic rate constants. The existing model calibration predicted effluent COD_s concentrations which were in reasonable accordance with the best performing case study. However, since this system was known to operate under extreme hydraulic conditions and with most probably impeded performance this does not represent a significant validation of the current calibration.

7.6.3.3. COD_p reduction

The COD_p effluent concentration is defined as an input parameter and can therefore inherently not be predicted by the current model. Nevertheless the COD_p effluent concentration ranges presented in this thesis reflect dry weather observations in practice and may contribute to reduction rate estimates in future design and modelling attempts.

7.6.4. Future investigations

The modelling exercises point towards a number of important future investigations in order to firmly establish some of the presented conclusions:

- The assumption that the effect of long term particulate washout from the systems is negligible has to be investigated with field experiments.
- The possibility of using sludge biodegradability as a proxy for future sludge activity assessments should be investigated by measuring and comparing both parameters on well monitored full-scale plants. This would include the identification and the testing of a robust sludge biodegradability measurement method. If this can be achieved this method would have the advantage over sludge build-up rate measurements through not being influenced by the difficult to observe particle washout. This method may also not require the access to all reactor chambers.
- The benchmark values presented for specific sludge build-up rates should be validated with observations on other well monitored full-scale systems operating under undisturbed conditions.
- The main rate limiting anaerobic sub-process in communal ABRs needs to be identified since it is probably not the hydrolysis.

Investigations concerning further future model development should deal with the question as to why the pH is significantly more sensitive to low alkalinity feed concentration in the model than in full scale reactors.

8. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

8.1. Observed design parameter values

Wastewater production measurements in several communities in central Java yielded an average per capita production of 81 l cap^{-1} d^{-1} with measured flows ranging from about 60 to 90 l cap^{-1} d^{-1}.

Long-term fluctuations in wastewater production of communities connected to DEWATS were found to be about 20%. The average diurnal peak-flow factor is 1.9 with a standard deviation of 20% across investigated systems and the strongest peak generally occurring in the morning for a duration of 2 to 3 h. Design assumptions for plants built in these regions are reasonably similar. The average monthly household income did not influence the flows since all visited communities had practically unlimited access to groundwater through shallow wells. Wastewater production in poor and water stressed sites in Bangalore/ India however was found to be as low as 30 l cap^{-1} d^{-1}.

Primary treatment effluent concentration measurements indicate that per capita organic loads are significantly lower than the generally assumed design value of 60 g BOD_5 cap^{-1} d^{-1}. The available data did not enable a direct quantification which will have to be made in future research. A more appropriate range so far suggested by the data is 20 to 40 g BOD_5 cap^{-1} d^{-1}.

Per capita nutrient loads were found to be similar to literature values. Effluent concentrations therefore mainly depend on the dilution through generated wastewater volumes. Approximate average concentrations of DEWATS anaerobic treatment step effluents were found to be 70 mg NH_4-N l^{-1} and 10 mg PO_4-P l^{-1}.

8.2. Factors limiting the performance of existing systems

8.2.1. Rain water intrusion

Field investigations have shown that large numbers of systems were exposed to severe flow surges during wet seasons. Such flow surges lead to up-flow velocities many times higher than assumed during design and dilute the feed wastewater probably over long periods of time. It is hypothesised that this caused the frequently observed sludge migrations across reactor chambers and significantly reduced methanogenic sludge activity in at least three Indonesian ABRs as observed during the wet-season 2013.

8.2.2. General under-loading

During a nationwide DEWATS survey in Indonesia, numerous systems loaded below design expectations featured surprisingly high effluent COD concentrations. High loaded systems had comparably low effluent concentrations, were however too few to allow strong conclusions to be drawn.

The highest loaded system of four case studies consistently showed the best results for the treatment efficiency indicators COD_t, COD_p and COD_s removal, number of chambers involved in significant removal, average SMA_{max} and per capita sludge accumulation.

The resulting hypothesis that ABRs operating under existing conditions do perform better with higher wastewater load goes in principle against the generally published view in literature that the HLR is the decisive treatment influencing operation parameters. The regular exposure to extreme flow surges may however have resulted in an increased resilience of the systems towards hydraulic loads therefore allowing comparably good treatment at high loads during dry weather periods.

8.2.3. Organic under-loading

Most SMA measurements indicate alternating activity strength across ABR chambers. Reactor chambers with high activity sludge are always followed by one or two chambers with significantly lower activities which are in turn followed by another chamber with increased activity. It is hypothesised that this phenomenon occurs due to general substrate limitation.

ABR feed concentrations in case studies were within the lowest applicable range for anaerobic digestion reported in literature. It is therefore hypothesised that treatment would improve with higher organic loading. Anaerobic modelling exercises confirmed this for increased COD_s and COD_t feed concentration.

8.2.4. Elevated raw-water salinity in coastal areas

Investigations on DEWATS across Java/Indonesia indicated a significantly higher salinity of raw-water at sites built close to the coast than at sites built inland. A large fraction of coastal plants had elevated effluent COD concentrations. It is therefore hypothesised that the treatment of these plants was impeded by raw-water salinity.

8.3. General performance of investigated DEWATS

8.3.1. Effluent concentrations

Measurements indicated guaranteed maximum concentrations of 200 mg COD_t l^{-1} for anaerobic DEWATS treatment effluent if the treated wastewater is non-saline which is significantly higher than design effluent concentrations. This however is based on systems of which the majority were hydraulically over-loaded for large parts of the year due to storm water intrusion. Furthermore, many systems were organically under-loaded. It is hypothesised that their treatment would improve significantly if their maximum hydraulic and general organic load was actually close to design.

Nutrient concentrations in the effluent of anaerobic DEWATS treatment steps are high and can exceed 100 mg NH_4-N l^{-1} and 15 mg PO_4-P l^{-1} in water-scarce areas. Per capita nutrient loads remained approximately constant across sites and in accordance to literature. Since no nutrient removal occurs inside anaerobic DEWATS reactors, effluent concentrations mainly depend on dilution and therefore on the per capita water consumption.

The average BOD_5/COD ratio of anaerobic treatment effluents measured at sixteen different DEWATS plants were 0.46 with a percent standard deviation of 38%. This ratio is high and indicates large fractions of biodegradable COD leaving the reactors untreated. Nonbiodegradable COD measurements performed on AF effluents confirmed this.

The time of day at which DEWATS effluent samples are drawn does not significantly influence the COD measurement outcome.

8.3.2. Digester and settler operation

The average HRTs of all case study pre-treatment steps were significantly larger than the value of 2 h proposed by Sasse (1998).

Plant feed concentration measurements were not part of this study. It was therefore not possible to directly assess the treatment efficiencies of the pre-treatment steps with the available data. The surprisingly low effluent concentrations measured in settler effluents indicate however that the pre-treatment design assumptions need to be revised. It appears that either the per capita organic loads were far lower or the pre-treatment efficiencies far greater than assumed.

Activity tests performed on sludge from three settlers indicated very low SMA in these reactors.

Operation of BGDs was not the primary focus of this study. One BGD however was monitored during the course of the investigations. Available data on effluent concentration and biogas production indicated a COD removal efficiency of at least 73%.

The measured average biogas production of communal DEWATS BGDs was 20 l cap^{-1} d^{-1} with a relative standard deviation of 36% across the eight systems on which measurements were performed.

No significant increase of per capita biogas production was observed with HRTs of above 2.5 d and it is proposed to use this value for the dimensioning of BGDs operating under DEWATS typical circumstances.

8.3.3. ABR operation

The average COD_t removal rates observed across the ABRs of three out of four investigated case studies were poor with 38%, 43% and 49%. Literature on laboratory scale systems and design procedures indicates a significantly higher expected removal of 65% to 90%. The ABR of the fourth case study DEWATS featured an average COD_t removal of 68% which is closer to the expected rate.

Field observations confirmed published laboratory investigations that most treatment occurs in the first two to three ABR chambers and little, if any, beyond.

Sludge accumulation rates observed in all four case study ABRs indicated good sludge stabilisation and therefore hydrolytic activity under the assumption that sludge washout during strong rain events was insignificant. This assumption is supported by the fact that little (if any) sludge accumulation was observed in most last AF chambers. The assumption will however have to be confirmed through long term solid washout measurements. The sludge accumulation rates were in all cases significantly lower than the rates predicted through particulate organics mass balances assuming simple accumulation in an inactive system. This was further confirmed through anaerobic modelling.

Sludge activity measurements indicated uneven SMA distribution across ABR chambers with the highest activity usually in the first chambers.

The fact that most active sludge established in the first ABR chambers indicates that these should never be desludged. Based on the available data, previously estimated desludging intervals of 2 y to 3 y could be extended to at least 4 y. Settlers will certainly require more frequent desludging. It should be investigated whether sludge transfer from settler-chambers into ABR-chambers is feasible when the settler is full in order to reduce the frequency of total plant desludging.

8.3.4. AF operation

The AFs of all three case studies significantly reduced COD_p and COD_s concentrations to levels the ABRs appeared unable to. In two cases the AFs were the only DEWATS reactors reducing COD_s concentrations at statistically significant levels. With 25% to 50% COD_t reduction none of the AFs however reached design and literature treatment expectations.

The effluent BOD_5/COD ratio of the last anaerobic treatment step (AF) was determined for three of the case studies and yielded 0.58, 0.68 and 0.77 respectively. These ratios are very high and indicate large fractions of biodegradable COD leaving the reactors untreated. Nonbiodegradable COD measurements performed on AF effluents confirmed this, inferring that better removal rates, especially regarding COD_s, may be possible.

Sludge accumulation measurements indicated that the AF growth media acted as sludge retention devices for sludge washed out of the ABR chambers due to storm water intrusion. SMA measurements in all cases yielded very little methanogenic activity of the sludge accumulated at the bottom of AFs.

8.4. ABR treatment modelling with ADM-3P

The ADM-3P model with an existing calibration was used in an attempt to create benchmark value ranges for the operational parameters “sludge build-up” and “effluent COD_s concentration” in order to interpret field data.

It became apparent during the modelling exercise that the existing model calibration is not appropriate for the benchmark value range creation for the operational parameter “effluent COD_s concentration”. The current model calibration is based on the assumption that hydrolysis represents the rate-limiting step which may not be correct for a solid-accumulating system such as the ABR. Future investigations will have to investigate which of the soluble phase reactions actually represents the mainly rate-limiting sub-process inside an ABR and the experimental effort needed for a more appropriate calibration in order to assess the future use of the ADM-3P model in such a context.

The existing model calibration predicted effluent COD_s concentrations which were in reasonable accordance with the best performing case study. However, since this system is known to have operated under extreme hydraulic conditions and with most probably impeded performance this does not represent a significant validation of the current calibration.

The existing model calibration was however successfully used to identify observed sludge accumulation rates in four case studies as representing an active hydrolytic system. It is therefore suggested to use the observed rates as benchmarks for future investigations.

8.5. Implications of findings on future design

8.5.1. Higher system loading than currently assumed may be possible

Plants loaded above design expectation performed well and modelling indicated that ABR treatment efficiency increases with increased organic load. Also, the fact that active sludge was able to establish inside all case study ABRs despite the extreme hydraulic loads these were exposed to, indicates that higher hydraulic loads may be tolerated by the system. $v_{up,max}$ values exceeding the existing design value of 1 m h^{-1} therefore appear possible. It is proposed to build and test an ABR prototype operated with 2 m h^{-1} $v_{up,max}$ (which corresponds to 1 m h^{-1} $v_{up,mean}$).

8.5.2. Controlling the feed

The above mentioned conclusions imply that engineering solutions have to urgently be found in order to limit the feed-flow to the maximal design value during rain events and to increase the organic concentration of the raw wastewater. Appendix A5 presents a technical concept on how to include a storm water overflow system to the DEWATS design which may solve some of the associated technical difficulties.

Increased feed concentration may be achieved by diverting parts of the grey-water from the community to a separate percolation bed.

At the same time it would be strongly advisable to reduce the nutrient content of the DEWATS-feed in order to limit the discharge of strongly eutrophic nutrients to recipient water bodies. Since the largest nutrient source in communal wastewater is the urine, urine-diversion combined with reuse or onsite-percolation appears to be the obvious solution. Factors to consider for the urine percolation will be soil type, local groundwater dynamics and minimum distances to existing shallow wells. Also the pH-stabilizing effect which urine has on the anaerobic treatment will have to be taken into account.

8.5.3. Proposed future DEWATS reactor setups

The above mentioned results imply an optimum DEWATS reactor setup which includes a pre-treatment step followed by a four chamber ABR and a two to three chamber AF. It is proposed to reduce the size of the settler to an HRT below 10 h in order to increase the organic load to the ABR. It is further suggested to double the size of the fourth ABR chamber in order to reduce the up-flow velocity inside it and improve its solid retention. The effluent from the ABR to the AF should further remain as solid-free as possible which could be achieved by including a small lamella clarifier before the effluent.

8.6. Implications of findings on future treatment monitoring methods

8.6.1. Estimating sludge activity

Sludge activity investigations are crucial for the monitoring and evaluation of DEWATS reactor performance. There is however little experience available on this topic. Two approaches were used and documented in this thesis. They both have shown to produce meaningful qualitative results. Both methods identified independently the same system with the highest sludge activity of all four. This result was in accordance with the other available treatment efficiency indicators COD_t, COD_p and COD_s removal and number of chambers involved in significant removal.

SMA measurements are cheap and not difficult to conduct. They enable the comparison of the acetoclastic activity across the chambers of an ABR and the assessment of changes over time and over changing operational conditions. They require:

- the ability to perform sludge-VS measurements
- the ability to perform the SMA measurement within one week after sampling
- the ability to store the sludge samples at a temperature of 2°C to 6°C
- skilled laboratory and field staff or close supervision during sampling and the experiment

Based on the measurements presented here, a benchmark value of 0.2 g COD g VS^{-1} d^{-1} is proposed for methanogenically active ABR sludge.

Per capita sludge accumulation is considered a very robust indicator because it represents the integrated loading history of the plant as opposed to point in time stream measurements and sludge activity investigations. It requires the ability to:

- measure the sludge heights in all ABR and AF chambers
- access trustworthy information on the operation history of the plant (especially on desludging)
- assess the number of connected people
- check the reticulation system for severe blockages and breaks

Based on the measurements presented here, a benchmark value of 3 l cap^{-1} y^{-1} is proposed for hydrolytically active ABR sludge. The method is based on the assumption that long-term solid washout from the ABR is negligible. Although field observations support this, measurements will have to be conducted in future to confirm.

Anaerobic treatment modelling further indicated that a low biodegradable VS fraction of accumulated ABR sludge may be used as an indicator for high hydrolytical sludge activity. This indicator would have similar advantages to the “per capita sludge accumulation” since it would also represent the cumulated plant loading history. It may not require access to all reactor chambers and to information about the true number of connected people which at times may be difficult to obtain. The adequate measurement methodology however still needs to be identified and tested for robustness. The indicator would further have to be tested on several systems of varying sludge activity in order to validate this method.

8.6.2. Further helpful parameters

EC measurements are cheap, very easy to perform during field investigations and can provide useful information on wastewater dilution through rain when done regularly at the same site.

Turbidity measurements have the same advantages (low costs, simplicity) and were found to be very helpful in monitoring changes in particle retention throughout the reactors when done regularly. COD_p measurements allow a direct quantification of particulate organics but are much more prone to errors and produce far more erratic data which can be difficult to interpret on their own.

8.7. Future research needs

This study was unable to directly examine the correctness of the existing DEWATS design procedure since all investigated systems were affected by storm water and most were under-loaded. Also, the crucial question about the maximum loading rate tolerated by these systems remains unanswered. *It is therefore absolutely essential for the thorough understanding of DEWATS reactors to conduct future research on highly loaded full-scale systems which are not storm water affected. It is strongly recommended to investigate several systems at once in order to minimize the dependency of research outcomes on the correct operational environment of only one system.*

It is suggested to continue the monitoring of all four case studies presented in this thesis in order to consolidate the existing data-set and the here presented conclusions and in order to document future operational changes.

It is further suggested to conduct detailed plant feed concentration measurement campaigns at a minimum of two sites in order to quantify the per capita COD production and to verify the estimations presented here.

The data gathered at the case study BWC during operational Phase II did not allow strong conclusions to be drawn at the time this thesis was written. It did however indicate an increase in overall ABR treatment with increased organic loading. Confirmation of this could be gained by upholding the operational conditions and continuing system monitoring. A suggested future monitoring schedule for BWC has been detailed in Section 6.8.5.

The AF chambers of the case study system MM should be completely desludged in order to measure the subsequent sludge washout from the ABR and validate the here presented sludge accumulation values.

Future in-depth investigations at the here presented case studies should put their emphasis on:

- hydraulic load
- SMA
- long term solids washout of systems
- the biodegradable VS content and VS fraction of DEWATS sludges
- the soluble organic fractions of supernatants and effluent in order to gain better insight on the rate limiting anaerobic sub-processes

It is also advisable to, at least partly, repeat the Indonesian-wide survey presented in this thesis in order to consolidate the available data. Effluent COD investigations should include fractionated COD measurements, performed as multiple measurements, if possible on different days. EC measurements should be performed on samples taken from a representative number of wells and other water sources used by one community. A number of research questions arose from the observations made using the currently available data. They were formulated as hypotheses that should be further investigated with the future consolidated dataset:

- Elevated raw-water salinity affects the treatment of DEWATS.
- Elevated raw-water salinity affects the treatment of low loaded DEWATS more than higher loaded plants.
- High loaded plants perform better than normal loaded plants.

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10. APPENDIX A1: NATIONAL DISCHARGE STANDARDS

Table 45: National discharge standards of selected countries

Country	Discharge to	pH$	BOD$_5$$	COD$	TSS$	TDS$	Grease and oil$	NH$_4$-N$	PO$_4$-P$	Reference
Cambodia	Public water area and sewer	5 to 9	80	100	80	2000	15	7*	2	Cambodia (2014)
Germany	Water bodies+	n.a.	40	150	n.a.	n.a.	n.a.	n.a.	n.a.	Germany (2014)
India	Inland surface water	5.5 to 9	30	250	n.a.	n.a.	10	100*	5	India (2014)
Indonesia	Water bodies	5 to 9	100	n.a.	100	n.a.	n.a.	n.a.	n.a.	Indonesia (2014)
Lao	Wastewater discharge standard from Urban area	6 to 9.5	50	150	n.a.	2000	20	40*	n.a.	Lao (2014)
Philippines	Inland waters	6.5 to 9	50	100	70	n.a.	5	n.a.	n.a.	Philippines (2014)
South Africa	Inland waters	5.5 to 9.5	n.a.	75	n.a.	n.a.	n.a.	4.5	3	Africa (2014)
South Africa	Agricultural areas when discharge is < 5 000 m³ ww d^{-1}	6 to 9	n.a.	400	n.a.	n.a.	n.a.	n.a.	n.a.	Africa (2014)
Tanzania	Water bodies	6 to 8.5	30	60	100	n.a.	n.a.	n.a.	n.a.	Tanzania (2014)
Vietnam	Water bodies not used for domestic water supply	5 to 9	50	n.a.	100	1000	n.a.	10*	10	Vietnam (2014)

$ in mg l^{-1}, * specified as N Kjeldahl, + from small scale wwtp treating less than 60 kg BOD$_5$ d^{-1}

11. APPENDIX A2: SPECIFIC METHANOGENIC ACTIVITY (SMA) METHODOLOGY TESTING RESULTS

11.1. General

The Specific Methanogenic Activity (SMA) test investigates the acetoclastic methanogenic activity of an anaerobic sludge by measuring the amount of CH_4 produced by a known amount of sludge (expressed as VS) under ideal substrate (acetate) saturated conditions. It is expressed as „ml CH_4 (as COD-equivalents) g VS^{-1} d^{-1}".

Acetoclastic methanogenic activity accounts for up to 70% of the methane production in the anaerobic digestion of communal wastewater and for most of the conversion of COD (Seghezzo, 2004). Since methanogenesis represents the last and often most sensitive step in the chain of anaerobic digestion processes, the SMA of a sludge is often used as an indicator for its general anaerobic activity (Souto et al., 2010).

There is no existing standard SMA method. The tests presented in this section were performed in order to test and adapt an existing methodology (Pietruschka, 2013) to DEWATS sludge and to estimate the error associated with this measurement.

Another very common test, the Biochemical Methane Potential (BMP) test, used to study the degradation of a substrate, can be performed with a very similar methodology as the SMA. It is however important to realize that its objective is very different: the BMP test studies the properties of a substrate, whereas the SMA investigates the properties of a sludge.

11.2. Methodology

11.2.1. General information

The SMA setup was used as described in Pietruschka (2013) (see Figure 170 and Figure 171) and consisted of a reactor bottle, containing a known amount of sludge with a known amount of substrate, which is connected to a displacement bottle. The reactor bottle was not stirred which may have led to mass transfer problems. The NaOH solution inside the displacement bottle bound all the CO_2 from the accumulating biogas and therefore allowed the direct determination of the CH_4 production (Souto et al., 2010). The reactor bottle temperature was regulated by a temperature controlled water-bath at 35°C. All measurements were done using NaAc as substrate, in triplicate and with triplicate controls. The processed sludge samples were starved at 35°C for 24 h prior to the experiment in order to remove residual substrate from the sample liquor.

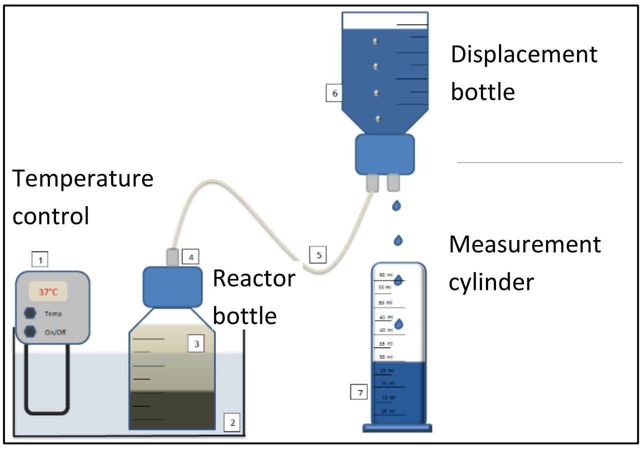


Figure 170: Conceptual representation of the SMA setup with temperature controlled water-bath, reactor bottle, displacement bottle and measurement cylinder, adapted from Pietruschka (2013)

Figure 171: SMA setup in Yogyakarta with twelve displacement bottles and measuring cylinders, water-bath with temperature control containing the reactor bottles is in the background

Sludge was sampled on-site with a Plexiglas core-sampler. All settled sludge heights were recorded enabling the calculation of the sampled settled sludge volume. The content of the core-sampler was then decanted in order to remove most wastewater from the sample. The exact sample volume after decanting was recorded in order to determine the dilution of settled sludge by wastewater. All solid determinations and SMA tests were done using homogenised aliquots of these samples.

SMA tests should be performed with constant VS reactor content. Sludge volume was chosen rather than sludge VS-mass in order to simplify the procedure. In practice there is insufficient time to measure the VS content of the sludge before running the SMA experiment. The sludge was always decanted to the point where its viscosity was liquid enough to enable easy handling. This generally represented a VS concentration of approximately 35 g VS l^{-1}.

Sludge samples were stored between 2°C and 6°C without being exposed to light.

Specific methanogenic (acetoclastic) activity was determined from the data as described by Soto et al. (1993):

Following Soto et al. (1993) maximum SMA (SMA_{max}) should be determined on the linear section of the cumulative methane production curve during the first hours of the experiment, when VFAs are still high, kinetics are therefore substrate saturated and the influence of other processes can be considered negligible. Cho et al. (2005) defines the SMA_{max} as the peak on a SMA vs. time plot. Accordingly, SMA is expressed as „ml CH_4 (as COD-equivalents) g VS^{-1} d^{-1}".

SMA tests with liquid displacement such as presented in Pietruschka (2013) are reported to be accurate for sludge activities above 0.05 g COD g VS^{-1} d^{-1} (Soto et al., 1993).

All solid measurements presented in this section were performed at the Gadjah Mada University, Analytical Chemistry university laboratory, Yogyakarta. They were done as triplicates with standard deviations of 0.2% to 4.1% and 1.9 to 8.4% for TS and VS respectively. Sextuplicate measurements taken initially to assess the accuracy of the method showed standard deviations of 1.7% and 1.4% for TS and VS respectively. Tests were done on sludge samples from ABR 1/ Minomartani. At first the analytical balance was suspected to be a source of large error due to its age and exposure to unskilled staff. A theoretical balance-inaccuracy of 5 mg however only leads to a standard deviation of 0.9% and 1.7% for the sextuplicate TS and VS measurements respectively. The balance was manually calibrated before each set of measurements and is considered to have an error far smaller than 5 mg.

11.2.2. Calculations and data-processing

The factor f_{bg} which represents the COD value of wet CH_4 volume unit at 20°C is 1/385 g COD ml CH_4^{-1} (Soto et al., 1993). Following the Ideal Gas Law, this leads to a factor of 1/396 at 28°C and sea-level, which is representative for measurements in Yogyakarta and 1/445 at 28°C and 950 m altitude which is representative for measurements in Bangalore.

SMA-values are represented as moving averages of recorded CH_4-volume production over 4 h by using the data-points 2 h before and after the respective time point. This was done in order to reduce the influence of short term fluctuations in the gas-production and therefore determine more representative SMA values.

11.2.3. SMA in literature

Table 46 includes SMA measurement outcomes in available literature showing the reactor type, wastewater type and the substrate used in the tests. Previous tests performed by Pietruschka (2013) on ABR sludge indicate similar or slightly lower SMA than reported in other publications on processes treating communal wastewater. However it must be born in mind that a large variation of methodologies in SMA investigations has been reported which makes direct comparisons difficult. Pietruschka (2013) proposed to use tests run with a high performance anaerobic sludge from a full scale UASB reactor treating brewery wastewater as benchmark for ABR tests.

Table 46: SMA measurement outcomes in literature

Author	Reactor type where sludge originates from	Wastewater type	Substrate used in test	Measured SMA_{max} g COD g VS^{-1} d^{-1}
Hutnan et al. (1999)	Bench-scale ABR	Synthetic complex	NaAc	0.7 – 1.0
Colleran et al. (1992)	Full-scale digester	Sewage	NaAc	0.13
Soto et al. (1993)	Full-scale digester	Process water mussel factory	VFA mixture	0.81
Ince et al. (2001)	Lab-scale UASB	Pharmaceutical wastewater	Ac	0.18
Sorensen and Ahring (1993)	Lab-scale digester	Hh solid waste	Ac	0.05
de Lucena et al. (2011)	Full-scale UASB	Communal wastewater	VFA mixture	0.34
Moussavi et al. (2010)	Upflow septic tank	Communal wastewater	Ac	0.07
Moussavi et al. (2010)	Upflow septic tank	Communal wastewater	Communal wastewater	0.04
Souto et al. (2010)	Bench -scale UASB	Communal wastewater	Ac	0.08
Souto et al. (2010)	Bench -scale UASB	Communal wastewater	Communal wastewater	0.07
Castro et al. (2002)	Anaerobic lagoon	Yeast producing factory	Ac	0.2
Pietruschka (2013)	Full-scale ABR – NLM chamber 1	Communal	NaAc	0.05
Pietruschka (2013)	Full-scale ABR – NLM chamber 2	Communal	NaAc	0.01
Pietruschka (2013)	Full-scale UASB	Brewery wastewater	NaAc	0.21

Author	Reactor type where sludge originates from	Wastewater type	Substrate used in test	Measured SMA_{max} g COD g VS^{-1} d^{-1}
Nasr et al. (2009)	Bench-scale ABR	Communal wastewater	VFA mixture	0.05 – 0.1

11.3. Effect of varying substrate to inoculum (S/I) ratio

The substrate to inoculum (S/I) ratio has a strong influence on the SMA test outcome (Cho et al., 2005; Souto et al., 2010). Best S/I-ratio is reported to be 0.4 to 0.6 g COD g VS^{-1} (Cho et al., 2005) and 0.125 to 0.75 g COD g VS^{-1} (Souto et al., 2010) with NaAc as substrate.

The effect of substrate and inoculum concentration on the CH_4 production was tested in order to find the best combination for DEWATS sludge. A good ratio should lead to linear gas production, minimal lag-phase and minimal standard deviation between multiple runs. The linear gas production is an indication of substrate saturated kinetics and a prerequisite to identify SMA_{max} of a sludge (Soto et al., 1993). Insufficient substrate addition could lead to only a very short period of linear gas-production or no linear gas production at all. Too much substrate on the other hand could shock the sludge which would lead to a period of adaptation or lag-phase in the biogas production.

11.3.1. Varying substrate concentration

Table 47 shows the details of an experiment in which the amount of inoculum was kept constant with substrate concentrations representing 0.25, 0.5 and 1 g COD l^{-1}. Technical details about the DEWATS plant can be found in Section 6.5.2.

Table 47: Experimental details, variation of substrate concentration with constant amount of inoculum

Plant	MM	MM	MM
Sampling point	**ABR 1**	**ABR 1**	**ABR 1**
Date of sampling	12.02.2013	12.02.2013	12.02.2013
Date of measurement	13.02.2013	13.02.2013	13.02.2013
Time between measurement and sampling (d)	1	1	1
Inoculum volume in bottle (l)	0.15	0.15	0.15
Conc of pure inoculum (g VS l^{-1})	31.6	31.6	31.6
Inoculum in bottle (g VS)	4.74	4.74	4.74
Substrate conc in bottle (g COD l^{-1})	0.25	0.5	1
Calc. max CH_4 prod (ml CH_4 g VS^{-1})	5.1	10.2	20.3
S/I ratio (g COD g VS^{-1})	0.01	0.03	0.05

As seen on Figure 172 all three cumulative CH_4-production curves show a similar linear gradient at the beginning of the test run. The two lower substrate concentration curve slopes become non-linear and the rate decrease after approximately 7 h indicating substrate limitation. In both cases all added substrate was depleted after approximately 45 h. 1 g COD l^{-1} substrate concentration on the other hand induced a much longer period of substrate saturation and therefore linear cumulative CH_4-production. The proposition in Pietruschka (2013) that 1 g COD l^{-1} as substrate concentration should be used was therefore confirmed for DEWATS-sludge in Indonesia. Data in the same work had previously indicated that 2.5 g COD l^{-1} substrate concentration leads to long lag-phases before CH_4 production, which is possibly due to process-inhibition.

The stoichiometrically calculated CH_4 productions for the different amounts of added substrate (NaAc) are 5, 10 and 20 ml CH_4 g VS^{-1} for 0.25, 0.5 and 1 g COD l^{-1} respectively. As can be seen on Figure 172, most of the production curves asymptote towards the respective values during the second halves of the experiments, indicating good data consistency. The reason for the slight excess CH_4 production during the run with 1 g COD l^{-1} was not identified.

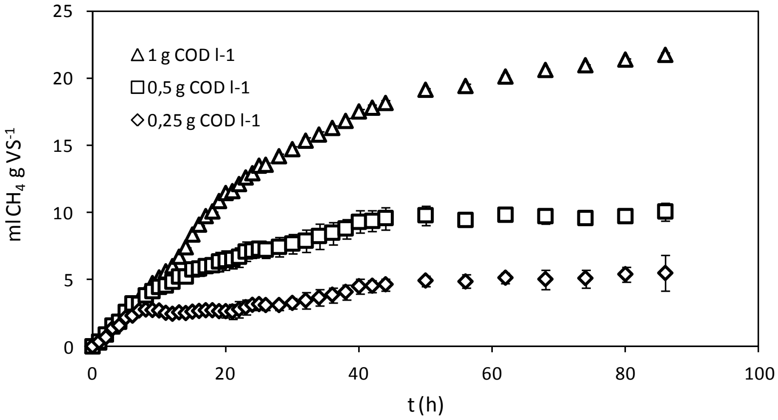


Figure 172: Cumulative CH_4 production at constant inoculum (sludge) volume (150 ml) and varying substrate concentrations, the theoretical maximal CH_4 productions for the different amounts of added substrate (NaAc) are 5, 10 and 20 ml CH_4 g VS^{-1} for 0.25, 0.5 and 1 g COD l^{-1} respectively, data points are averages of triplicates and control has been subtracted, error-bars indicate the sum of standard deviations of triplicate tests and triplicate controls, sludge sample: ABR 1, Minomartani

11.3.2. Varying inoculum volume

Table 48 shows the details of two experiments in which the substrate concentration is kept constant at 1 g COD l^{-1} and the inoculum volume is varied. Both experiments, although preformed with the same sludge, are not directly comparable because of different sludge storage periods.

Table 48: Experimental details, variation of inoculum volume with constant substrate concentration

	Figure 173a and Figure 174a		Figure 173b and Figure 174b	
Plant	**MM**	**MM**	**MM**	**MM**
Sampling point	**ABR 1**	**ABR 1**	**ABR 1**	**ABR 1**
Date of sampling	21.02.2013	21.02.2013	21.02.2013	21.02.2013
Date of measurement	22.02.2013	22.02.2013	02.03.2013	02.03.2013
Time between measurement and sampling (d)	1	1	9	9
Inoculum volume in bottle (l)	0.1	0.15	0.15	0.2
Conc. of pure inoculum (g VS l^{-1})	28.02	28.02	28.02	28.02
Inoculum in bottle (g VS)	2.80	4.20	4.20	5.60
Substrate conc. in bottle (g COD l^{-1})	1	1	1	1
S/I ratio (g COD g VS^{-1})	0.09	0.06	0.06	0.04

Figure 173a shows less of a deviation from linear CH_4 production at the higher inoculum volume leading to an S/I ratio of 0.06 and less standard deviation between triplicates, especially after 20 h of test run.

Figure 173b shows no obvious difference between 150 and 200 ml inoculum representing 0.06 and 0.04 S/I ratio.

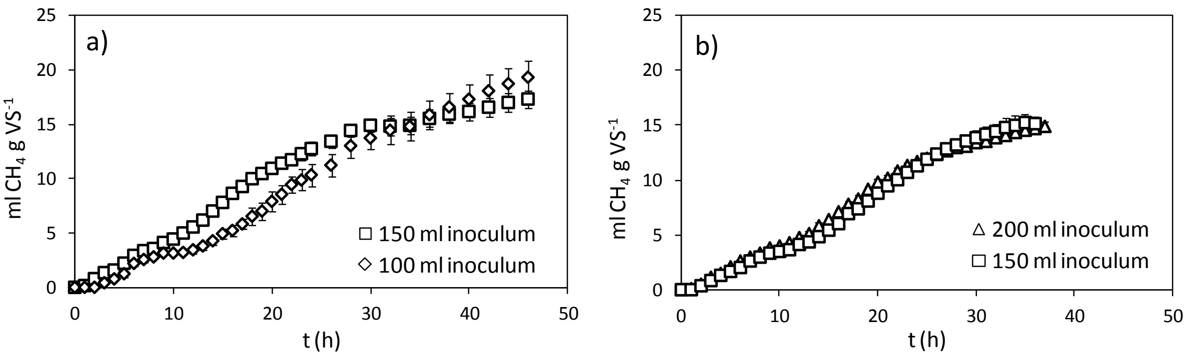


Figure 173 a and b: Cumulative CH_4 production at constant substrate (NaAc) concentration (1 g COD l^{-1}) and varying inoculum (sludge) volume, data points are averages of triplicates (except for the 150 ml sludge concentration curve on Figure 173b: duplicates) and control has been subtracted, error-bars indicate the sum of standard deviations of triplicate tests and triplicate controls, the theoretical maximal CH_4 production is 20 ml CH_4 g VS^{-1}, sludge sample: ABR 1, Minomartani

Figure 174a shows a similar maximum SMA of approximately 0.045 g COD g VS^{-1} d^{-1}, with the 100 ml inoculum curve however shifted by approximately 5 h. Figure 174b on the other hand indicates a lower time-lag between both curves and the same maximum activity after approximately 20 h.

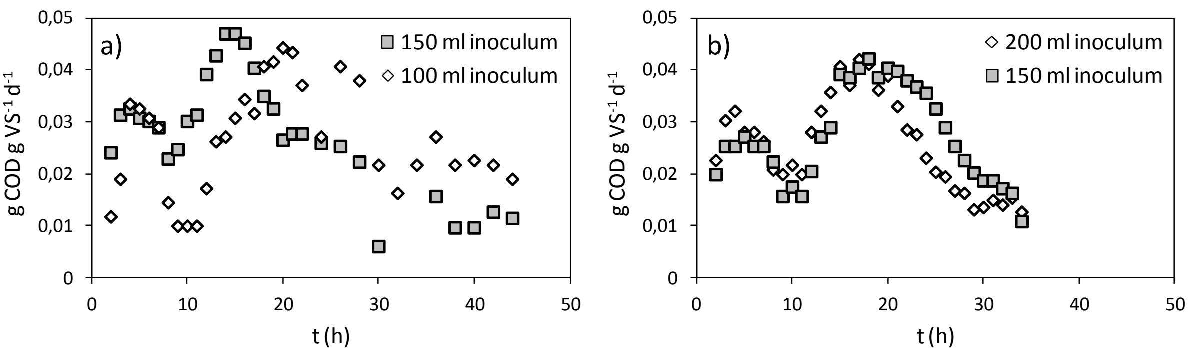


Figure 174 a and b: SMA curves of the experiments depicted in Figure 173, each data point represents the moving average over 4 h (t_i ±2 h)

11.3.3. Conclusion

SMA tests should be done with 150 ml sludge since, with the setup used, this was shown to be the least amount of sludge needed to produce near-linear cumulative curves of small value for the standard deviation of triplicates. 1 g COD l^{-1} substrate concentration was found to be adequate for DEWATS ABR sludge.

11.4. Effect of applying a second dose of substrate

11.4.1. Testing

Cho et al. (2005) found during their SMA tests that sludge should be allowed to stabilize and adapt to the substrate during a first test run and to assess the actual SMA with a second or third dose of substrate. Experiments with three different inoculums are presented here which were run with a second consecutive addition of substrate after 40 h test duration. Experimental details are presented in Table 49.

Table 49: Experimental details, effect of applying a second dose of substrate

	Sludge 1	Sludge 2	Sludge 3
Plant	**ST**	**ST**	**MM**
SP	**ABR 4**	**ABR 5**	**ABR 5**
Date of sampling	08.03.2013	08.03.2013	12.02.2013
Date of measurement	14.03.2013	14.03.2013	06.03.2013
Time from measurement to sampling (d)	6	6	22
Inoculum volume in bottle (l)	0.15	0.15	0.15
Conc. of pure inoculum (g VS l^{-1})	35.1	40.9	40.9
Inoculum in bottle (g VS)	5.3	6.1	6.135
Substrate conc. in bottle (g COD l^{-1})	1	1	1
S/I ratio (g COD g VS^{-1})	0.05	0.04	0.04

Figure 175 shows the cumulative methane production curves of the three runs detailed in Table 49. The experiment with ABR 5 sludge from Minomartani shows no significant difference in the average cumulative gas production induced by substrate. However sludge 1 and sludge 2 reacted differently to the second substrate addition with a much faster decline of activity than during the first 20 h of the experiments. It could generally be observed in all runs that the standard deviation of triplicate runs increased significantly after the second consecutive substrate addition. This was especially the case during saturated or near-saturated conditions during the hours after substrate addition.

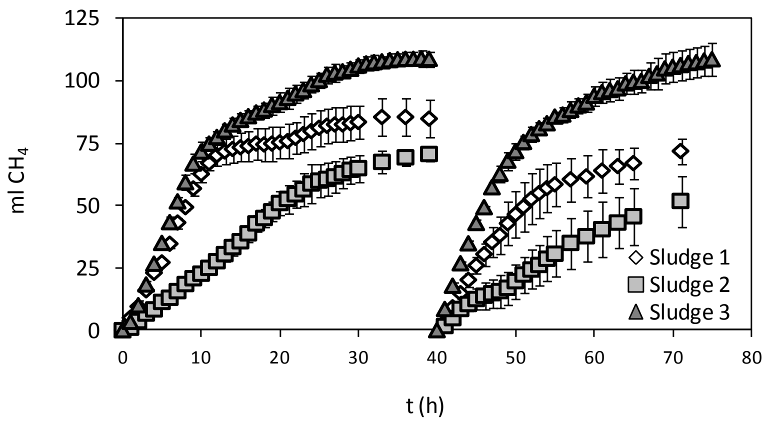


Figure 175: Cumulative CH_4 production with second substrate addition after 40 h, data points are averages of triplicates and control has been subtracted, error-bars indicate the sum of standard deviations of triplicate tests and triplicate controls, the theoretical maximal CH_4 production is 100 ml CH_4, sludge samples: ABR 4 and ABR 5, Santan, ABR 5, Minomartani

The SMA curves in Figure 176 confirm the observations made above. The SMA of sludge 3 was essentially the same during the two halves of the experiment with a slightly higher SMA_{max} a few hours after the second substrate addition. A clear decrease of SMA_{max} was noticed with sludge 1.

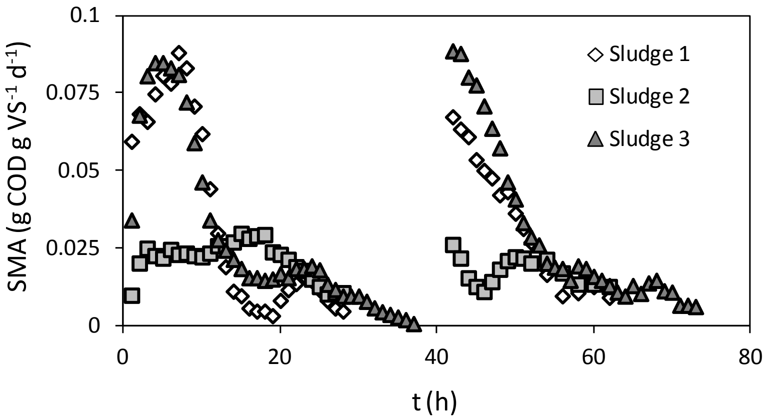


Figure 176: SMA curves of the experiments depicted in Figure 175, every data point represents the moving average over 4 h (t_i ±2 h)

11.4.2. Conclusion

SMA_{max} of DEWATS sludge should be determined with only one substrate addition. The hypothesised sludge adaptation to the substrate during a preliminary exposure to the substrate was not observed with DEWATS sludge. On the contrary, the experimental error (expressed as standard deviation of triplicates) generally increased after the second addition of substrate and a reduction of maximum SMA was observed in 2 of 3 sludges. This difference to observations in literature sources might be caused by differences in the experimental setup: the here presented method, for example, does not include the addition of macro nutrients and micro nutrients to the reaction vessels. Those nutrients might become limiting factors after a certain experiment duration.

11.5. Effect of sludge storage on sludge SMA

11.5.1. Testing

Knowing the effect of storage upon sludge activity is obviously extremely important in order to produce comparable data. DEWATS plants are often remote and maximum tolerable storage times for samples are a critical factor for the logistics of field investigations.

Anaerobic sludge is generally considered to be very stable over long periods. Change of sludge activity however has been reported by Colleran et al. (1992) for different anaerobic sludges after 65 d to 121 d of storage at 4°C. Castro et al. (2002) reported the least change of acetoclastic activity through refrigeration (20% to 40% activity reduction after 2 to 5 months storage).

Literature on this issue concerning low-activity sludge such as found in communal ABR systems could not be found. It was therefore crucial to investigate this point.

Table 50 contains the experimental details of three runs performed with the same sludge after 1 d, 9 d and 50 d of storage in the same container at 2°C to 6°C without being exposed to light.

Table 50: Experimental details, effect of sludge storage on sludge SMA

Plant	**MM**	**MM**	**MM**
Sampling point	**ABR 1**	**ABR 1**	**ABR 1**
Date of sampling	21.02.2013	21.02.2013	21.02.2013
Date of measurement	22.02.2013	02.03.2013	12.04.2013
Time between measurement and sampling (d)	1	9	50
Inoculum volume in bottle (l)	0.15	0.15	0.15
Conc. of pure inoculum (g VS l^{-1})	28.02	28.02	28.02
Inoculum in bottle (g VS)	4.20	4.20	4.20
Substrate conc. in bottle (g COD l^{-1})	1	1	1
S/I ratio (g COD g VS^{-1})	0.06	0.06	0.06

Figure 8 shows the cumulative CH_4 production curves of all three runs. The resulting difference in SMA values and increasing lag phase can clearly be seen in Figure 178. The longer the storage time the slower the sludge reaction to substrate addition and the lower the resulting SMA_{max}. The latter decreases by approximately 10% after 9 d and by 20% after 50 d.

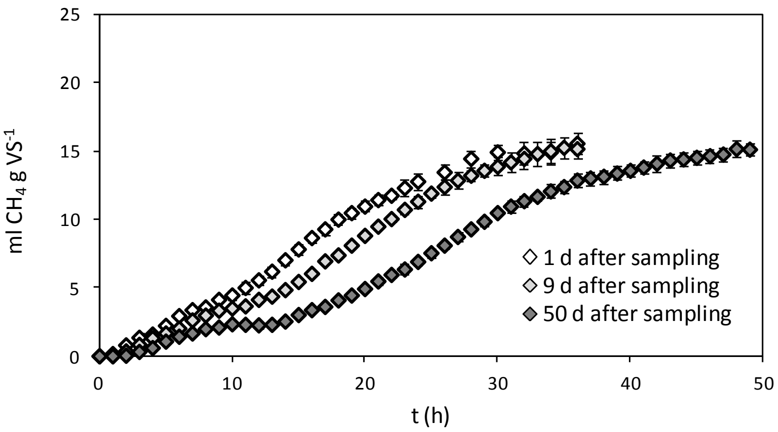


Figure 177: Cumulative CH_4 production at constant substrate (NaAc) concentration (1 g COD l^{-1}), constant inoculum (sludge) volume (150 ml) and varying storage times, data-points are averages of triplicates and control has been subtracted, error-bars indicate the sum of standard deviations of triplicate tests and triplicate controls, the theoretical maximal CH_4 production is 20 ml CH_4 g VS^{-1}, sludge sample: ABR 1, Minomartani

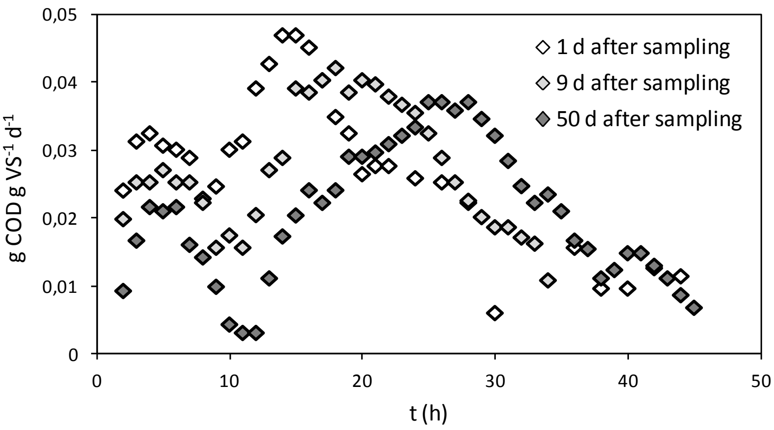


Figure 178: SMA curves of the experiments depicted in Figure 177, every data-point represents the running average over 4 h (t_i ±2 h)

The same type of experiment was carried out with four other sludge samples taken from the DEWATS plant Santan (see Section 6.6.2 for technical details). The sludge activities were measured after 1 d to 6 d and 30 d of storage under exactly the same experimental conditions. The Figure 179 a, b, c and d present the cumulative CH_4 production curves of these experiments. The sludge activity reduction is evident and seems to be more pronounced for sludges with little initial activity (ABR 2 and ABR 5).

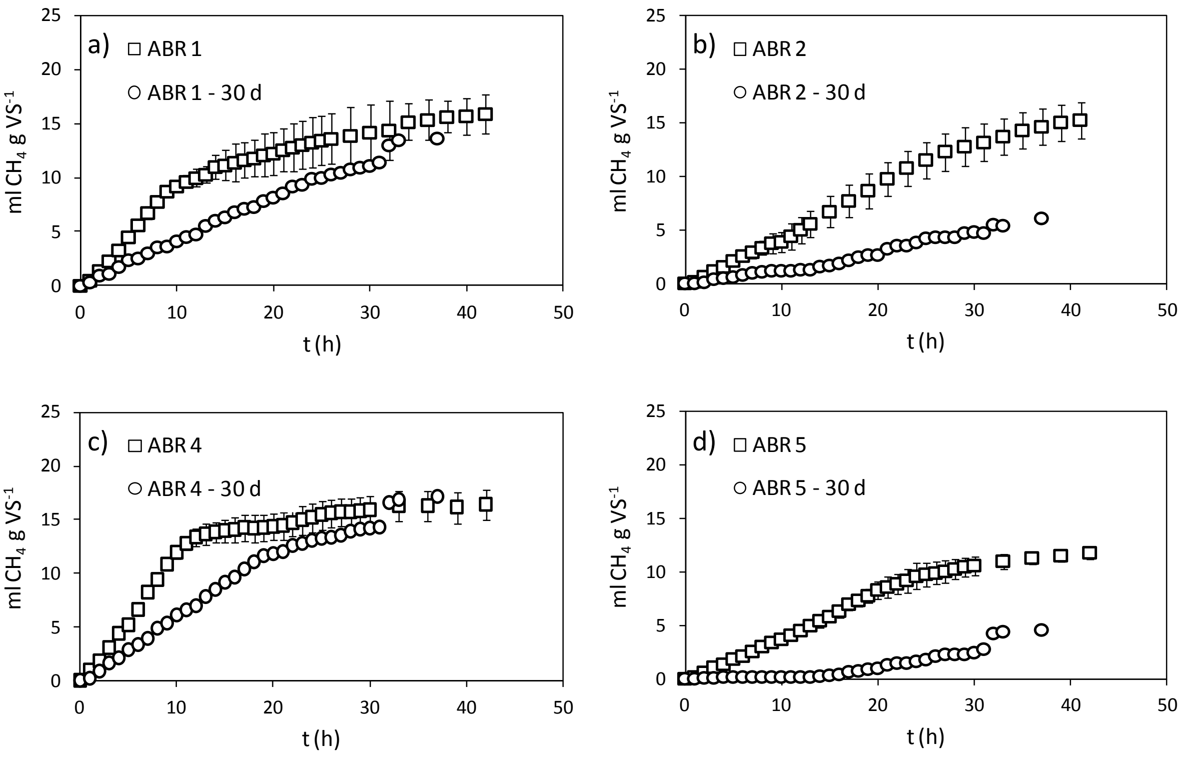


Figure 179 a, b, c and d: Cumulative CH_4 production at constant substrate (NaAc) concentration (1 g COD l^{-1}), constant inoculum (sludge) volume (150 ml) and varying storage time (1 d to 6 d after sampling and 30 d later), data points of the runs right after sampling are averages of triplicates and error-bars indicate the sum of standard deviations of triplicate tests and triplicate controls, later runs were done as single measurements, controls have been subtracted for all data-points, the theoretical maximal CH_4 production is 20 ml CH_4 gVS^{-1}, sludge sample points: ABR 1, ABR 2, ABR 4 and ABR 5, Santan

11.5.2. Conclusion

Storage clearly has an adverse, and in some cases strongly adverse, effect on the responsiveness and activity of acetoclastic methanogens. This is shown by increased time-lag in SMA curves and reduced SMA_{max} values. It appears that the lower the sludge activity the stronger the negative effect of storage on the sludge. Current results therefore suggest that DEWATS sludges should be processed as soon as possible after sampling within the period of one week.

11.6. Multiple measurements with sludges from identical sampling points

11.6.1. Testing

This paragraph investigates the variation of multiple consecutive SMA measurements of sludge taken from the same sampling point but on different days. This variation indicates combined sampling and measurement errors and potential short term fluctuations of sludge activity in the system.

The effect of storm water ingress on this comparison can be ruled out since all samples were taken during a period with regular heavy rains at the end of the wet season. It is assumed that potential sludge migration would have occurred at the beginning of the wet season and that the sludge inventory remained comparably constant at the end of the wet season. Table 51 presents the experimental details of the data and the respective SMA_{max} values of the runs. Samples were taken from a total of four sample points on two plants. The runs have a relative standard deviation of 1% to 12%. The time period between two sampling campaigns varies between 9 d and 70 d. There does not seem to be any relationship between the length of time between the two measurements and their standard deviation.

Figure 180 summarizes the comparison of the multiple runs.

Table 51: Comparison of SMA_{max} values of multiple runs, all dates in the year 2013

Dataset	**1**		**2**		**3**		**4**	
Plant	**MM**		**MM**		**ST**		**ST**	
SP	**ABR 1**	**ABR 1**	**ABR 5**	**ABR 5**	**Settler 2**	**Settler 2**	**ABR 4**	**ABR 4**
Date of sampling	12.02.	21.02	15.03.	15.05.	08.03.	17.05.	08.03.	17.05.
Time between measurement and sampling (d)	1	1	4	1	1	1	6	1
Time between two measurement campaigns (d)	9		61		70		70	
Inoculum volume in bottle (l)	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
Conc. of pure inoculum (g VS l^{-1})	32	28	32	41	28	33	35	34
Inoculum in bottle (g VS)	5	4	5	6	4	5	5	5
Substrate conc. in bottle (g COD l^{-1})	1	1	1	1	1	1	1	1
SMA_{max} (g COD g VS^{-1} d^{-1})	0.03	0.03	0.10	0.12	0.02	0.02	0.08	0.08
M (g COD g VS^{-1} d^{-1})	0.03		0.11		0.02		0.08	
SD (g COD g VS^{-1} d^{-1})	0.0015		0.0127		0.0013		0.0009	
RSD of duplicate consecutive measurements	5%		12%		8%		1%	

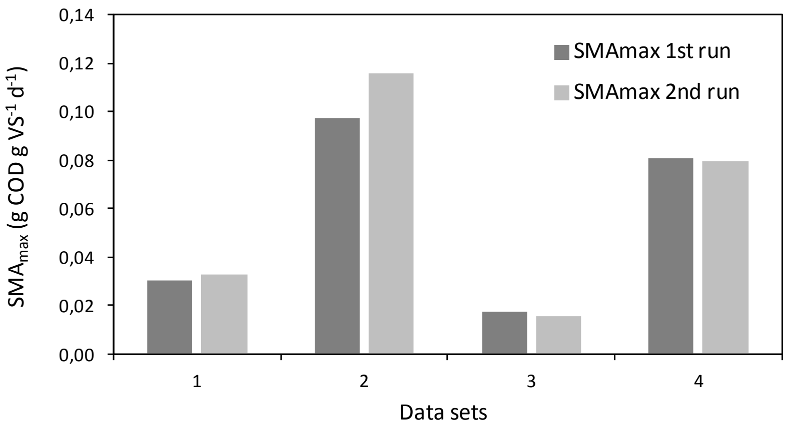


Figure 180: Comparison of SMA_{max} values of multiple runs

11.6.2. Conclusion

Double SMA_{max} measurements of samples taken from the same sampling points up to 3 months apart have a relative standard deviation of 1% to 12%. This standard deviation indicates the combined variation expected with SMA_{max} measurements due to sampling errors, measurement errors and potential short term sludge activity variations in the reactors since changes of external factors such as load variations and storm water ingress can be ruled out.

11.7. Summary of conclusions on SMA methodology

Observations made in this study indicate that SMA tests for DEWATS sludge should be conducted with 1 g COD l^{-1} substrate concentration and 150 ml sludge resulting in an approximate S/I ratio of 0.05 g COD g VS^{-1}. This however is lower than the optimal range reported by Souto et al. (2010) (0.125 to 0.75 g COD g VS^{-1}) and Cho et al. (2005) (0.4 to 0.6 g COD g VS^{-1}). The experimental data has shown that an approximation to those ranges should not be done for DEWATS ABR sludge: tests with lower inoculum volumes (100 ml) were shown to lead to non-linear biogas production. This could be due to mass transfer limitation. Higher substrate concentrations on the other hand have been shown in previous work (Pietruschka, 2013) to inhibit the CH_4 production.

Acclimatisation of sludge through multiple substrate addition was not observed and multiple substrate addition leads to reduced SMA_{max} values. Experiments should therefore be run with single substrate addition.

DEWATS sludge storage times should not exceed one week since storage was clearly shown to have an adverse, and in some cases highly adverse, effect on the responsiveness and activity of acetoclastic methanogens.

The standard deviation of triplicate measurements was found to be very small, especially during the most decisive first 10 h to 15 h of the experiment. Duplicate runs should therefore suffice for future investigations.

Double SMA measurements of samples taken from the same sampling points up to three months apart have a relative standard deviation of between 1% and 12%.

12. APPENDIX A3: NONBIODEGRADABLE COD

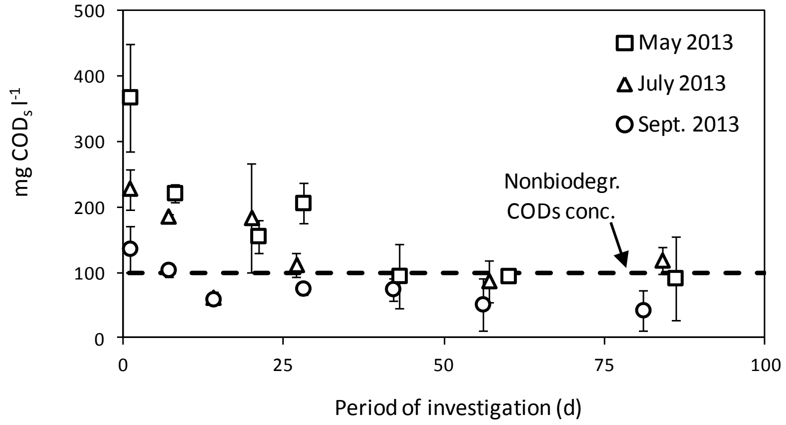


Figure 181: Nonbiodegradable effluent COD_s concentration measurements done on samples taken at three different dates at BWC (indicated as month and year), data-points represent the averages of duplicate measurements on duplicate samples, error-bars indicate the standard deviation of these four values

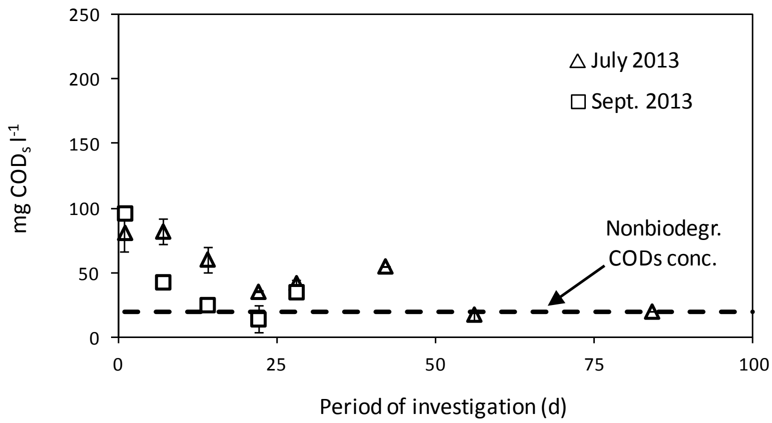


Figure 182: Nonbiodegradable effluent COD_s concentration measurements done on samples taken at three different dates at GB (indicated as month and year), data-points represent the averages of single and duplicate measurements on duplicate samples, error-bars indicate the standard deviation of these three values

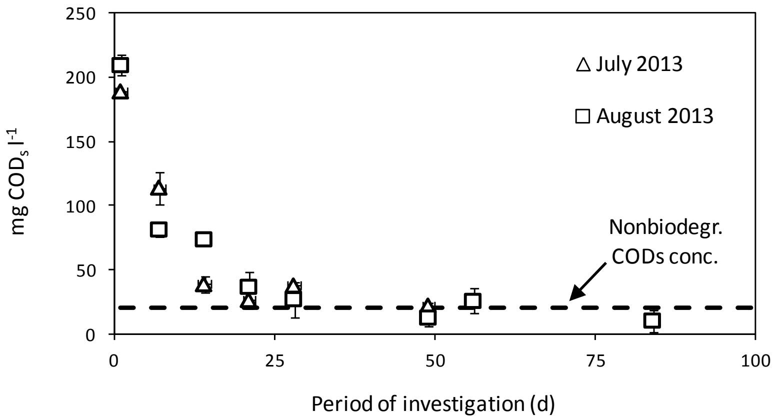


Figure 183: Nonbiodegradable effluent COD_s concentration measurements done on samples taken at three different dates at MM (indicated as month and year), data-points represent the averages of single and duplicate measurements on duplicate samples, error-bars indicate the standard deviation of these three values

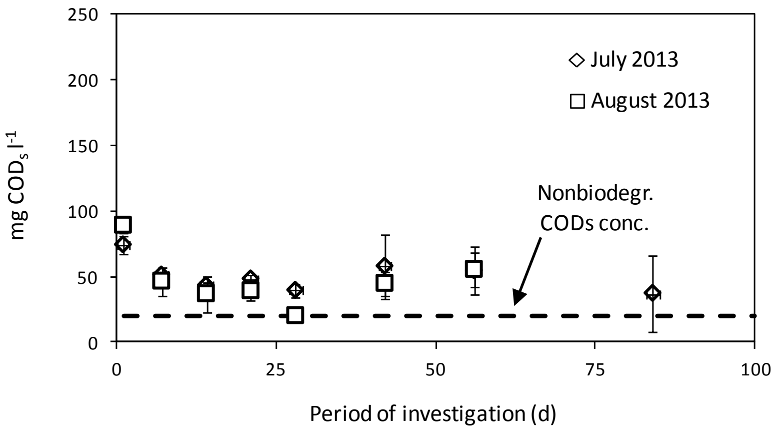


Figure 184: Nonbiodegradable effluent COD_s concentration measurements done on samples taken at three different dates at ST (indicated as month and year), data-points represent the averages of single and duplicate measurements on duplicate samples, error-bars indicate the standard deviation of these three values

13. APPENDIX A4: ADM-3P MODEL PARAMETERS

Table 52: Model-parameters and their values as adopted from Ikumi (2011) and (Sam-Soon et al. (1991))

Parameter	Value	Units	Description	Reference
K_ac	0.015	d^{-1}	Decay rate constant acetogens	Sam-Soon et al. (1991)
K_ad	0.041	d^{-1}	Decay rate constant acidogens	Sam-Soon et al. (1991)
K_am	0.037	d^{-1}	Decay rate constant acetoclastic methanogens	Sam-Soon et al. (1991)
K_bp	1.603	d^{-1}	Hydrolysis rate biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
K_bps	1.796	d^{-1}	Hydrolysis rate biodegradable particulate organics	Ikumi (2011)
K_fs	10	d^{-1}	Hydrolysis rate biodegradable particulate organics	Ikumi (2011)
K_hm	0.01	d^{-1}	Decay rate constant hydrogenotrophic methanogens	Sam-Soon et al. (1991)
kdis_cap	350		Dissolution constant of calcium phosphate	Ikumi (2011)
kdis_mgkp	1		Dissolution constant of K-struvite	Ikumi (2011)
kdis_stru	8000		Dissolution constant of struvite	Ikumi (2011)
ki_am	1.15E-06	mol l^{-1}	H^+ inhibition coefficient for acetoclastic methanogens	Ikumi (2011)
ki_H2	1.25	g m^{-3}	Inhibition coefficient for H_2 in acidogenesis	Ikumi (2011)
ki_hm	0.00053	mol l^{-1}	H^+ inhibition coefficient for hydrogenotrophic methanogens	Ikumi (2011)
ks_ac	290	g m^{-3}	Half saturation coefficient acetogens	Sam-Soon et al. (1991)
ks_ad	150	g m^{-3}	Half saturation coefficient acidogens	Sam-Soon et al. (1991)
ks_am	350	g m^{-3}	Half saturation coefficient acetoclastic methanogens	Sam-Soon et al. (1991)
ks_bp	5.387	g m^{-3}	Half saturation coefficient biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
ks_bps	7.962	g m^{-3}	Half saturation coefficient biodegradable particulate organics	Ikumi (2011)
ks_hm	2.5	g m^{-3}	Half saturation coefficient hydrogenotrophic menthanogens	Sam-Soon et al. (1991)
mu_ac	1.15	d^{-1}	Maximum specific growth rate acetogens	Sam-Soon et al. (1991)
mu_ad	0.85	d^{-1}	Maximum specific growth rate acidogens	Sam-Soon et al. (1991)
mu_am	0.375	d^{-1}	Maximum specific growth rate acetoclastic methanogens	Sam-Soon et al. (1991)
mu_hm	0.4	d^{-1}	Maximum specific growth rate hydrogenotrophic methanogens	Sam-Soon et al. (1991)
a_bp	0.227		N/C ratio biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
a_bps	0.064		N/C ratio biodegradable particulate organics	Ikumi (2011)
a_e	0.1		N/C ratio endogenous residue	Ikumi (2011)
a_f	0.009		N/C ratio fermentable soluble organics	Ikumi (2011)
a_o	0.166		N/C ratio organisms	Ikumi (2011)
a_up	0.1		N/C ratio unbiodegradable particulates	Ikumi (2011)
a_us	0.086		N/C ratio unbiodegradable solubles	Ikumi (2011)
b_bp	0.031		P/C ratio biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
b_bps	0.01		P/C ratio biodegradable particulate organics	Ikumi (2011)
b_e	0.035		P/C ratio endogenous residue	Ikumi (2011)

Parameter	Value	Units	Description	Reference
b_f	0.011		P/C ratio fermentable soluble organics	Ikumi (2011)
b_o	0.023		P/C ratio organisms	Ikumi (2011)
b_up	0.035		P/C ratio unbiodegradable particulates	Ikumi (2011)
b_us	0.01		P/C ratio unbiodegradable solubles	Ikumi (2011)
f_e	0.08			Ikumi (2011)
Y_AC	0.042		Yield coefficient acidogenesis	Sam-Soon et al. (1991)
Y_AD	0.091		Yield coefficient acetogenesis (low H_2 partial pressure)	Sam-Soon et al. (1991)
Y_AH	0.1074		Yield coefficient acetogenesis (high H_2 partial pressure)	Ikumi (2011)
Y_AM	0.041		Yield coefficient acteoclastic methanogenesis	Sam-Soon et al. (1991)
y_bp	1.454		H/C ratio biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
y_bps	2.19		H/C ratio biodegradable particulate organics	Ikumi (2011)
y_e	1.32		H/C ratio endogenous residue	Ikumi (2011)
y_f	1.899		H/C ratio fermentable soluble organics	Ikumi (2011)
Y_HM	0.039		Yield coefficient	Sam-Soon et al. (1991)
y_o	1.485		H/C ratio organisms	Ikumi (2011)
y_up	1.32		H/C ratio unbiodegradable particulates	Ikumi (2011)
y_us	1.753		H/C ratio unbiodegradable solubles	Ikumi (2011)
z_bp	0.357		O/C ratio biodegradable particulate organics (resulting from MO decay)	Ikumi (2011)
z_bps	0.653		O/C ratio biodegradable particulate organics	Ikumi (2011)
z_e	0.443		O/C ratio endogenous residue	Ikumi (2011)
z_f	0.698		O/C ratio fermentable soluble organics	Ikumi (2011)
z_o	0.424		O/C ratio organisms	Ikumi (2011)
z_up	0.443		O/C ratio unbiodegradable particulates	Ikumi (2011)
z_us	0.586		O/C ratio unbiodegradable solubles	Ikumi (2011)

14. APPENDIX A5: A STORM WATER OVERFLOW CONCEPT FOR DEWATS

Typical storm water overflow systems limit the plant feed flow to the maximum design value by reducing the flow-profile of the feed piping. The maximum design flows of communal DEWATS are however so small that the correspondingly small flow-profiles would be extremely susceptible to blockages by solids contained in the plant feed.

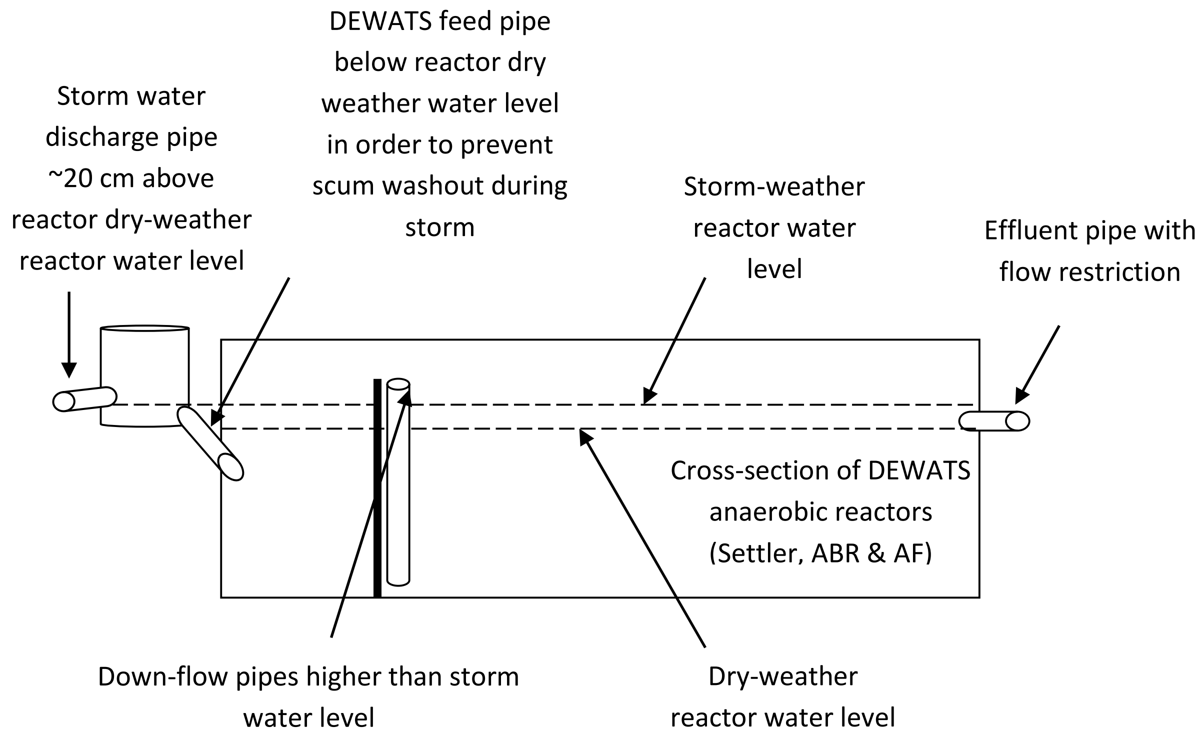
The sketch below outlines a concept which may solve this problem by reducing the pipe diameter at the plant effluent instead of at the plant feed.

This procedure has the advantage of:

- No blocking at plant inlet since the initial feed pipe diameter is maintained.
- The design peak-flow is maintained throughout plant operation.
- The piping restriction at the effluent pipe (see Figure below) can easily be accessed and cleaned if needed.
- The piping restriction at the effluent pipe can easily be tested and varied at design stage for different flows (assuming that dimensioning the correct pipe reduction purely through calculations will be difficult since the hydraulic resistance of scum layers, AF filter material and reactors containing sludge are unknown).
- A water level increase of 20 cm inside the reactors represents for the average plant design (300 connected people) about 2 m³ of retained wastewater from the "first flush" which may contain large amounts of solids.
- Discharged wastewater will mainly consist of rainwater since the "first flush" is retained inside the DEWATS.

This procedure implies the following design changes:

- Lowering the plant feed pipe below dry-weather reactor water level (in order to prevent settler scum washout during storm)
- Including a shaft at plant feed with a storm water discharge approximately 20 cm above dry-weather reactor water level
- Slight extension of ABR down-flow pipes above water level
- Slight extension of AF desludging shaft pipes above water level
- Easy access to the effluent pipe where the restriction pipe-cap is fitted
- Flow restriction pipe-cap needs to be fitted to effluent pipe at a standard height (the height difference between flow restriction and reactor water level has to be standardised for all plants in order to guarantee the same water pressure on the flow restriction and thus the same maximum flow)
- Flow restriction-caps need to be standardized for different design peak flows


DEWATS feed pipe below reactor dry weather water level in order to prevent scum washout during storm
Storm water discharge pipe ~20 cm above reactor dry-weather reactor water level
Storm-weather reactor water level
Effluent pipe with flow restriction
Cross-section of DEWATS anaerobic reactors (Settler, ABR & AF)
Down-flow pipes higher than storm water level
Dry-weather reactor water level

15. APPENDIX A6: ACCESS TO RAW DATA AND CALCULATIONS

Raw data and calculations presented in this dissertation are hosted by BORDA. Access credentials may be requested at *office@borda.de*.

The table below presents the folder structure containing the data and calculation spreadsheets.

Table 53: Folder structure containing the raw data and calculations presented in this dissertation

Folder	Subfolder	Description
Chapter 4		Contains raw data and calculations presented and discussed in Chapter 4
Chapter 5		Contains raw data and calculations presented and discussed in Chapter 5
Chapter 6	Case study A	Contains raw data and calculations presented and discussed in Chapter 6.3
	Case study B	Contains raw data and calculations presented and discussed in Chapter 6.4
	Case study C	Contains raw data and calculations presented and discussed in Chapter 6.5
	Case study D	Contains raw data and calculations presented and discussed in Chapter 6.6
	Comparing case studies	Contains raw data and calculations presented and discussed in Chapter 6.7
Chapter 7	Case study A Case study B Case study C Case study D	Contains model input data derivations from raw field data and model output raw data processing for each case study
	WEST® files	Contains all relevant files to run the here presented experiments on WEST®
Literature	Chinese papers I	Contains the English translations of eighteen Chinese papers that were, based on their English titles, initially thought to be relevant to this study
	Chinese papers II	Contains the English translations of eleven Chinese papers discussed in Chapter 2.3
	Publications resulting from this thesis	Contains nine conference papers, one report and one M.Sc. thesis which were produced during the course of this study, all listed in Table 4
Methodology	Field work	Contains eleven SOPs detailing the procedures of various field investigations
	Lab work	Contains four SOPs detailing the procedures of various laboratory measurements