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Katharina Müller

A systemic approach to implementation of sanitation and agricultural water reuse

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A systemic approach to implementation of sanitation and agricultural water reuse

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Dipl.-Geoökol. Katharina Müller
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Korreferenten: Prof. Dr. Susanne Lackner
Prof. Dr.-Ing. Jörg Drewes

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Herstellung: Lasertype GmbH, Holzhofallee 19
64295 Darmstadt

Vertrieb: Institut IWAR
TU Darmstadt
Franziska-Braun-Straße 7
64287 Darmstadt
Telefon: 06151 / 16 20301
Telefax: 06151 / 16 20305

Abstract

In the light of a growing global urban population and increasing water scarcity, previous work in the field of sanitation and water reuse has emphasized the need for a holistic and integrated view of all components involved in sanitation systems. A systemic approach is needed to recover the resources contained in sewage and to maximize their benefits. There is a consensus that this approach requires further support.

The main obstacles to the implementation of new approaches to urban water management are the lack of knowledge regarding inherent uncertainties and risks, practical management challenges, the available institutional capacities and capacities to facilitate community involvement, financial considerations, and institutional and personal biases that act as barriers.

This dissertation explores the main challenges and knowledge gaps encountered during implementation of a project on sanitation and water reuse in North Namibia. The objective is to fill the encountered knowledge gaps and to demonstrate the implications that the observations have in practice. This reduces obstacles to the implementation of new concepts in urban water management and maximizes the achievable benefits of sanitation systems.

The implemented infrastructure includes various types of sanitation facilities, a vacuum sewer system, a wastewater treatment plant with sedimentation and anaerobic pretreatment, aerobic treatment and secondary clarification, microscreening and UV disinfection. The treated water is stored in a pond and applied to agricultural fields via surface drip lines. The reclaimed water is used for the production of vegetables for human consumption.

The main results of this study and its consequences for practice can be summarized as follows:

- The specific water use and the specific loads in wastewater from shared sanitation facilities differ considerably from those of individual sanitation facilities. Hence, the wastewater characteristics are also different, which has implications for wastewater and sludge treatment, nutrient and salt management for water reuse on agricultural fields and the energy recovery potential from the wastewater constituents and agricultural biomass.
- The structural layout of shared sanitation facilities needs to fit with the desired management and billing system. Particularly important aspects are the collection of revenues and control of visitor flows.
- Tariff levels, the method of revenue collection, and the population density influence the utilization and hence the quantities and characteristics of the wastewater from shared sanitation facilities. This already needs to be considered during planning. Generation of sufficient revenues for cost recovery is difficult in low-income areas with a low population density.
- In settings where national regulations regarding reclaimed water quality do not (yet) exist, the recommendations in this study can be used to develop relevant water quality criteria.

The recommendations in existing international guidelines are complemented with site-specific water quality limits for the protection of irrigation infrastructure (turbidity, chemical oxygen demand, biochemical oxygen demand), the required water quality prior to UV disinfection (turbidity, total suspended solids, particle size), and prevention of eutrophication and negative effects on plants (nitrogen, phosphorus and potassium).

- Water storage facilities should be considered as an additional treatment step that contributes to the reliability of the water reclamation process and to achieving the required water quality.
- The risks of soil salinization and overfertilisation were less serious than expected in this case. However, in other settings with, e.g., a higher proportion of wastewater from individual households, measures for control of salts and nutrient input to agricultural fields need to be implemented as suggested.
- Residues of crops irrigated with reclaimed water can contribute only to a limited extent to biogas and electricity generation via co-digestion with sewage sludge. The market value of the crops is usually higher than the value of the producible electricity.
- Important impediments to co-generation in Southern Africa are the tariff structures of the local electricity supply entities. Rebates or credits for electricity fed into the grid are usually not possible. Additionally, fixed costs constitute a major part of the electricity costs. Thus, for the given tariff structure, co-generation can only reduce electricity costs if the produced electricity is consumed immediately on site.

For the first time, a sanitation system has been analyzed from a holistic perspective, providing detailed specifications for planning, data monitoring and influencing factors. This is a sound basis for better planning and implementation of similar projects. The knowledge gaps that caused misconceptions and difficulties during realization of this project are now closed or addressed and can, at least, be realistically assessed right from the start.

Kurzfassung

Globales Bevölkerungswachstum, zunehmende Urbanisierung und steigende Wasserknappheit erfordern eine ganzheitliche, integrierte Vorgehensweise hinsichtlich Wasserver-, Abwasserentsorgung und Wasserwiederverwendung. Ein systemischer Ansatz ist notwendig, um im Abwasser enthaltene Ressourcen zurückzugewinnen und den durch Sanitärversorgung erzielbaren Mehrwert zu maximieren. Diese Vorgehensweise muss in Zukunft weitere Verbreitung finden.

Hindernisse für die Implementierung von neuen Konzepten im urbanen Wassermanagement sind hauptsächlich fehlendes Wissen hinsichtlich systemimmanenter Unsicherheiten und Risiken, Managementherausforderungen in der Praxis, die zur Verfügung stehenden institutionellen Kapazitäten, die zur Verfügung stehenden Kapazitäten zur Einbindung der lokalen Bevölkerung, finanzielle Erwägungen sowie als Barrieren wirkende institutionelle und persönliche Neigungen.

Die vorliegende Dissertation untersucht die wesentlichen Herausforderungen und Wissenslücken während der Implementierung eines Projekts zur Abwassersammlung, -behandlung und Wasserwiederverwendung in Nord-Namibia. Die im Rahmen dieses Projekts implementierte Infrastruktur umfasst verschiedene Arten von Sanitäranlagen, eine Vakuumkanalisation, eine Kläranlage mit Sedimentation und anaerober Vorbehandlung des Abwassers, aerober Behandlung mit Nachklärung, Mikrosiebung und UV Desinfektion. Das behandelte Abwasser wird in einem Becken gespeichert und mit oberirdischer Tröpfchenbewässerung auf landwirtschaftlichen Flächen aufgebracht. Das Wasser wird für die Produktion von Gemüse für den menschlichen Verzehr verwendet.

Ziel ist es, die zuvor erwähnten Wissenslücken zu schließen und ihre Bedeutung für die Praxis herauszuarbeiten. Somit werden Hemmnisse für die Umsetzung von neuen Konzepten im urbanen Wassermanagement reduziert und der erzielbare Nutzen optimiert.

Die wichtigsten Ergebnisse dieser Arbeit sind wie folgt zusammengefasst:

- Der spezifische Wasserbedarf und die spezifischen Frachten im Abwasser gemeinschaftlich genutzter Sanitäreinrichtungen unterscheiden sich deutlich von denen individuell genutzter Sanitäreinrichtungen. Entsprechend unterscheiden sich auch die Eigenschaften des Abwassers, was Konsequenzen für die Abwasser- und Schlammbehandlung, das Nährstoff- und Salzmanagement bei Wiederverwendung in der Landwirtschaft und die energetische Verwertung der Abwasserinhaltssoffe und landwirtschaftlicher Biomasse nach sich zieht.
- Die bauliche Gestaltung der gemeinsam genutzten Sanitäreinrichtungen muss zu dem gewünschten Management- und Gebührenabrechnungssystem passen. Besonders wichtige Aspekte sind die Sammlung der Gebühren und die Kontrolle der Besucherströme.
- Tarifniveaus, die Art der Gebührensammlung und die Bevölkerungsdichte beeinflussen die Nutzung und damit die Mengen und Eigenschaften des Abwassers von gemeinsam genutzten Sanitäreinrichtungen. Das muss bereits während der Planung berücksichtigt werden.

- An Standorten, an denen nationale Regularien hinsichtlich der Wasserqualität für die Wiederverwendung (noch) nicht existieren, können die Empfehlungen in dieser Arbeit genutzt werden um relevante Kriterien zu entwickeln. Diese Dissertation ergänzt existierende internationalen Richtlinien mit standortspezifischen Grenzwerten für die erforderliche Wasserqualität für den Schutz der Bewässerungsinfrastruktur (Trübung, chemischer Sauerstoffbedarf, biochemischer Sauerstoffbedarf), die notwendige Wasserqualität vor einer UV-Desinfektion (Trübung, abfiltrierbare Stoffe, Partikelgröße) und zur Vermeidung von Eutrophierung und negativer Effekte auf Pflanzen (Stickstoff, Phosphor und Kalium).
- Einrichtungen zur Speicherung von Wasser sollten als zusätzlicher Behandlungsschritt betrachtet werden, der die Verlässlichkeit des Prozesses der Wasserwiederverwendung erhöht und zum Erreichen der erforderlichen Wasserqualität beiträgt.
- Die Risiken der Bodenversalzung und Überdüngung waren in diesem Fall weniger schwerwiegend als erwartet. An anderen Standorten mit z. B. einem höheren Anteil von Abwasser aus individuellen Sanitäreinrichtungen müssen Maßnahmen zur Kontrolle von Salz- und Nährstoffeinträgen auf landwirtschaftliche Felder wie vorgeschlagen implementiert werden.
- Pflanzen und Ernterückstände können nur in begrenztem Ausmaß über Co-Vergärung mit Abwasserschlamm zur Erzeugung von Biogas und Elektrizität beitragen. Der Marktwert der Feldfrüchte ist in der Regel höher als der Wert der daraus produzierbaren Elektrizität.
- Im südlichen Afrika bestehen durch die Tarifstrukturen der lokalen Stromversorger erhebliche Hindernisse für die Realisierung von Co-Vergärung. Vergünstigungen oder Gutschriften für in das Netz eingespeiste Elektrizität sind in den meisten Fällen nicht möglich. Hinzu kommt, dass Fixkosten häufig einen großen Teil der Stromkosten ausmachen. Deshalb können für die vorliegende Tarifstruktur Elektrizitätskosten nur durch Co-Vergärung reduziert werden, wenn der produzierte Strom sofort vor Ort verbraucht wird.

Zum ersten Mal wurde ein System zur Abwassersammlung, -behandlung und Wasserwiederverwendung aus einer ganzheitlichen Perspektive analysiert und detaillierte Informationen hinsichtlich Planung, Monitoring und wichtiger Einflussgrößen während der Realisierung dargestellt. Das ist eine solide Grundlage für die bessere Planung und Implementierung von vergleichbaren Projekten. Wissenslücken, die zu falschen Annahmen und Schwierigkeiten während der Umsetzung führten, wurden geschlossen beziehungsweise adressiert und können nun von Anfang an realistisch eingeschätzt werden.

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List of abbreviations

abbreviation	meaning
µS	microsiemens
a	year, for Latin "annus"
AVC	Aranos Village Council
BMBF	Bundesministerium für Bildung und Forschung (German Federal Ministry of Education and Research)
BOD_5	5-day biochemical oxygen demand
BPC	Botswana Power Corporation
BWP	Botswana Pula
BWT	Bilfinger Water Technologies
c	concentration
CAB	community ablution block
CAS	conventional activated sludge process
CENORED	Central North Regional Electricity Distributor
CH_4	methane
CHC	community health club
CLTS	community-led total sanitation
COD	chemical oxygen demand
d	day
DALY	disability-adjusted life year
DCOD	dissolved chemical oxygen demand
DRFN	Desert Research Foundation of Namibia
dS	decisiemens
E. coli	*Escherichia coli*
EC	electrical conductivity
ECB	Electricity Control Board
ERONGORED	Erongo Regional Electricity Distributor
EUR	euro
FAO	Food and Agriculture Organization of the United Nations

FM	fresh matter
FNU	formazine nephelometric unit
gp	growth period
ha	hectare
HBT	health based target
HE	helminth egg
HI	harvest index
ISOE	Institute for social-ecological Research
JMP	Joint Monitoring Programme
KaMu	Karasburg Municipality
KeMu	Keetmanshoop Municipality
kWh	kilowatt hour
LC	lamella clarifier
MaMu	Mariental Municipality
MAWF	Ministry of Agriculture, Water and Forestry Namibia
MDG	Millennium Development Goal
meq	milliequivalents
MME	Ministry of Mines and Energy Namibia
MPN	most probable number
n	number of measurements
NGO	Non-governmental organization
N. Z.	New Zealand
NAD	Namibian dollar
NamPower	Namibia Power Corporation
NamWater	Namibia Water Corporation
NEF	National Energy Fund
NGO	Non-governmental organization
NORED	Northern Regional Electricity Distribution Company
NTU	nephelometric turbidity unit
o&m	operation and maintenance

OECD	Organization for Economic Co-operation and Development
OPE	Oshakati Premier Electric
OTC	Outapi Town Council
PCOD	particulate chemical oxygen demand
pe	population equivalent
PHAST	participatory hygiene and sanitation transformation
PRA	participatory rural appraisal
RBC	rotating biological contactor
RED	regional electricity distributor
RO	reverse osmosis membrane filtration
SAPP	Southern African Power Pool
SAR	Sodium adsorption ratio
SBR	sequencing batch reactor
sd	standard deviation
SFB	submerged fixed bed reactor
SPARC	Society for the Promotion of Area Resource Centers
spp.	species pluralis, “several species”
SS	suspended solids
SSA/SSP	strategic sanitation approach/strategic sanitation planning
TC	total coliforms
TCOD	total chemical oxygen demand
TDS	total dissolved solids
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TOU	time of use (Mariental municipality) type of user (Botswana Power Corporation)
TP	total phosphorus
TS	total solids
TSS	total suspended solids
TUDa	Technische Universität Darmstadt
UASB	upflow anaerobic sludge blanket

USEPA	U. S. Environmental Protection Agency
UV	ultraviolet
VS	volatile solids
Wh	watt hour
WHO	World Health Organization
WSUP	Water & Sanitation for the Urban Poor
ww	wastewater
wwtp	wastewater treatment plant
ZAR	South African Rand
η_{el}	electrical efficiency
η_{therm}	thermal efficiency

Glossary

term	definition
agricultural water use	"Water used for soil cultivation, crop production and livestock uses" (Leverenz and Asano 2011)
anaerobic digestion	"The engineered methanogenic decomposition of organic matter, carried out in reactor vessels, called digesters, that may be mixed or unmixed and heated or unheated" (Wilkie 2008)
anaerobic ponds	"Anaerobic ponds are 2-5 m deep and receive such a high organic loading […] that they contain no dissolved oxygen and no algae […]. They function much like open septic tanks, and their primary function is BOD removal." (Mara 1998)
basic sanitation	Basic sanitation includes actions at all levels to: "(a) Develop and implement efficient household sanitation systems; (b) Improve sanitation in public institutions, especially schools; (c) Promote safe hygiene practices; (d) Promote education and outreach focused on children, as agents of behavioral change; (e) Promote affordable and socially and culturally acceptable technologies and practices; (f) Develop innovative financing and partnership mechanisms; (g) Integrate sanitation into water resources management strategies." (UN 2002)
co-digestion	"The combined digestion of two or more substrates" (Grosser *et al.* 2013; Mata-Alvarez *et al.* 2014)
domestic water use	"Domestic water use includes water for normal household purposes, such as drinking, food preparation, bathing, washing clothes and dishes, flushing toilets and watering lawn and gardens." (Leverenz and Asano 2011)
evaporation pond	Disposal option for brine from membrane processes or agricultural drainage water (Tanji and Kielen 2002; Tchobanoglous *et al.* 2004). The water is evaporated "in natural depressions or specially designed unlined basins. The impounded water dissipates through evaporation and inadvertent seepage losses, and deposits salts and trace elements." (Tanji and Kielen 2002)
facultative pond	"Facultative ponds are designed for BOD removal on the basis of a relatively low surface loading […] to permit the development of a healthy algal population as the oxygen for BOD removal by the pond bacteria is mostly generated by algal photosynthesis." (Mara 1998)

improved drinking water source	A drinking water source “that, by the nature of its construction, adequately protects the source from outside contamination, particularly fecal matter.” (WHOandUNICEFÉ
improved sanitation facilities	Improved sanitation facilities “ensure hygienic separation of human excreta from human contact.” (WHO and UNICEF 2013)
informal settlements	synonyms: slum, squatter settlement, unplanned neighborhood “A settlement in an urban area in which more than half of the inhabitants live in inadequate housing and lack basic services.” (UN-HABITAT 2006)
irrigation water use	“Artificial application of water on lands to assist in the growing of crops and pastures or to maintain vegetative growth in recreational lands such as parks and golf courses.” (Leverenz and Asano 2011)
Joint Monitoring Program	“The WHO/UNICEF Joint Monitoring Programme for Water Supply and Sanitation (JMP), which began monitoring the sector in 1990, has provided regular estimates of progress towards the MDG targets, tracking changes over the 25 years to 2015.” (UNICEF and WHO 2015)
maturation pond	“A series of maturation ponds receives the effluent from the facultative pond, and the size and number of maturation ponds is governed mainly by the required bacteriological quality of the final effluent”. (Mara 1998)
Millennium Development Goals	The Millennium Development Goals were defined in 2000 by the United Nations General Assembly (UN 2000). Since then, “they have served as a shared framework for global action and cooperation on development.” (UN-HABITAT 2014)
multiple barrier approach	A multiple barrier approach “interrupts the flow of pathogens from the environment (wastewater, crops, soil etc.) to people. […] The available measures for health protection can thus be grouped into five main categories: (1) Waste treatment, (2) Crop restriction, (3) Irrigation technique, (4) Human exposure control, and (5) Chemotherapy and vaccination.” (Carr *et al.* 2004)
municipal water use	“The water withdrawals made by the populations of cities, towns, and housing estates, and domestic and public services and enterprises. Also includes water used directly to provide for the needs of urban populations, which consume high-quality water from city water supply systems.” (Leverenz and Asano 2011)

open defecation	"When human faeces are disposed of in fields, forests, bushes, open bodies of water, beaches or other open spaces or disposed of with solid waste." (WHO and UNICEF 2013)
oxidation pond	synonyms: wastewater lagoon, waste stabilization pond (Bitton 2011) "Shallow man-made basins into which wastewater flows and from which, after a retention time of many days (rather than several hours in conventional treatment processes), a well treated effluent is discharged." (Mara 1998)
reclaimed water	"Reclaimed water is a treated effluent suitable for an intended water reuse application." (Leverenz and Asano 2011)
sanitation	Sanitation is "a multi-step process in which human excreta and wastewater are managed from the point of generation to the point of use or ultimate disposal." (Tilley *et al.* 2008)
sanitation system	A sanitation system "collects excreta, transports it to a suitable location and/or stores it for treatment, treats it, reuses it and/or discharges it to the environment. A good sanitation system also minimizes or removes health risks and negative impacts on the environment" (IWA 2005). "A sanitation system also includes the management, operation and maintenance (O&M) required to ensure that the system functions safely and sustainably" (Tilley *et al.* 2008).
shared sanitation facilities	"Sanitation facilities […] shared between two or more households." (WHO and UNICEF 2013)
unimproved sanitation facilities	Unimproved sanitation facilities "do not ensure hygienic separation of human excreta from human contact." (WHO and UNICEF 2013)
water reclamation	"Treatment or processing of wastewater to make it reusable with definable treatment reliability and meeting appropriate water-quality criteria." (Leverenz and Asano 2011)
water recycling	"Water recycling normally involves only one use or user and the effluent from the user is captured and redirected back into that use scheme. In this context, water recycling is predominantly practiced in industry. […] The term recycled water is used synonymously with reclaimed water, particularly in California." (Leverenz and Asano 2011)

water reuse	"The use of treated wastewater for beneficial purposes such as agricultural irrigation and industrial cooling." (Leverenz and Asano 2011)

1 Introduction

1.1 The global situation and challenges regarding water supply and sanitation

The growing global population increases the demand for water, food, goods and energy (UNESCO 2015). In 2050, the world's population will reach 9.7 billion (UN DESA 2015), with 40% living in river basins suffering from severe water stress (OECD 2012). Between 2000 and 2050, the global water demand will increase by 55% (OECD 2012).

Water quality deterioration caused by insufficient agricultural and wastewater management is also a major concern (OECD 2012). Whereas surface and groundwater quality should be restored or stabilized in most OECD countries by 2050, it will further deteriorate in countries outside the OECD (OECD 2012). Urbanization increases the number of people without access to water and sanitation in cities, most notably in slums in the developing world, which puts additional pressure on local water resources (UNESCO 2015).

To meet the food demand in 2050, agriculture needs to increase global food production by 60% and, in developing countries, by 100% (Alexandratos and Bruinsma 2012). This can only be achieved by increasing water use efficiency, reducing water losses, increasing crop productivity with regard to water, and rational use of fertilizers (UNESCO 2015). Climate change is altering hydrological systems, with an overall negative impact on crop yields (IPCC 2014).

Already in 1977, at the World Water Conference in Mar del Plata, Argentina, the "International Drinking Water Supply and Sanitation Decade" was announced (Black 1998). The slogan for the 1980s became "Water and Sanitation for All" which summarizes its fundamental objective to provide safe water and sanitation for all people (Kalbermatten *et al.* 1980b).

In 1992, the Agenda 21 was passed at the United Nations Conference on Environment and Development in Rio de Janeiro (UN 1992). It includes calls for action to allow an environmentally sound, sustainable development and postulates developing new and alternative water resources, to prevent and control water pollution, to protect water resources against depletion, contamination and harm, to contribute to the management of scarce water resources through the promotion and expansion of reuse in agriculture, and to contribute to the development and application of clean technologies.

The Millennium Declaration from September 2000 reaffirmed the principles expressed in the Agenda 21 and furthermore contains the fundamental development goals of all UN member states (UN 2000). The main objectives regarding water are "to halve, by the year 2015, […] the proportion of people who are unable to reach or to afford safe drinking water", […] "to stop the unsustainable exploitation of water resources by developing water management strategies at the regional, national and local levels that promote both equitable access and adequate supplies" (UN 2000).

At the Johannesburg Summit 2002, tangible steps and quantifiable targets were identified for the implementation of the contents of the Agenda 21 and the Millennium Development Goals (MDG) (UN 2002). It was agreed to "integrate sanitation into water resources management strategies" and to "halve, by the year 2015, […] the proportion of people who do not have access to basic sanitation".

Between 1990 and 2015, 2.6 billion people gained access to improved drinking water sources and 2.1 billion people gained access to improved sanitation (UNICEF and WHO 2015). The global MDG for drinking water was met in 2010 but the global objective for sanitation was not achieved (UNICEF and WHO 2015). In 2015, 2.4 billion humans lived in unsatisfactory sanitary circumstances (UNICEF and WHO 2015). 1.5 million children die each year from diarrhea (UNICEF and WHO 2009); 88% of all diarrheal diseases are caused by contaminated drinking water, inadequate sanitation systems and insufficient hygiene (Prüss-Üstün 2008). More than 90% of the wastewater in low-income countries is discharged without treatment into water bodies (Sato *et al.* 2013).

Since 2015, water and sanitation goals and targets are defined in the "2030 Agenda for sustainable Development", which includes 17 sustainability development goals (UN 2015). Water and sanitation are addressed in goal 6: "ensure availability and sustainable management of water and sanitation for all" (UN 2015).

Development targets have been declared repeatedly. Progress has been made but, in many cases, the pre-set goals have not been met. The main challenges regarding water supply and sanitation have not changed markedly during the last decades. The available water resources are limited and more water and nutrients will be needed for production of food, goods and energy in the future. Nevertheless, a considerable proportion of the world's population does not have access to improved water and sanitation.

1.2 What is the problem?

A great effort has been made to fulfill the international development goals. Developed countries have managed to solve many of these problems. The question arises why it is so difficult to improve the conditions in less developed regions of the world.

1.2.1 Conventional and new concepts for water supply and sanitation

Sanitation systems consisting of toilets, sewers, wastewater and sludge treatment have been widely established in Europe and North America (IWA 2005). This is the result of a long development, starting with very simple collection of excreta in the 19th century, e.g., via cesspools or bucket systems (Roccaro *et al.* 2014). Later, as health hazards and other problems required further measures, wastewater collection was carried out by centralized sewer systems (Roccaro *et al.* 2014). Wastewater treatment also started with very basic approaches (e.g., natural anaerobic digestion in pits, soil treatment) until, in the first decades of the 20th century, the first larger scale biological treatment processes were put into operation (Roccaro *et al.*

2014). Stepwise, waterborne sewer systems and activated sludge plants became the standard for wastewater collection and treatment (Lüthi *et al.* 2009; Roccaro *et al.* 2014; Schertenlaib 2005). The introduction of water supply and sewers for wide, specified areas was probably the most important medical milestone since 1840 (Ferriman 2007). Why not apply this approach to other regions of the world? Conventional wastewater collection and treatment primarily considers wastewater and its constituents as problems that have to be eliminated from the water and disposed of (Henze 2008). Hence, this approach has been subjected to discussion (Daigger 2009; Gujer 2007b; Harremoës 1999; Lüthi *et al.* 2009; Oosterveer and Spaargaren 2010; Otterpohl *et al.* 1997; Panesar *et al.* 2011; Schertenlaib 2005; Tiberghien *et al.* 2011; Wilderer 2005). In 2000, the 'Bellagio principles' for sustainable sanitation were formulated, explicitly demanding that "waste should be considered a resource, and its management should be holistic and form part of integrated water resources, nutrient flows and waste management processes" (Eawag and Sandec 2000). Increasingly, wastewater is seen as a resource for water, nutrients, biosolids and energy (Cornel *et al.* 2011; Guest *et al.* 2009; Werner *et al.* 2003; Wilsenach *et al.* 2003).

However, new concepts for water supply and sanitation play only a minor role in the practical world (Marlow *et al.* 2013). In most cases, design engineers and stakeholders favor conventional approaches and technologies instead of new approaches (Nelson and Murray 2008). The main obstacles for the implementation of new approaches in sanitation systems are (1) a lack of knowledge regarding the uncertainties, risks and effects on existing system components when introducing new infrastructure solutions, (2) practical management challenges (for instance, regarding the increased complexity of the system), available institutional capacities and community involvement, (3) financial considerations, e.g., regarding smaller system sizes and external effects) and (4) institutional and personal bias that act as barriers to implementation of unconventional solutions (Marlow *et al.* 2013).

1.2.2 Challenges in developing countries

The proportion of people with access to sanitation facilities is especially low in developing countries. Here, only 62% of the population has access to improved sanitation, compared to 96% of the population living in developed regions (UNICEF and WHO 2015). Major problems for provision of water supply and sanitation in developing countries are (1) the general lack of infrastructure, (2) improved infrastructure that does not deliver, (3) inadequate attention to economic, institutional and social aspects and (4) hidden infrastructure failures, i.e., projects that look successful at first sight but are actually underperforming (Starkl *et al.* 2013). Starkl *et al.* (2013) note that most projects fail due to well-known reasons: inadequate planning, lack of external support, lack of funding and hygienic risks (due to technological shortcomings or because the technology becomes unsafe under realistic conditions). The wide spectrum of problems and aspects that need to be controlled during planning very often threaten successful implementation (Starkl *et al.* 2013).

Several publications deal with obstacles to implementation of water supply and sanitation infrastructure in developing regions (e.g., WHO and UNICEF (2000), Fry *et al.* (2008), Montgomery and Elimelech (2007), Nelson and Murray (2008), UN DESA (2008), EU (2012), Cross and Morel (2005), Wang *et al.* (2014), LaFond (1995), IWA (2005), Mwangi (2008)). The points discussed mainly center around finance and investments, capacity development, operation and maintenance, governance, policies, as well as institutional and monitoring issues.

1.2.3 What needs to be done?

These challenges have been addressed repeatedly since the early1980s (Kalbermatten *et al.* 1980a; Kennedy-Walker *et al.* 2014). Since the late 1980s and most notably in the 1990s, many different approaches have been developed that have the objective of facilitating and assisting in planning and implementation of sanitation interventions of any kind (Black 1998). Examples of these approaches are participatory planning tools such as PRA ("participatory rural appraisal"), methodologies for hygiene promotion, such as PHAST ("participatory hygiene and sanitation transformation") or CHCs ("community health clubs"), approaches for sanitation promotion e.g., community-led total sanitation (CLTS), and planning approaches for urban areas such as Sanitation 21 and SSA/SSP ("strategic sanitation approach"/"strategic sanitation planning") (Peal *et al.* 2010).

Traditional infrastructure planning usually focuses on a single aspect, such as the construction of a new sewer system or provision of toilet facilities and, commonly, the design does not exceed the single component`s boundary (McConville *et al.* 2011; Schramm and Kluge 2013). When considering only partial solutions, specific user requirements and the utilization contexts at the various system levels cannot be accounted for (Schramm and Kluge 2013).

New approaches in concept development and new technologies allow the linking of components that were considered separately in the past (McConville *et al.* 2011; Schramm *et al.* 2013). Examples of such new concepts and technologies include reclamation of treated water, source separation of wastewater flows, recovery of nutrients, anaerobic treatment of wastewater, co-digestion of sewage sludge with organic waste, generation of electrical and thermal energy from biogas generated by anaerobic digestion of sewage (sludge), introduction of innovative management and operating concepts and the objective of an overall improvement of resource efficiency when using water (Schramm *et al.* 2013). Thus, by broadening the system boundaries, interactions between components can be addressed to recover resources and maximize the benefits.

There is a consensus that, for comprehensive implementation of sanitation concepts, a holistic and integrative view of all system components is required (IWA 2005; McConville *et al.* 2011; Parkinson *et al.* 2011; Tiberghien *et al.* 2011; Voulvoulis 2012; Wilderer 2005). Whereas donor agencies are, in principle, aware of the need for a holistic viewpoint, this approach also needs to be promoted among water and sanitation engineers (Murphy *et al.* 2009; Tiberghien *et al.* 2011).

From these considerations, it is obvious that two main points make transfer of research knowledge into practice difficult. First, there is a knowledge gap that needs to be filled in order to change the perception of new approaches among stakeholders, compared to conventional approaches. The second point is the need to consider the entire water chain to maximize achievable benefits and to better match the individual components with each other.

1.3 Contribution of this work

This dissertation gathers all aspects that are required to analyze a sanitation system from a holistic perspective. For the first time, the complete framework is presented, instead of the examination of single aspects. The findings were developed using a practical example. They were gained from actually encountered tasks, challenges and obstacles.

The foundations of this dissertation are collected data and works carried out during planning and implementing a project on sanitation and water reuse in North Namibia between 2009 and 2015. The preliminary studies that were the basis for this initiative already started in 2004 (Kluge *et al.* 2008). The objective was to implement a comprehensive sanitation system that combines the technical and socio-economic aspects in an integrated, systemic approach.

In this dissertation, an integrated systemic approach means considering the entire water chain, from water supply and sanitation provision through to the reclaimed water for irrigation. Water is used several times for different purposes. It is used for personal hygiene, as a carrier medium to transport excreta away from humans to ensure maximum hygiene, as irrigation water containing nutrients and organics, as a source for organic material that can be converted to biogas and electricity, and, finally as a source for biosolids.

Implementation of this project in North Namibia meant handling many different issues and included a large variety of works that had to be carried out. The tag cloud in Figure 1 illustrates the diversity of the topics dealt with during the project period. First of all, the local conditions in the project area were the initial point for developing a suitable sanitation system. Many different aspects, such as environmental and socio-economic conditions and the national political agenda regarding water supply and sanitation had to be considered. Planning and implementation started with a community-based situation assessment, as a part of the demand responsive approach used for development of the technical layout (Kramm and Deffner 2017). The comprehensive assessment of the local conditions was the basis for choosing the technical infrastructure that would best comply with the local town council's agenda for local development and residents' needs and wishes.

The implemented hardware had to fit the local conditions. Appropriate technical components and treatment technologies had to be chosen from the available options. Their implementation was accompanied by numerous organizational tasks, including detailed planning and design of the facilities, support of tendering processes, construction and start-up of the facilities, development of operating procedures and management structures of the implemented infrastructure, introduction of a tariff and billing system, establishment of operation and maintenance routines and coordination of the project partners' activities – in brief, everything needed for successful implementation.

From the topics addressed and the activities carried out in the course of the project, some of the emerging tasks and questions could be resolved easily, because enough information was accessible. For instance, when choosing the type of sewers and wastewater treatment steps, it was possible to refer to abundant literature and comparative studies. Thus, there was enough information at hand for making a qualified decision about the most suitable technology for the local conditions in this project.

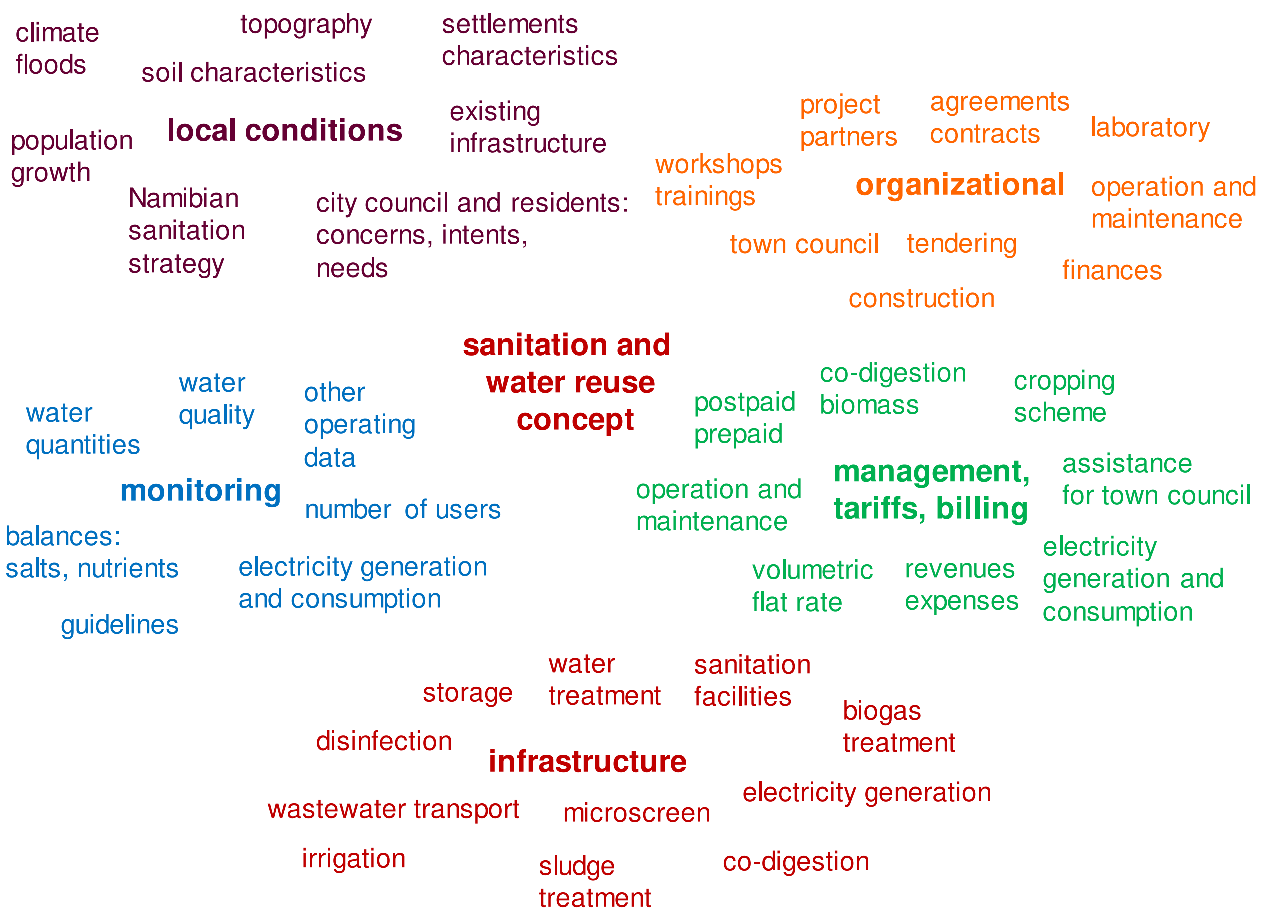

Figure 1 Tag cloud with the main issues dealt with during implementation of the sanitation system in Outapi

In contrast, some issues posed major challenges, because no or only limited information was available from local experts or the published literature that could contribute to decision-making. For instance, irrigation water quality objectives were required for planning wastewater treatment steps and irrigation infrastructure. The effluent of the wastewater treatment plant should be adjusted to provide a water quality that best meets the requirements for the chosen crops, considering the local soil and climatic conditions. However, this proved to be relatively difficult because constituents and concentrations that are of concern in wastewater differ from those in water sources that are usually used as irrigation water, such as surface water or groundwater.

A further example is the use pattern of shared sanitation facilities or the wastewater characteristics that could be expected from shared sanitation facilities in informal settlements. Because no such information was available, the water quantities, loads and concentrations to be expected in the untreated water had to be estimated.

1.4 Research objectives

This dissertation focuses on issues that required further clarification and additional information during the planning and implementation of this project. It addresses very different aspects that, taken together, represented knowledge gaps or impediments to realization of this project. The intention of this dissertation is to outline these aspects and to contribute to filling the encountered knowledge gaps and to demonstrate the implications that the observations may have in practice. This may reduce obstacles to implementation of new concepts in urban water management and maximize the achievable benefits of sanitation systems. The research objectives are:

- To describe the planning process and to investigate the main influencing factors as the basis for the chosen setup of the implemented sanitation system
- To provide, for the first time, data about the wastewater characteristics of shared sanitation facilities as a basis for better planning
- To identify the preconditions under which operation and maintenance cost recovery of shared sanitation facilities would be possible
- To analyze and compare capital costs with comparable infrastructures in the region
- To develop appropriate water quality objectives for water reuse in agricultural irrigation
- To elaborate a nutrient and salt balance for the implemented sanitation system and to discuss and quantify the impact of possible management measures
- To present and discuss data related to the energy consumption of the plant in Outapi
- To identify the main obstacles to the implementation of energetic utilization of organics contained in wastewater and to co-digestion with agricultural residues

Some results of this work have been published in scientific articles and book chapters or were under review when this dissertation was being completed. The contents of Chapter 4.1 "Planning and implementation of wastewater collection, transport and treatment facilities" are partly presented in Müller *et al.* (2017). Some results related to the communal washhouse were published in the Journal of Water, Sanitation and Hygiene for development (Müller *et al.* 2016) at the time of completion of this work. Chapter 4.5 "Quality of the reclaimed water on the water quality criteria" is almost entirely published in Müller and Cornel (2017). At an early stage, some aspects regarding salinity and salt management were published (Müller and Cornel 2015) but subsequently underwent an extensive revision.

1.5 Outline of this work

This chapter introduced the global situation regarding water supply and sanitation, the main challenges regarding planning and implementation of sanitation systems, and the main motivation and objectives of this dissertation and how topics were chosen. The second chapter provides background information on previous and current research. Chapter 3 presents details about the materials and methods and also includes a description of the project that provided the data and topics for this work.

Chapter 4 contains the results and discussion. It starts with an overview of the most important points during planning (Chapter 4.1) and presents the monitored water quantities and concentrations of the wastewater constituents (Chapter 4.2). These data are the basis for the topics addressed in the following sections.

Chapter 4.3 focuses on the untreated water quality and the loads from each type of sanitation facility. The results on the overall water use and loads and the utilization of the shared sanitation facilities are presented. The specific loads and the specific water use are deduced for each type of sanitation facility. The most important points to be considered for planning and implementation of shared sanitation are summarized. A separate section (Chapter 4.4) discusses operation and maintenance and capital costs of the shared sanitation facilities. It examines under which conditions o&m cost recovery would be possible.

Chapter 4.5 focuses on the quality of the reclaimed water and which water quality criteria should be used. Internationally established guidelines were applied to the local context as far as possible. Additional water quality objectives were suggested, where necessary, to meet the requirements of this project in particular and of water reuse projects in general. Emphasis is placed on water quality requirements prior to UV disinfection and on nutrient requirements of cultivated crops.

Chapter 4.6 deals with the salt and nutrient loads and concentrations and the risks of salinization and overfertilization. Planning data are compared with monitoring data. Measures for management of salt and nutrient loads and concentrations are discussed.

Chapter 4.7 of the results and discussion section deals with energetic aspects that arose during planning and implementation. It describes the potential methane yield and electricity generation from agricultural crops and residues and highlights the obstacles experienced during implementation of this project. It includes an overview of the monitored electricity consumption, saving potentials, and the importance of the electricity tariff structure of the local power provider. A comparison with tariff structures in the region is carried out.

Chapter 5 briefly reviews the results, summarizes the main conclusions, assesses the lessons learnt and provides key messages and further research recommendations.

2 Background

This chapter provides a brief survey of the available knowledge on the topics presented in the later sections of this dissertation. It starts with the state of sanitation provision on a global scale and in Sub-Saharan Africa. Because shared sanitation facilities are frequently addressed in the later chapters, a separate section is devoted to this theme, providing a short outline on the essential characteristics of shared sanitation and their contribution to sanitation provision in developing countries.

The next sections continue with the characteristics of municipal sewage (Chapter 2.2), the relevance and application areas of water reuse (Chapter 2.3) and provide an overview of water quality objectives for agricultural irrigation with an emphasis on reclamation of treated municipal wastewater (Chapter 2.4).

Salt and nutrient management was an important issue during implementation of this project. Thus, Chapters 2.5 and 2.6 deal with the importance of salts and nutrients in water reuse schemes, as well as with potential risks and management practices.

The main characteristics of vacuum sewers are presented in Chapter 2.7. The background for the results related to the energetic aspects is provided in Chapter 4.7 that deals with co-digestion of sewage sludge and agricultural residues.

The last part (Chapter 2.9) provides a review of some characteristics of the project region by providing information on water resources, urbanization, existing wastewater infrastructure, the risk of floods and the Namibian electricity sector.

2.1 The gap in sanitation provision and the significance of shared sanitation

91% of the world's population has access to improved drinking water sources, but only 68% has access to improved sanitation (UNICEF and WHO 2015). In Sub-Saharan Africa, only 68% of the population has access to improved drinking water sources, only 30% has access to improved sanitation, 20% uses shared sanitation facilities, 27% uses unimproved sanitation facilities and 23% practices open defecation (UNICEF and WHO 2015). In this region, only 16% of the population has a water connection in the home or on the property (UNICEF and WHO 2015) and about half of the households with a water connection has flush toilets (WHO and UNICEF 2013). Accordingly, waterborne sanitation with sewers is rare in Sub-Saharan Africa. About half of the larger cities in this region has sewer systems at its disposal (Banerjee and Morella 2011). Only in Namibia, South Africa and Senegal do some utilities offer area-wide access to sanitation (Banerjee and Morella 2011).

In 2009, 33% of the population in developing countries lived in informal settlements (UN-HABITAT 2013). This percentage is 62% in Sub-Saharan Africa (UN-HABITAT 2013). In view of this high proportion of people living in informal areas, it is clear that they have to be included in attempts to substantially increase access to sanitation.

Informal settlements are characterized by substandard housing structures (often built with non-permanent materials), high population densities and overcrowding, limited access opportunities (due to, e.g., lack of surfaced roads, decaying buildings), insecure tenure and low income of the population (UN-HABITAT 2003a). It is obvious that, under such conditions, sanitation provision may not be feasible on an individual (household) level. This fact is acknowledged by a number of authors (IWA 2005; Kariuki *et al.* 2003; Mara 2005; Schouten and Mathenge 2010).

One option to improve access to sanitation in these areas is shared sanitation. In cases where individual provision is not possible, this approach can improve the local sanitary conditions by providing a basic level of sanitation (Bond *et al.* 2013; Eales *et al.* 2013; Rheinländer *et al.* 2015; Schaub-Jones 2006; Verhagen *et al.* 2008). Shared sanitation facilities "are proving highly effective, because they concentrate usage in one place and so make sewer connections, management and operation financially viable" (Eales 2008).

Norman (2011)distinguishes household toilets, shared toilets, community toilets and public toilets (Figure 2). Household toilets are affiliated with a single household, shared toilets are assigned to several households in a single building or plot, community toilets are shared by a group of households in a community and public toilets can be used by anybody, because they are located in public spaces (Norman 2011). However, these definitions are not fixed and vary between authors. For instance, WHO and UNICEF (2008) consider public toilets as a type of shared toilet.

In 1990, sharing was common practice for 160 million people or 7% of the world`s urban population and, in 2015, this number had more than doubled, to 394 million people or 10% of the urban population (UNICEF and WHO 2015). In earlier publications of the Joint Monitoring Programme, these numbers were even higher. According to the data in WHO and UNICEF (2013), 205 million people in urban areas were using shared sanitation in 1990 and 470 million in 2011.

Thus, shared sanitation is widely practiced. This is particularly the case in informal settlements (Schouten and Mathenge 2010) and in urban Sub-Saharan Africa, where sharing of sanitation facilities is very common: 30% of the population used shared facilities in 2011 (WHO and UNICEF 2013).

Shared sanitation facilities are not considered improved facilities by the Joint Monitoring Program (UNICEF and WHO 2015; WHO and UNICEF 2013, 2008). If shared sanitation facilities were considered improved, the world would have met the MDG of 77% (68% of the global population with access to improved sanitation plus the 9% using shared sanitation facilities, UNICEF and WHO (2015)). One reason for not considering shared sanitation as improved sanitation is the concern that such facilities may be less hygienic than private household facilities (UNICEF and WHO 2010). Another reason is that the data used by the Joint Monitoring Program do not allow any differentiation among shared sanitation facilities (UNICEF and

WHO 2010). It is acknowledged that this procedure might underestimate the proportion of people using improved sanitation facilities (UNICEF and WHO 2010).

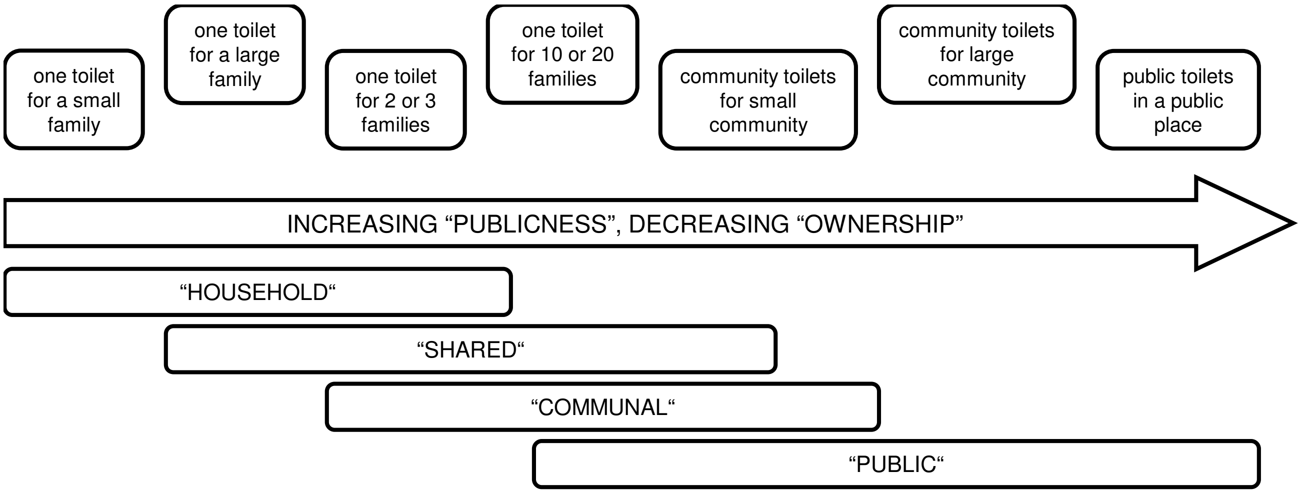

Figure 2 Definition of household, shared, communal and public toilets (Norman (2011), modified)

However, a number of authors recognize the contribution that well-managed shared sanitation facilities can make in providing access to water and sanitation (Bond *et al.* 2013; Cumming *et al.* 2014; Exley *et al.* 2015; Mara 2005; Mosler *et al.* 2014; Nelson and Murray 2008; Norman 2011; Schouten and Mathenge 2010; Sijbesma 2011). Hence, the overall classification of shared sanitation facilities as being unimproved is questioned. Where individual sanitation is not feasible on a household level, shared sanitation facilities are an option for providing such services to the residents.

There are a number of initiatives that offer shared communal or public facilities to urban areas, using a comprehensive approach and emphasizing successful operation, maintenance and funding. In some Indian cities, die NGOs Sulabh and SPARC (Society for the Promotion of Area Resource Centers) serve public places and poor residential areas with shared sanitation facilities (Burra *et al.* 2003; Colin and Nijssen 2007; Pathak 1999). In Indonesia, the government funds community-managed decentralized wastewater treatment systems. 77% are community sanitation centers providing water and sanitation services for 20 to 100 households (Eales *et al.* 2013). In Kenya, Umande Trust implements “BioCentres”. These are community latrine blocks which are owned, built and operated by the communities (Aubrey 2009). Another approach, also located in Kenya, involves “Ikotoilets” (Karugu 2011; Njeru 2014; Ziegler *et al.* 2013). Besides sanitation services, Ikotoilets integrate other services, such as cold refreshments with snacks and newspaper vending. The NGO WSUP (Water and Sanitation for the Urban Poor) has supported implementation of public and communal sanitation facilities in Madagascar, Kenya, Mozambique and India (Norman 2011). In South Africa, the eThekwini municipality is introducing community ablution blocks (CABs) as an interim sanitation service for upgrading informal settlements (Crous *et al.* 2013; DHS 2009).

The vast majority of existing shared sanitation facilities is badly managed (Collignon and Vézina 2000; UN-HABITAT 2003b; WHO and UNICEF 2008). Whilst implementation of the physical structures is feasible, the main challenge is long-term sustainability and, particularly, the establishment of effective maintenance structures (Collignon and Vézina 2000; Nelson and Murray 2008; Verhagen *et al.* 2008). If these challenges were to be overcome, shared sanitation facilities could substantially increase sanitation coverage in regions that cannot be serviced by conventional approaches: "Without reconsidering shared sanitation, the MDG, and future targets, are unlikely to be met" (Exley *et al.* 2015).

2.2 Characteristics of domestic sewage

When planning wastewater transport and treatment facilities, information about wastewater quantities, flow rates, and constituent loadings is necessary (Tchobanoglous *et al.* 2004). Thus, input data about the expected number of users, specific water uses, and loads of common wastewater constituents have to be specified.

2.2.1 Water quantities

The average water use per person per day can be as high as 575 L/(person×d) in the United States, or between 200 and 300 L/(person×d), as in most European countries, down to less than 50 L/(person×d) in, e.g., Angola, Mozambique or Ghana (UNDP 2006). Because access to water is limited in informal settlements, the per capita water use is even lower. Investigations by Uhlendahl *et al.* (2010) in informal settlements in Windhoek report a water use of 27 L/(person×d). For informal settlements in Outapi, a similar water use of 35 L/(person×d) was reported in community workshops conducted by Deffner and Mazambani (2010).

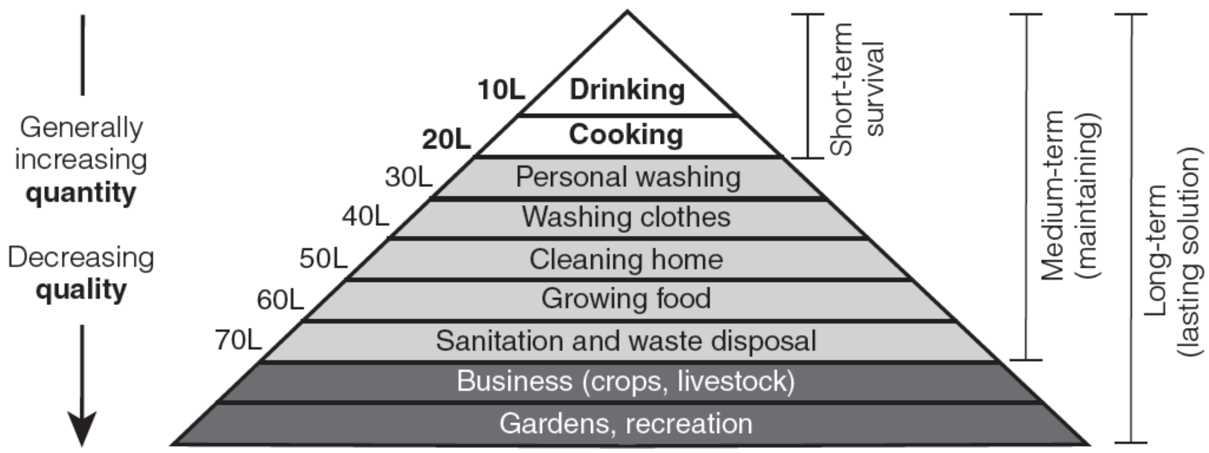

Figure 3 Hierarchy of water requirements (WHO and WEDC 2011)

Gleick (1996) gives recommendations on minimum water quantities required per person per day for human needs: 5 L to maintain physiological processes (in tropical and subtropical climates), 15 L for washing/bathing, 10 L for food preparation and 20 L for waste disposal and related hygiene; thus 50 L/(person×d) in total. WHO and WEDC (2011) consider 70 L/(user×d) as a minimum water requirement for domestic needs (Figure 3).

2.2.2 Overview of the main constituents

Wastewater constituents are unequally distributed between urine, feces and greywater (Figure 4). Most of the total chemical oxygen demand (TCOD), biochemical oxygen demand (BOD) and total solids (TS) are contained in greywater and feces. Total nitrogen (TN), total phosphorus (TP) and potassium (K) are mainly contained in urine.

TS can be further divided into non-filterable total suspended solids (TSS) and filterable total dissolved solids (TDS) (Sperling 2007c). The mineral compounds cannot be oxidized by heat (fixed fraction). Organic compounds can be oxidized (volatile fraction) (Sperling 2007c).

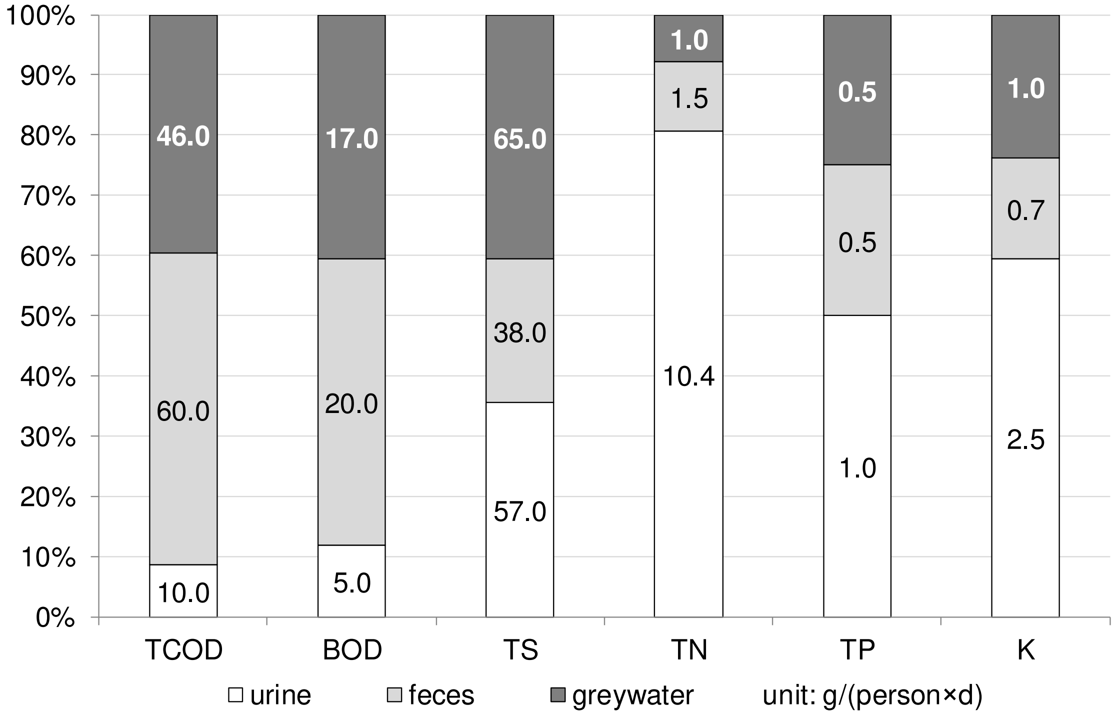

Figure 4 Distribution of wastewater constituents between urine, feces and greywater (Meinzinger and Oldenburg (2009), modified)

The main components of organic matter are proteins, carbohydrates and lipids (Sperling 2007c). Organic matter can be determined either indirectly (e.g. via the BOD or COD) or directly (e.g. TOC = total organic carbon) (Sperling 2007c). The BOD is usually measured at 5 days and 20°C (BOD_5) (Sperling 2007c). It represents the oxygen consumption of microorganisms for degradation of the organic matter in the water sample (Sperling 2007c).

The TCOD and dissolved COD (DCOD) represent "the quantity of oxygen required to chemically stabilize the carbonaceous organic matter" (Sperling 2007c). The TOC is obtained by measuring the CO_2 from converted organic carbon (Sperling 2007c).

TN includes organic nitrogen, ammonium nitrogen, nitrite and nitrate nitrogen (Tchobanoglous *et al.* 2004). In untreated wastewater, roughly 60% of the TN is contained in the form of ammonium nitrogen and 40% in the form of organic nitrogen (Sperling 2007c; Tchobanoglous *et al.* 2004). Nitrate nitrogen can amount to 3% of the total nitrogen (Sperling 2007c).

Untreated wastewater contains phosphorus in the forms of orthophosphate, polyphosphate and organic phosphate (Tchobanoglous *et al.* 2004). Orthophosphate constitutes about 64% of the TP (Henze 2008). In the sewers and during wastewater treatment, polyphosphate is slowly converted to orthophosphate via hydrolysis (Sperling 2007c).

A huge variety of organisms can be present in wastewater (Tchobanoglous *et al.* 2004). Some of the bacteria, protozoa, helminths and viruses may be pathogenic (Tchobanoglous *et al.* 2004). They are added to the water because they are contained in excreta from infected humans (and possibly animals) (WHO 1989).

The survival times of pathogens range between less than two days for e.g. protozoa and *V. cholerae* on crops up to years for *Ascaris* eggs in water and soil (Table 1). These pathogens have to be managed when reusing water or whenever there is a certain health risk for humans.

It is possible to manage pathogens at each stage of a water reuse scheme (Carr *et al.* 2004). Measures may be taken during wastewater treatment, storage, irrigation water delivery, for crops on the field or in the form of human exposure control, chemotherapy and vaccination (Carr *et al.* 2004) or produce washing, peeling and cooking (WHO 2006). A multiple barrier approach means that pathogens are reduced or harmful contact with humans is prevented at several points (Carr *et al.* 2004).

The vast number of human pathogens has led to the development of the indicator concept (Cooper and Olivieri 1998). Indicator organisms are used in microbiological water analysis to indicate the presence of pathogens (Cooper and Olivieri 1998). They should only be present when fecal contamination occurs, not reproduce outside the host, have roughly the same die-off as the respective substituted pathogen and should be easily and quickly determinable (Cooper and Olivieri 1998; Havelaar *et al.* 2001). Total coliforms, fecal coliforms and *E. coli* are most frequently used as indicator organisms in reclaimed water (Paranychianakis *et al.* 2015). Other indicator organisms for fecal contamination are *Klebsiella*, fecal streptococci, Enterococci, *Clostridium perfringens* and *Pseudomonas aeruginosa* (Tchobanoglous *et al.* 2004). *E. coli* is used in the WHO (2006) guidelines and considered best suited to represent fecal contamination (Paranychianakis *et al.* 2015).

Total coliform bacteria are gram-negative, rod-shaped bacteria (Bartram and Ballance 1996). In microbiological analyses, they are identified by fermentation of lactose in 24 h to 48 h at 35°C (Tchobanoglous *et al.* 2004). They include thermotolerant coliforms, bacteria from fecal origin but also bacteria occurring in e.g. soil (Bartram and Ballance 1996). Thermotolerant coliforms ferment lactose at temperatures between 44°C or 44.5°C (Bartram and Ballance 1996). They also include non-fecal organisms, but are sometimes referred to as fecal coliforms

(Bartram and Ballance 1996). 95% of the thermotolerant coliforms in water are *E. coli* (Bartram and Ballance 1996).

Enterococci is the collective term for the two human-specific strains of fecal streptococci: *Streptococcus faecalis* and *Streptococcus faecium* (Tchobanoglous *et al.* 2004). They have a higher survival rate in water than thermotolerant and total coliforms (Bartram and Ballance 1996). Enterococci and *E. coli* always indicate fecal contamination (Bartram and Ballance 1996).

Table 1 Survival times of viruses, bacteria, protozoa and helminths in water, soil and on crops (various sources compiled in WHO (2006))

	survival times (days)		
organism	**fresh water and sewage**	**crops**	**soil**
viruses			
enteroviruses	< 120 and usually < 50	< 60 and usually < 15	< 100 and usually < 20
bacteria			
thermotolerant coliforms	< 60 and usually < 30	< 30 and usually < 15	< 70 and usually < 20
Salmonella spp.	< 60 and usually < 30	< 30 and usually < 15	< 70 and usually < 20
Shigella spp.	< 30 and usually < 10	< 10 and usually < 5	no data
V. cholerae	no data	< 5 and usually < 2	< 20 and usually < 10
protozoa			
E. histolytica cysts	< 30 and usually < 15	< 10 and usually < 2	< 20 and usually < 10
Cryptosporidium oocysts	< 180 and usually < 70	< 3 and usually < 2	< 150 and usually < 75
helminths			
Ascaris eggs	years	< 60 and usually < 30	years
tapeworm eggs	many months	< 60 and usually < 30	many months

2.2.3 Specific loads

Table 2 gives an overview of daily per capita loads of wastewater constituents published in the literature. Specific loads for the total chemical oxygen demand (TCOD) range from 93.9 g/(person×d) in Sweden to 190 g/(person×d) in the USA. The BOD ranges from 34.2 g/(person×d) in Egypt to 80.0 g/(person×d) in the USA. Total nitrogen (TN) varies from 5.9 g/(person×d) to 13.7 g/(person×d). Total phosphorus (TP) ranges from 1.0 g/(person×d) in developing countries to 4.1 g/(person×d) in the USA (in the 1970s). Total solids (TS) vary between 105 g/(person×d) and 212 g/(person×d). Total suspended solids (TSS) range from 35.0 g/(person×d) to 90.0 g/(person×d) and total dissolved solids (TDS) are about 120 g/(person×d).

Dietary habits influence the specific loads. For instance, vegetal food stuffs contain twice as much phosphorus and more than five as much potassium per gram of protein than animal food stuffs (Jönsson and Vinneras 2004). The distribution of the nutrients between urine and feces is determined by the digestibility of the food (Jönsson and Vinneras 2004). Digested substances tend to end up in urine. Undigested substances are excreted via feces. Hence, if a population consumes a high proportion of highly processed food, a larger proportion of the nutrients is excreted via urine (Jönsson and Vinneras 2004). If a population primarily consumes food with lower digestibility, more fecal matter is excreted (Jönsson and Vinneras 2004).

In general, most values available in the literature refer to Europe or North America. DWA (2008c) and Meinzinger and Oldenburg (2009) summarize median values from 148 citations on per capita loads in wastewater; these mainly represent European data. Other frequently cited literature such as Siegrist *et al.* (1976), Crites and Tchobanoglous (1998) and USEPA (1992b) refer to studies conducted in the 1960s and 1970s in the USA. Values given in, e.g., Henze (1997) and Jönsson *et al.* (2005) can be tracked back to Scandinavian and German publications. Sperling (2007c) reports typical per capita loads for "raw domestic sewage in developing countries" that were derived from several sources but cannot be reproduced. WRC (1993) provides some assumptions for human wastes in developing urban areas in South Africa. Henze *et al.* (1997) give per capita loads for several countries, including two African examples (Egypt and Uganda) without further information on the origin of these data.

Table 2 Literature values for TCOD, BOD, TN, TP, TS, TSS and TDS input during water use, unit: g/(person×d), BOD = BOD_5 except for Sweden = BOD_7

reference	loads (g/(person×d)) TCOD	BOD	TN	TP	TS	TSS	TDS
Sperling (2007c) (typical value, developing countries)	100	50.0	8.0	1.0	180	60.0	120
WRC (1993) (assumption, Southern Africa)	100	no data	10.0	2.5	no data	no data	no data
Henze *et al.* (1997) (Egypt)	no data	34.2	11.0	1.4	no data	54.8	no data
Henze *et al.* (1997) (Uganda)	no data	61.6	11.0	1.4	no data	47.9	no data
DWA (2008c) (median, Central Europe)	117	43.0	12.9	2.0	166	no data	no data
ATV-DVWK (2000) (85th percentile, Germany)	120	60.0	11.0	1.8	no data	70.0	no data
Crites and Tchobanoglous (1998) (typical value without ground-up kitchen waste, USA)	190	80.0	13.0	3.2	212	90.0	120
Siegrist *et al.* (1976) (households with typical appliances but omitting garbage disposal, USA)	no data	49.5	5.9	4.1	no data	35.0	no data
USEPA (1992b) (typical residential wastewater, USA)	120	42.5	11.5	1.5	143	42.5	no data
Jönsson *et al.* (2005) (household wastewater, Sweden)	93.9	53.6	13.7	1.9	105	39.3	no data

2.2.4 Water quantities and loads from shared sanitation facilities

There is a sufficient data basis that can be used for estimating the daily per capita water use and loads of common wastewater constituents. When providing individual sanitation, it can be expected that almost all of the water plus excreta is collectable. For shared sanitation, the future number of users and the ultimate use of the provided facilities are less definite, because some individuals might use them more often or for other purposes than others. Hence, water use and constituent loadings will certainly differ from typical values for individual sanitation facilities.

However, only a limited number of publications includes information on utilization rates and water use in shared sanitation facilities. The majority of these documents was not available when planning of the project referred to in this dissertation started in 2009.

Examples of shared sanitation from Kenya report mean utilization rates of 416 users per day (range: 50 to 625, Schouten and Mathenge (2010)) and 600 users per day (Lüthi *et al.* 2011a). Biran *et al.* (2011) report about 482 users per day for facilities in India (range: 124 to 896). For an example in Madagascar, 220 "defecations per day" are given (Norman 2011). These examples include facilities with differing equipment; thus, the number of uses is also referred to the number of installed toilets (Table 3). Then, the mean utilization rate is 59 uses per toilet and day (range: 13 to 156).

Table 3 Utilization of shared sanitation facilities in Kenya and India

	Kibera, Kenya Schouten and Mathenge (2010)							**Bhopal, India Biran *et al.* (2011)**						
facility no.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**1**	**2**	**3**	**4**	**5**	**6**	**7**
number of toilets	4	5	8	9	4	6	4	12	4	8	14	20	15	20
uses/d	500	600	450	575	625	110	50	896	124	554	465	556	343	435
uses/(toilet×d)	125	120	56	64	156	18	13	75	31	69	33	28	23	22

Biran *et al.* (2011) monitored utilization during one day at 7 facilities and provide some details on how data collection was carried out. The other examples do not provide such information. Neither long-term monitoring data nor data that could explain variations or trends in utilization have been published. Factors that influence utilization, such as the characteristics of the sanitation facilities (e.g. tariffs, opening hours), or characteristics of the settlement (e.g. population density, income situation of the residents) are not addressed. Also, a distinction between the number of users and the number of uses is not made. It must be assumed that even though the number of users is given, the authors actually mean the number of uses per day, which does not allow conclusions about the actual number of users.

Water quantity data are only available for four facilities in South Africa (Crous *et al.* (2013), see Chapter 4.3.1.1, page 93). Data on anticipated loads or concentrations of wastewater from shared sanitation facilities are not available in any publication.

A review of issues to be considered for implementation is given by Norman (2011). Some well-documented case studies can provide guidance for planning shared sanitation facilities, e.g., regarding the institutional context and organizational issues (Biran *et al.* 2011; Biran and Jenkins 2010; Burra *et al.* 2003; Cousins 2004; Hobson 2000; Mazeau *et al.* 2014; Mazeau 2013; Roma and Jeffrey 2010; Schouten and Mathenge 2010; Tumwebaze and Mosler 2014; WaterAid India 2008). To date, research mainly provides knowledge on general feasibility, institutional and sociological aspects and appropriateness of shared sanitation facilities. However, for planning and implementation in a more comprehensive context, further information about expected wastewater quantities, concentrations and loads is necessary.

2.3 The relevance of water reuse

The global water resources are scarce and the continued growth in demand for water, especially for food production represents an increasing challenge (UNESCO 2015). It affects mostly the population in developing and emerging countries, in arid or semi-arid regions and in densely populated regions, particularly in the rapidly growing megacities (UNDP 2006).

Water reclamation and water reuse open up new water resources, reduce the need for fresh water and the discharge of (treated) wastewater into surface waters (Cornel *et al.* 2011). In regions where the water supply due to long transport routes, pumping or high treatment costs is energy intensive and expensive, the reuse of adequately treated water is an alternative with lower energy consumption and lower cost as compared to the use of fresh water (Cornel *et al.* 2011). In addition, valuable freshwater resources such as high quality groundwater are protected by the alternative use of reclaimed water (Cornel *et al.* 2011).

There are several factors triggering water reuse. Physical factors are defined by the characteristics of the local environment and social factors arise from the public (Jimenez and Asano 2008). Economic and political factors also play a role (Jimenez and Asano 2008). The main drivers for water reuse are therefore water scarcity, droughts, the need for a reliable water source, spatial proximity of generated wastewater and irrigation demand, increasing water demand due to urbanization and growing domestic and industrial consumption, the necessity or wish to protect the water quality and quantity in water bodies and wetlands, favorable and/or stringent guidelines and regulations as well as changes in wastewater treatment philosophy and the need for reducing treatment costs (Bischel *et al.* 2012; Jimenez and Asano 2008; Kellis *et al.* 2013; Miller 2006; Paranychianakis *et al.* 2015; Raschid-Sally and Jayakody 2009).

Possible reuse applications are summarized in Table 4. They include urban reuse for e.g. irrigation of public parks, golf courses, landscaped areas, but also fire protection, dust control, toilet and urinal flushing and use for decorative water features (USEPA 1992a).

Industrial reuse can be facilitated in the form of in-plant recycling but also by reuse of reclaimed municipal water (USEPA 1992a). The main uses are for evaporation and cooling water, boiler-feed water, process water and irrigation and maintenance of plant grounds (USEPA 1992a).

Recreational and environmental reuse includes use of the reclaimed water for maintenance and landscape ponds or sites for swimming, fishing and boating but also snowmaking, stream augmentation and wetlands (USEPA 1992a).

Groundwater recharge with reclaimed water is carried out to provide a barrier for saltwater intrusion into the aquifer, to provide further treatment of the water for future reuse, to augment the potable or nonpotable aquifers, to store the water in the aquifer or to control or prevent ground subsidence (USEPA 1992a).

Water reuse in agriculture may include irrigation of food crops, processed food crops and non-food crops (USEPA 2012). Agriculture is the largest water consumer. Globally, 69% of the freshwater withdrawals are used for agricultural irrigation (Figure 5). This percentage varies among regions. It is relatively low in Europe (22%) and Northern America (43%) and relatively high in Asia (81%), Northern Africa (84%) and Sub-Saharan Africa (80%).

Table 4 Possible water reuse applications (USEPA (2012), modified)

category of reuse		**description**
urban reuse	unrestricted	the use of reclaimed water for nonpotable applications in municipal settings where public access is not restricted
	restricted	the use of reclaimed water for nonpotable applications in municipal settings where public access is controlled or restricted by physical or institutional barriers, such as fencing, advisory signage, or temporal access restriction
agricultural reuse	food crops	the use of reclaimed water to irrigate food crops that are intended for human consumption
	processed food crops and non-food crops	The use of reclaimed water to irrigate crops that are either processed before human consumption or not consumed by humans
impoundments	unrestricted	The use of reclaimed water in an impoundment in which no limitations are imposed on body-contact water recreation activities
	restricted	The use of reclaimed water in an impoundment where body contact is restricted
environmental reuse		The use of reclaimed water to create, enhance, sustain or augment water bodies including wetlands, aquatic habitats or stream flow
industrial reuse		The use of reclaimed water in industrial applications and facilities, power production and extraction of fossil fuels
groundwater recharge	nonpotable reuse	The use of reclaimed water to recharge aquifers that are not used as potable water source
potable reuse	indirect	Augmentation of a drinking water source (surface or groundwater) with reclaimed water followed by an environmental buffer that precedes normal drinking water treatment
	direct	the introduction of reclaimed water (with or without retention in an engineered storage buffer) directly into a water treatment plant, either collocated or remote from the advanced wastewater treatment system

Agricultural irrigation is the most important water reuse application and still offers a high potential for reusing reclaimed water (Jimenez and Asano 2008). There are no comprehensive studies on applied water quantities or information on irrigated areas worldwide available but it is estimated that 1.5 to 6.6% of the agricultural area is irrigated with (possibly treated) wastewater (Sato *et al.* 2013).

Water reuse is mainly practiced in regions with climate-related water shortages (Jimenez and Asano 2008). Already 5,000 years ago water reuse was practiced during periods of water shortage e.g. on Crete (Angelakis *et al.* 1999). In the 19th century and the beginning of the 20th century, land application of sewage was widely practiced as a disposal method in the major cities of Europe (Roccaro *et al.* 2014), the USA (Asano 1998) and Australia (Stevens *et al.* 2006).

In 1918, the US state of California published the first set of rules with limits for water reuse for irrigation (Crook 1998). In the US, water reuse was practiced at first in the drier states (e.g., California and Colorado) due to increasing urbanization, population growth and more stringent requirements on wastewater treatment (effluent water quality) (Tchobanoglous *et al.* 2004). Since the 1970s, this approach was also carried out in Florida due to the high population growth and increased need to protect the local water resources (Tchobanoglous *et al.* 2004).

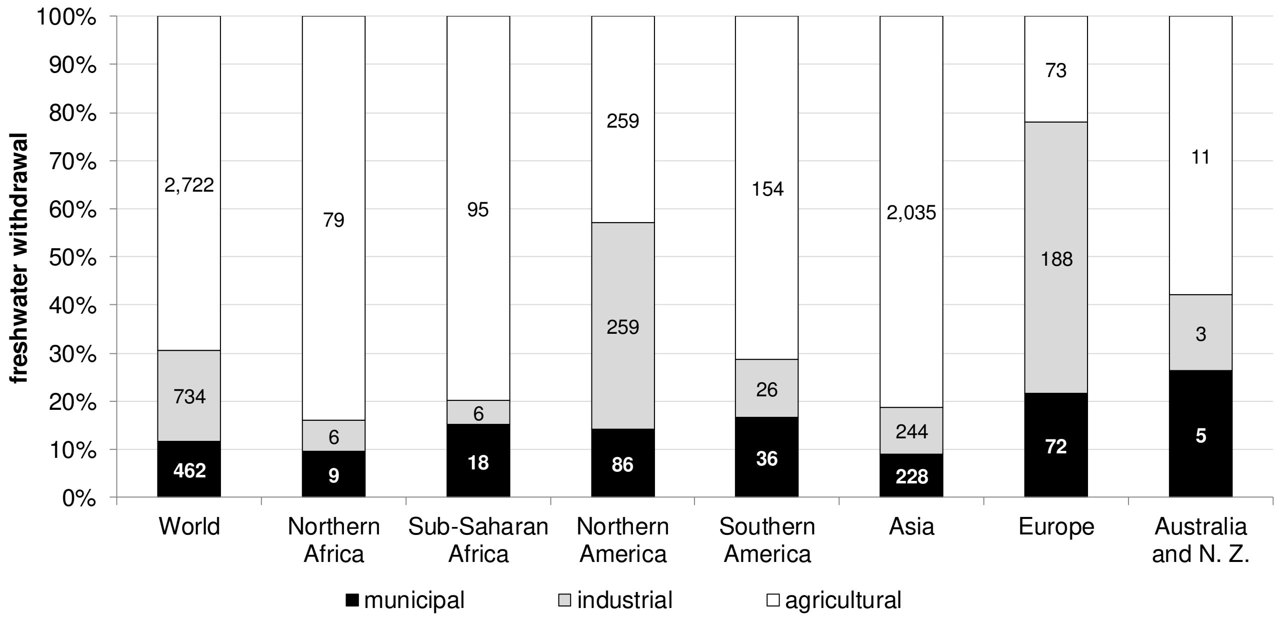

Figure 5 Freshwater withdrawals by sector, unit: km³/a, N. Z. = New Zealand (FAO 2014a)

In Europe, only a few projects for the reuse of treated municipal wastewater are known from the early 1990s, whereas in 2006 the number of water reuse projects identified by the EU project AQUAREC was more than 200 (Bixio *et al.* 2006). Similar to the aforementioned developments in the United States also the European projects were mainly implemented in (coastal) areas marked by drought and on islands of southern Europe and further north in urbanized areas of the humid Europe (Bixio *et al.* 2006).

Figure 6 presents an overview of water reuse projects worldwide. The basis for this compilation is project information retrieved from databases and the literature, as well as consultations with national experts (Bixio *et al.* 2006). The focus was on technologically advanced projects. The small number of water reuse projects in some regions (e.g., Sub-Saharan Africa, Latin America) can be explained by the fact that technically advanced systems for wastewater collection and treatment are largely lacking in these areas.

This is also the case for Namibia and South Africa. However, some technically advanced water reuse projects do exist. In Windhoek, the waters of the Goreangab dam and the Gammams Wastewater Treatment Plant have been treated for potable reuse in the Goreangab Reclamation

Plant since 1969 (Lahnsteiner and Lempert 2007; Pisani 2006). In South Africa, there are several cases where municipal and industrial wastewater is collected from larger areas, treated, and used for irrigation (eThekwini metropolitan authority, Kwazulu-Natal Province, and City of Cape Town in the Western Cape Province) (Adewumi *et al.* 2010). Adewumi *et al.* (2010) report further examples for water reuse within individual buildings in two South African cities (Carnarvon and Kimberly) and water reuse for garden irrigation and toilet flushing in several buildings in another settlement (Lynedoch Eco-Village).

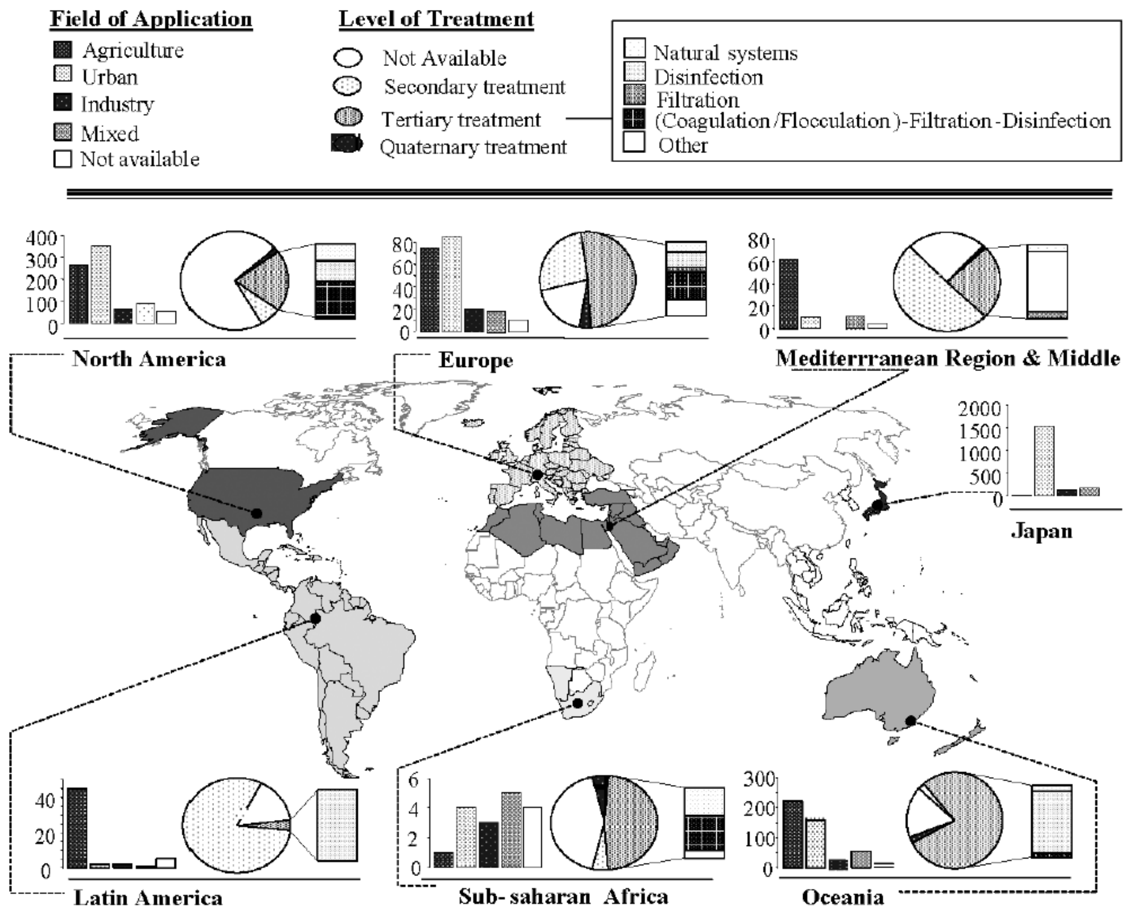

Figure 6 Number of water reuse projects worldwide with applications and type of treatment (Bixio *et al.* 2005)

The available official data on wastewater treatment and reuse are sparse, particularly in Sub-Saharan Africa (Sato *et al.* 2013). Of the 48 Sub-Saharan African countries studied, data regarding wastewater, treatment and reuse were not available for 32 countries, partial information was available for 13 countries and complete (but outdated) information was available for only 3 countries (Sato *et al.* 2013).

The use of untreated wastewater for irrigation is the simplest and most problematic approach to water reclamation. However, this is common practice in emerging and developing countries (Raschid-Sally and Jayakody 2009). Urbanization, population growth, and drought trigger water reuse whenever wastewater (treated or untreated) is the only available water resource (Raschid-Sally *et al.* 2005; Tchobanoglous *et al.* 2004). Under such circumstances, it is very commonly used for agricultural irrigation, often in the form of small-scale agricultural activities and subsistence agriculture (Raschid-Sally *et al.* 2005; Sato *et al.* 2013). Sato *et al.* (2013) mention examples in the peri-urban areas around Kumasi (Ghana), Dakar (Senegal), Bulawayo (Zimbabwe), and Nairobi (Kenya).

In principle, water reuse plants do not differ from conventional wastewater treatment plants. They differ mainly in terms of the quality requirement for the treated effluent: for instance, whether it is intended for, e.g., agricultural irrigation or toilet flushing and not for discharge. Other requirements depend on the local conditions, e.g., existing regulations, hygienic aspects, economic aspects, requirements for the operating personnel, and (irrigation) technology and therefore differ from case to case.

With regard to selecting appropriate technology options in the field of wastewater collection and treatment, an abundance of review literature and other tools that can be used for planning in developed and developing countries exists (e.g., Joksimovic *et al.* (2008), Tilley *et al.* (2008), GTZ (2002), Holt and James (2006), Peal *et al.* (2010), IWA (2005), Staben (2008), DWA (2009), Werner *et al.* (2003), USEPA (1992b), Lüthi *et al.* (2011b)).

The AQUAREC research project investigated the technology options or treatment steps most commonly applied in water reuse projects and developed a set of typical or standard schemes. In agricultural water reuse, the following concepts were identified as being representative for the majority of water reuse projects (AQUAREC 2006b; Bixio *et al.* 2005):

- Conventional activated sludge process + disinfection (e.g., chlorination or UV radiation): chlorination is, worldwide, the most commonly used method for disinfection (applied in 85% of the investigated cases); prior to disinfection additional coagulation/flocculation, sedimentation and filtration steps may be applied, after which the water can be used for restricted or unrestricted agricultural irrigation (depending on the treatment chain)
- Soil aquifer treatment: after conventional activated sludge treatment (with nitrogen and phosphorus removal), the effluent percolates into the soil and the re-extracted water can be used for irrigation
- Oxidation ponds: depending on the configuration and kind of oxidation pond, the water may be suitable for restricted or for unrestricted irrigation; in some cases, the pond effluent is chlorinated; the disadvantages of oxidation ponds are the huge space requirement and greenhouse gas emissions
- Constructed wetlands: conventionally treated wastewater (phosphorus and nitrogen eliminated) passes a constructed wetland; in most cases, the effluent is reused for environmental conservation but may also be used for agricultural irrigation

2.4 Water quality objectives for irrigation

Monitoring the quality of irrigation water is necessary to protect human health, soil, plants and water bodies and to prevent the deterioration of irrigation infrastructures (Ayers and Westcot

1985). Furthermore, regular sampling and water analyses are required to collect routine operating data of wastewater treatment and water reclamation plants and to evaluate wastewater treatment processes (Tchobanoglous *et al.* 2004).

In many countries, water quality objectives are defined in national standards (Gurel *et al.* 2007; Havelaar *et al.* 2001; Paranychianakis *et al.* 2015). Where they do not (yet) exist, international guidelines of the WHO (2006, 1989) and the FAO (Ayers and Westcot 1985; Pescod 1992) or other well-established regulations (e.g., State of California (2015), USEPA (2012)) are used to develop national standards (Blumenthal *et al.* 2000; Gurel *et al.* 2007). Most water reuse guidelines and related publications focus on public health issues, whereas environmental protection (eutrophication, salinization, adverse effects of trace elements and trace organic compounds) plays a minor role in the literature (Paranychianakis *et al.* 2015).

Among developing countries, roughly half do not have regulations regarding irrigation with treated wastewater (Raschid-Sally and Jayakody 2009). Lack of legislation is a major obstacle for the implementation of water reuse projects (Angelakis *et al.* 1999; Hochstrat *et al.* 2008; Miller 2006). When realizing water reuse projects in countries without official regulations, either the implementing stakeholders (for instance, a municipality or NGO) need to formulate their own water quality objectives or existing guidelines can be used to assist in monitoring the water quality.

The FAO and WHO guidelines (Ayers and Westcot 1985; WHO 1989) have been widely incorporated into national regulations and are considered suitable for developing countries (Crook 1991; Havelaar *et al.* 2001; Paranychianakis *et al.* 2015). The FAO (1985) guidelines (Ayers and Westcot 1985) give recommendations on physical and chemical water quality objectives to prevent harmful effects on soil, plants and irrigation equipment. However, they do not specifically address the use of treated wastewater for agricultural irrigation. The presented water quality parameters differ from the range of parameters commonly used in wastewater treatment. A subsequent publication (Pescod 1992) is targeted towards the reclamation of treated wastewater for agricultural irrigation, but contains the same recommended limits as in Ayers and Westcot (1985). For instance, although it is acknowledged that organics contained in the water may lead to the clogging of drip irrigation systems, no recommendation on acceptable maximum values is given for aggregate organic constituents (e.g., BOD_5 or TCOD).

WHO (2006) contains recommendations for the required $\log_{10}$ reduction of indicator organisms to achieve a specific health-based target (HBT). The recommended HBT is an additional burden of disease of $\leq 10^{-6}$ DALYs per person per year. Disability-adjusted life years (DALYs) are “calculated as the present value of the future years of disability-free life that are lost as the result of the premature deaths or cases of disability occurring in a particular year” (The World Bank 1993). Health impairment through the reuse of water (e.g., by consumption of irrigated crops) may not exceed this value. This goal can be achieved with a combination of measures.

The WHO (2006) guidelines provide a comprehensive framework for monitoring microbial water quality in agricultural water reuse. However, they have not been used intensively in the development of national standards since their release (Paranychianakis *et al.* 2015). Two examples for the adoption of the WHO (2006) guidelines or the use of DALYs for setting HBTs are available. These are the Ghanaian guidelines for agricultural irrigation (Amponsah *et al.* 2015) and the Australian guidelines for water recycling (NRMMC *et al.* 2008).

2.5 Salts in irrigation water

The salt content is the most important parameter when evaluating the suitability of water for agricultural irrigation (Tchobanoglous *et al.* 2004). Salts need to be controlled to prevent their accumulation in soil and to maintain salt concentrations that do not cause yield loss (Ayers and Westcot 1985). In the long run, sustainable agricultural land management of irrigated areas is only possible if salts are removed from the root zone (Ayers and Westcot 1985).

Dissolved salts in irrigation water can cause water stress in plants (i.e., water is present in the soil but not available) and high concentrations of specific ions can have toxic effects and cause ionic imbalances in plant cells (Mengel 2001). Ions usually considered to cause salinity in soils are Na^+, Ca^{2+}, Mg^{2+}, K^+, Cl^-, SO_4^{2-}, HCO_3^- and CO_3^{2-} (Gauch 1972). N and P compounds usually only occur in excess in soils in connection with overfertilization (Gauch 1972).

Salinity is a measure for the content of dissolved salts in water (Eaton and Franson 2005). The exact salt content can only be determined by a complete chemical analysis (Eaton and Franson 2005). Since this procedure is very time consuming, salinity is often determined by drying and weighing of water samples or by use of a surrogate parameter, such as the electrical conductivity (EC), density, sound speed or refractive index (Eaton and Franson 2005). When using a surrogate parameter, the empirical relationship between salinity and the chosen parameter has to be known (Eaton and Franson 2005). The EC of water is “a measure of the ability of a solution to conduct an electrical current” (Tchobanoglous *et al.* 2004). Because the EC increases with increasing concentrations of ions in the water, it is very often used as a surrogate to express the amount of salts (Eaton and Franson 2005; Tchobanoglous *et al.* 2004).

In areas where rainfall is at least 500 mm per year and occurs in a relatively short period of time (a few months), infiltrating water is usually sufficient to leach salts from the soil (Ben-Hur 2004; Mechilia 2002). This might not be the case for areas where rainfall is low (Ayers and Westcot 1985; Letey 2000; Tanji and Wallender 2012). Then, leaching and drainage are a necessity for sustainable agricultural irrigation (Tanji and Wallender 2012). In regions with insufficient rainfall, around 25% of the irrigated areas are affected by soil salinization (FAO 2002).

Strategies for soil salinity control in agriculture usually only include measures on the agricultural fields. Possible measures are (FAO 2003; Pescod 1992):

- Removal of salts from soil: improvement of drainage, additional leaching, salts removed by harvested crops

- Reduction of salt input on fields: optimization of the irrigation system (e.g., drip irrigation instead of sprinkler irrigation), irrigation scheduling, evaporation reduction, fertilizer management, blending of the irrigation water
- Measures involving crops and soil: cultivation of salt resistant crops, adjustment of planting procedures, chemical treatment of soil

In water reuse schemes, salinity reduction can be carried out prior to wastewater treatment by source separation of urine or, during wastewater treatment, via operation of ion exchangers, electrodialysis or membrane filtration (reverse osmosis or nanofiltration) (Tchobanoglous *et al.* 2004). It has to be kept in mind that nutrients are also reduced. Application areas, potentials, advantages and disadvantages of these technologies are summarized in Norton-Brandao *et al.* (2013):

- Ion exchangers and electrodialysis are not used for reclamation of irrigation water. Ion exchangers have a low selectivity for monovalent ions, which leads to a lower desalination efficiency. Frequent regeneration of resins is required. Fouling in electrodialysis is a substantial issue.
- Nanofiltration has the disadvantage of only rejecting divalent ions, which leads to a higher sodium adsorption ratio in the irrigation water because monovalent ions such as Na^+ are not removed. Hence, the reclaimed water may have a negative effect on soil infiltration (see Section 4.5.2.6, page 132).
- Reverse osmosis membrane filtration allows removal of 90% of the TDS load. Most of the nutrients are removed and are not usable for fertilization. TSS and microbes lead to fouling of the membrane; thus, pretreatment of the water is required. This option is expensive in terms of capital and operational costs.

2.6 Nutrients in irrigation water

A nutrient is defined as "any substance that is used by an organism to provide nourishment, and to build and repair tissues" (Chesworth 2008). Macronutrients (C, H, O, N, P, K, Ca, Mg and S) are required by plants in relatively large quantities and constitute at least 0.1% of the dry weight of the plant (Chesworth 2008). Micronutrients (Zn, Fe, Mn, Cu, Mo, B, Cl) usually constitute less than 0.05% of the dry weight (Chesworth 2008).

If a plant is supplied with limited nutrients or an otherwise limiting factor, the yield will increase (Schubert 2006). As a general rule, every growth factor can become a stress factor if delivered in excess (Schubert 2006). For each nutrient contained in plants, there is a certain minimum concentration that needs to be exceeded in order to avoid deficiency and there is a maximum concentration that marks the beginning of toxic levels (Havlin *et al.* (2004), Figure 7, Figure 8).

At excessive concentration of a growth factor, yields decrease (Mengel 2001). For efficient cultivation of crops, the critical (minimum) concentration is set at 10% to 20% yield reduction, because this is the concentration with a relatively high yield, compared to the resources used for plant cultivation (Amberger 1996). Above this critical concentration, yield increase is relatively low because it is only achievable with a relatively high input of resources (Amberger 1996); crop increase is not linear (Schubert 2006). Yield response curves can differ, depending on the considered plant organ (vegetative or reproductive) or the characteristics of harvested products (Marschner 2002). This means that the yield response curve for high quality products, for instance, with a certain content of sugar or protein, may differ from the yield response curve of low quality products (Marschner 2002).

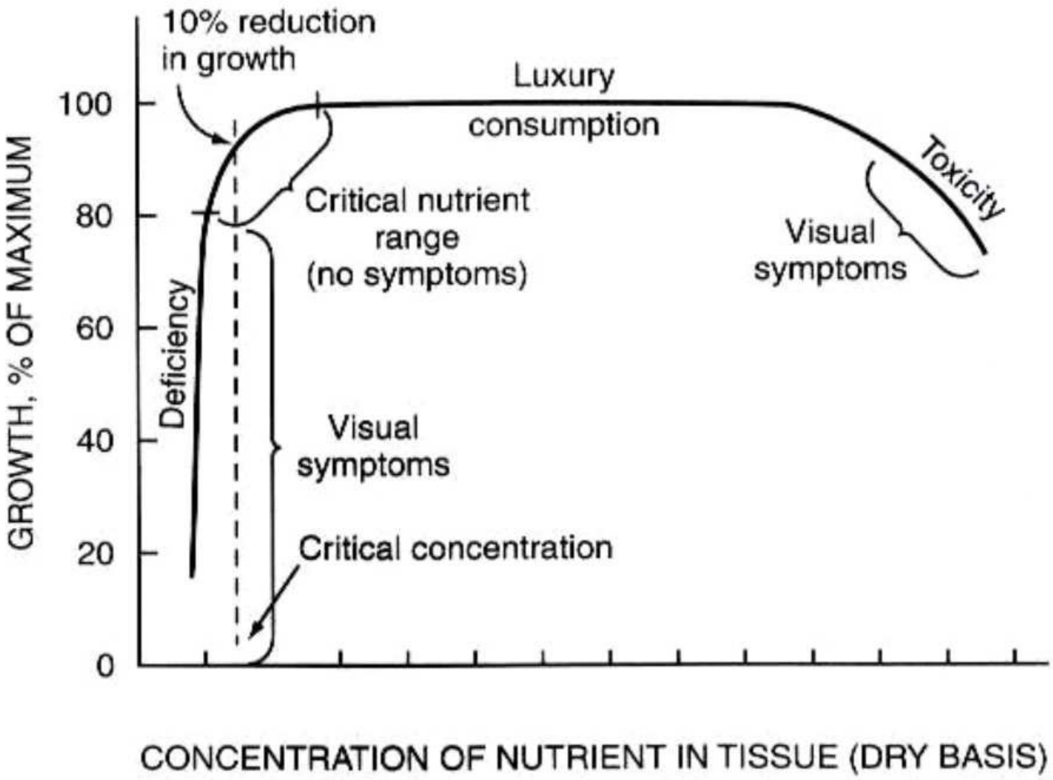

Figure 7 The relationship between plant nutrient concentration and plant growth (Havlin *et al.* 2004)

Deficiency of the main macronutrients can have the following effects on plants (Roy 2006):

- N: reduced growth rates, reduced crop yield, reduced protein content, short and thin appearance, poor tillering, small leaf area and yellow leaves, due to chlorosis
- P: retarded growth, retarded tillering, retarded root development, retarded ripening, decreased shoot/root ratio, bluish-green to reddish color of the leaves
- K: chlorosis, slow and stunted growth, weak stalks, bending of the stalks, higher vulnerability to pests and diseases, poor crop quality

Excess supply can have the following effects (Roy 2006):

- N: extended growing period and crop maturity, ammonia in the form of NH_3 in alkaline soil solution can be toxic to plants, nitrate may accumulate in leaves of, e.g., spinach or lettuce, and be reduced to nitrite, e.g., during storage without air and, thus, may cause methemoglobinemia

- P: watery edge of the leave tissue up to necrosis, die-off
- K: no direct negative effects on plants but indirect effects via reduced uptake of Ca, Mg and Na, which leads to an uneven supply with these micronutrients

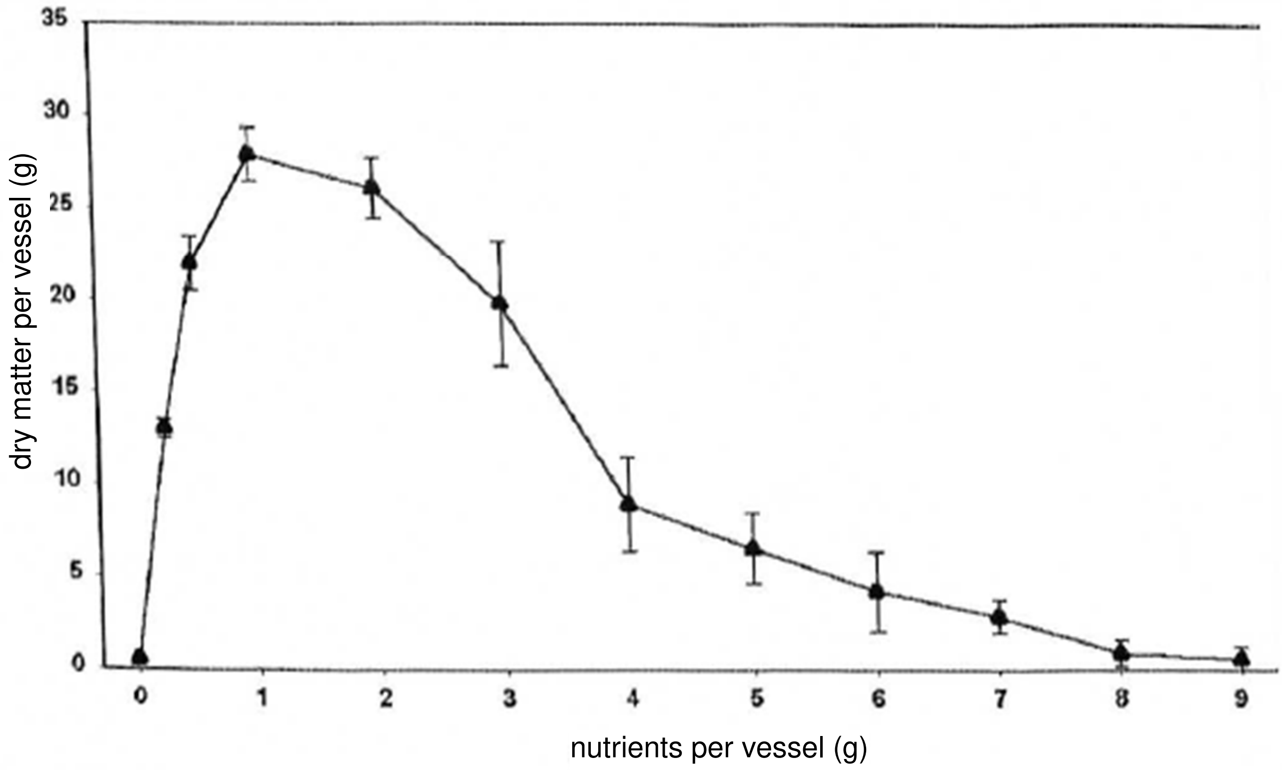

Figure 8 Influence of fertilization with nitrogen on growth of Lolium perenne (perennial rye-grass) in vessel experiments (Schubert (2006), modified, no information on the vessel size is given)

High nitrogen supply leads to reduced yield and increased vegetative growth (Stevens *et al.* 2006). The yield decline can be explained by the higher soluble salt content of the irrigation water with increasing N levels and accumulation of salts in the soil (Hochmuth 1987). For instance, potatoes and sugar beets develop smaller and fewer fruits (Stevens *et al.* 2006). Some plants, such as tree crops, are not affected and tomatoes even show higher yields (Stevens *et al.* 2006). High moisture content around melons and squashes due to excessive vegetative growth creates favorable conditions for rotting of fruits (Stevens *et al.* 2006).

Besides the quantity, the quality of crops can also be influenced in a positive or negative way by fertilization (Schubert 2006). For instance, too much nitrogen and phosphorus can cause a decrease in the firmness of several crops, such as apples (Sams 1999). Oranges produce grainy, pulpy fruits with less juice (Stevens *et al.* 2006). When using reclaimed water for irrigation, it is expected that increased levels of cations (above all Ca^{2+}) have a positive effect on firmness, texture and shelf life (Sheikh *et al.* 1998).

In conventional irrigation, toxicity is only an issue for Cu, Zn, Mn and B (Amberger 1996). Whereas the effects of N and P deficiency have been studied in most horticultural crops, this is not the case for the effects of oversupply with N and P (Benton Jones 1998; Stefanelli *et al.* 2010).

Negative effects due to excess P are said to be very unlikely in conventional irrigation (Roy 2006). But also when irrigating with reclaimed water, it is expected that there will be no negative effects due to excess P, since P is usually immobilized in the soil and thus no longer available for plants (Stevens *et al.* 2006).

There is little information available on the risk of overfertilization due to high nitrogen content in the reclaimed water. Ayers and Westcot (1985) recommend reducing nitrogen supply to crops later in the growing season by blending or utilization of other water sources and to adjust crop choice and crop rotation to the amount of nitrogen in the reclaimed water. Neubert (2003) concludes that high nitrogen loads and concentrations are most likely not a key problem but the occurrence of negative effects from excess nitrogen could be high, especially when dealing with more concentrated wastewater in arid countries.

Irrigation with reclaimed water requires that nutrient and water demands are matched, which increases the complexity of nutrient management (Stevens *et al.* 2006). However, it is already challenging to supply the fertilizer needed for achieving maximal yield at minimal cost under conventional nutrient management (Stevens *et al.* 2006). Altogether, more information on how to complement the nutrients contained in reclaimed water with additional nutrients from fertilizers is needed (Janssen *et al.* 2005).

2.7 Vacuum sewers

The aim of sewer systems is the establishment of hygienic conditions in settlements by quick transport of wastewater out of urban areas (Gujer 2007a; Roccaro *et al.* 2014). Surface water bodies or artificial ponds are often used for discharge (Tchobanoglous *et al.* 2004). Construction of wastewater treatment plants is a second step to limit pollution of these water bodies or to achieve an adequate water quality for reuse purposes (Gujer 2007a).

Conventional sewer systems use gravity for transport of liquids. Therefore, slope is necessary to assure gravitational flow in the pipes (Lens *et al.* 2001). Vacuum sewer systems may be an alternative to gravity sewer systems (Bowne *et al.* 1991; Tilley *et al.* 2008). They were presented for the first time around 1900 and introduced in several towns in Europe (Henze 2008), Mexico, Israel and the USA in the following decades (Read 2004).

As an example of one vacuum sewer system, a schematic overview of the main components of the ROEVAC vacuum sewer system is presented in Figure 9. The wastewater first flows in a gravity pipe from its point of origin to an outdoor collection chamber (Spaeth 2007). An interface valve is the entry point to the vacuum sewers (DWA 2008a). The interface valve opens once a certain water level inside the collection chamber is reached (DWA 2008a). As an alternative, the connection to the vacuum system can be facilitated by vacuum toilets or other sanitary items (DWA 2008a). A mixture of wastewater and air is transported in the vacuum sewers to a vacuum station, where the water is collected in a vacuum tank (Spaeth 2007). From the vacuum tank, the wastewater is fed to wastewater treatment by pumps or pneumatic conveyors (DWA 2008a). Usually, a pressure of 0.6 to 0.7 bar is required for operation, which is generated by vacuum pumps (DWA 2008a).

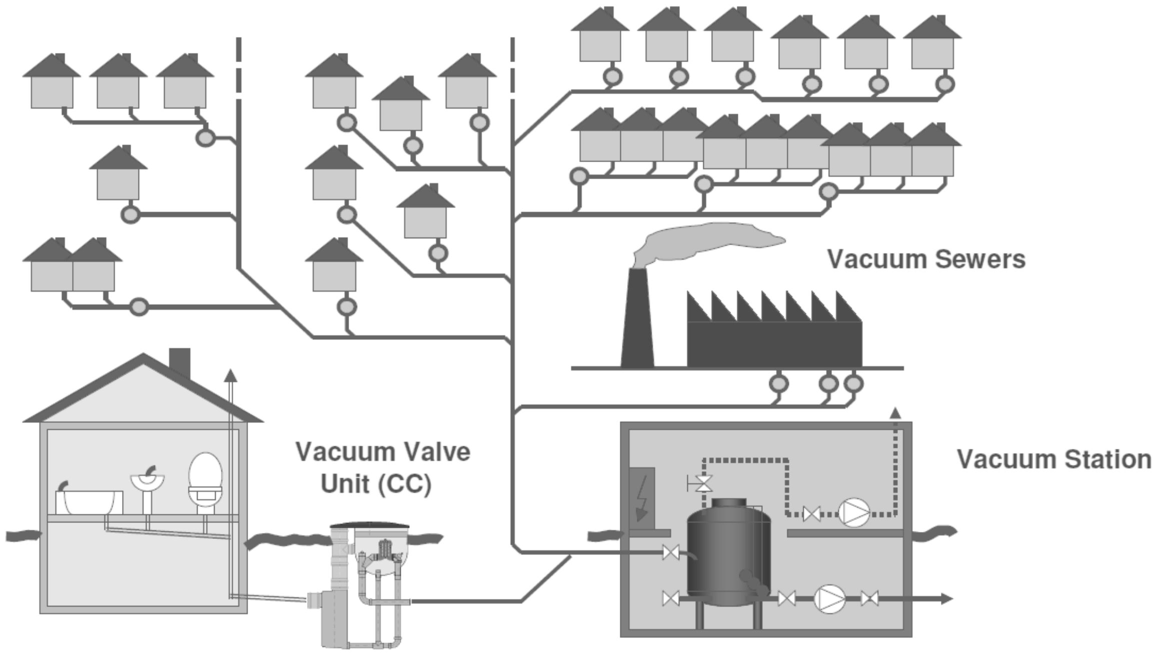

Figure 9 Schematic overview of the ROEVAC vacuum sewer system (main components, Spaeth (2007), modified), cc = collection chamber

There is a set of conditions under which the installation of a vacuum sewer system should be considered (Bowne *et al.* 1991; DWA 2008a; GTZ 2005):

- insufficiently sloped terrain or flat terrain
- unfavorable ground conditions or unstable soils
- new urban development in rural areas
- water-scarce regions
- water protection areas
- if nutrient and energy recycling is intended
- rural areas
- connection of low-level developments and buildings
- need for crossing of obstacles (e.g., water courses, ditches, utility lines)
- high groundwater table
- low population density
- seasonal or intermittent wastewater production (e.g., camp sites, weekend home developments)
- where disturbance (e.g., to traffic, structures, soil) is to be kept minimal

A comparison of gravity and vacuum sewers is given in Table 5. Vacuum sewers usually have a smaller diameter than gravity sewers and are made of PE or PVC (Behnke 2004). In flat areas, gravity sewer systems will reach substantial depths in order to ensure a minimum velocity of the wastewater, to avoid accumulation in the sewers (Little 2004). At a depth of 8 to 10 meters, a pumping station is required to lift the wastewater. Many pumping stations may be required, which increases costs for construction and operation of gravitational sewer systems (Lens *et al.* 2001). Hence, construction of vacuum sewers may be much easier, due to shallower trenches, because simpler or even no machinery is required for excavation and no manholes or lifting stations need to be installed (Behnke 2004).

Table 5 Comparison of gravity and vacuum sewer systems (Behnke (2004), modified)

	gravity system	**vacuum system**
pipe diameters	large (> DN 200)	small (DN 80 – DN 200)
pipe material	PE or stoneware or concrete	PE or PVC
trenches	deep (up to 8 m and more) and wide	shallow (1 – 1.4 m) and narrow
excavation	complicated and long term	simple and fast
machinery required	heavy machinery	simple or even no machinery
lifting stations	required as pipe is too low	not required
leakage	yes	no
traffic	high impact on local traffic	low impact on local traffic
pipeline for fresh water and wastewater in the same trench	not allowed	possible and allowed
system	open	closed
manholes	required	no manholes, only inspection pipes
dry sewers	possible	not possible
fouling of wastewater	possible	not possible
flushing of pipelines	sometimes required	not required

In water-scarce regions, it is often reported that flushing velocities in gravity sewer systems are too low to allow transport of solids (Little 2004). Salifu (1997), for instance, provides a résumé of the condition of gravity sewerage systems in three Ghanaian cities (Tema, Accra and Kumasi). The majority of the sewers were affected by sand accumulations. The velocity in the sewers was too low to ensure transport of solids. To prevent accumulation of solids, either the canal slope has to be increased or regular flushing of the sewer system is necessary. Both measurements result in higher operation and maintenance costs of a gravity sewer system (Bowne *et al.* 1991).

In vacuum sewer systems, flushing of sewers is not required and fouling usually does not occur because a mixture of air and wastewater is transported in the sewers (Behnke 2004). Further advantages are that vacuum sewers and tap water pipes can be installed in the same trench and that the impact on local traffic can be kept relatively low (Behnke 2004).

Under certain conditions, construction and operation of vacuum sewers can be much less expensive than construction and operation of gravity sewers (DWA 2008a). However, vacuum sewers usually do not drain rainwater, which has to be kept in mind for urban water management (DWA 2008a).

2.8 Co-digestion

Anaerobic digestion "is the engineered methanogenic decomposition of organic matter, carried out in reactor vessels, called digesters, that may be mixed or unmixed and heated or unheated" (Wilkie 2008). The advantages of anaerobic stabilization of sewage sludge and anaerobic treatment of (high-strength) wastewater, compared to aerobic treatment methods, are the lower energy requirement, the possibility to recover methane, higher volumetric organic loads compared to aerobic processes and, therefore smaller reactor volumes, the lower biomass production, lower nutrient requirements, long periods without feeding are unproblematic and lower operational and capital costs (Bitton 2011; Tchobanoglous *et al.* 2004).

Disadvantages include the longer start-up time, the slower process, higher sensitivity to toxic substances, production of odors and corrosive gases, and that anaerobic treatment of wastewater requires further aerobic treatment and eventually further nutrient elimination (Bitton 2011; Tchobanoglous *et al.* 2004).

In principle, methanogenesis is possible in a temperature range from 0°C to > 100°C; however, common temperature ranges for technical applications are > 10°C to 20°C (psychrophilic), 20°C to 45°C (mesophilic) and 45°C to 65°C (thermophilic) (Gallert *et al.* 2015). These temperature ranges vary slightly in the literature.

Operation in the thermophilic temperature range is connected with lower process stability, but the advantage is the obtained hygienization of the remaining biosolids (Gallert *et al.* 2015; Tchobanoglous *et al.* 2004). Hays (1977) concludes, from the reviewed literature, that helminth eggs are destroyed when kept at least 30 minutes at temperatures of 60°C.

Under standard conditions, 0.35 L methane can be produced per 1 g of converted COD (Tchobanoglous *et al.* 2004; Wilkie 2008). The biogas produced during anaerobic digestion contains 60% to 70% CH_4, 30% to 40% CO_2, 0.05% to 1% N_2, 0.02% O_2 and 50 ppm to 3,000 ppm H_2S (Andreoli *et al.* 2007; Tchobanoglous *et al.* 2004).

There are several options for utilizing the produced biogas. It can be directly used for: the production of process heat and steam, production of heat and electricity in an internal combustion engine (cogeneration engines), micro-turbines or fuel cell systems; it can also be upgraded and injected into existing natural gas pipelines or catalytically transformed into hydrogen, ethanol or methanol (Banerji *et al.* 2010; Wilkie 2008). If heat is produced, it is usually utilized for hot water production for use at the plant (Wilkie 2008). The combination of electricity generation and hot water production can provide an energy conversion efficiency of 65% to 85% (Wilkie 2008).

Co-digestion is the combined digestion of two or more substrates (Grosser *et al.* 2013; Mata-Alvarez *et al.* 2014). Usually, smaller amounts of additional substrates are added to a larger amount of a basic substrate (Shah 2014). Grosser *et al.* (2013) summarize the main advantages of co-digestion compared to mono-digestion; these are a higher process efficiency (e.g., increased biogas production, higher degradation, higher process stability), a better overall nutrient balance, increased organic loads, higher flexibility regarding the regulation of the pH, moisture content, buffer capacity and C:N ratio. Disadvantages are the transport costs of the co-substrate, the possible need for pretreatment of the co-substrates, and decreasing digester effluent quality (Grosser *et al.* 2013).

Possible main substrates are usually animal manure, sewage sludge or biowaste (Mata-Alvarez *et al.* 2014). Between 2010 and 2013, the scientific literature primarily reported the use of manure as the main substrate, followed by sewage sludge and the organic fraction of municipal solid waste (Mata-Alvarez *et al.* 2014). Co-substrates reported for sewage sludge are mostly industrial or municipal wastes, e.g., fats, oils, greases, fruit and vegetable waste, slaughterhouse waste, glycerol and algae (Mata-Alvarez *et al.* 2014). Historically, the organic fraction of municipal solid waste was the most frequently used co-substrate for digestion with sewage sludge (Mata-Alvarez *et al.* 2014).

According to Esposito *et al.* (2012), co-digestion processes have only been studied since the beginning of the 1990s. Unanswered research questions focus on the effect of temperature on the process performance, pretreatments, feeding systems, moisture content, and optimal stirring and mixture (Esposito *et al.* 2012).

The biochemical and microbiological fundamentals, design considerations, and details about the various anaerobic treatment processes are described extensively elsewhere (Andreoli *et al.* 2007; Bitton 2011; Chernicharo 2007; Forster 2003; Rosenwinkel *et al.* 2015; Tchobanoglous *et al.* 2004) and are therefore not presented here.

2.9 Introduction to North Namibia

The preceding Chapters 2.1 to 2.8 gave an introduction to the main topics covered in the later chapters of this dissertation. This section provides background information on some characteristics of the project region. It starts with an overview on available water resources, the significance of urbanization, and the existing wastewater infrastructure in North Namibia. The occurrence of floods is an important characteristic of the project region. Thus, this topic is also covered here. The last section presents selected details on the electricity sector in Namibia, as background information for the results on some energetic aspects of the project.

2.9.1 Available water resources

Namibia is the driest country in Sub-Saharan Africa (The World Bank 2009). Total water demand exceeds supply (The World Bank 2009). The following paragraphs provide a brief summary of groundwater and surface water resources in North Namibia, based on the information provided by Mendelsohn *et al.* (2000).

In most areas of the central north of Namibia, groundwater resources are not usable. The shallowest aquifers are entirely fed by seasonal rainfall and usually dry up during dry season. This discontinuous perched aquifer provides water only locally and lies on a less permeable layer. Underneath, the main shallow (saline) aquifer can be found at depths between 20 m and 40 m. In some areas, fresh water percolates to this aquifer and builds up a layer on top of the salty groundwater, due to its lower density.

In Outapi, the city where this study was carried out, it lies between 10 m to 20 m below the ground. Here, the water of the main shallow aquifer is very salty. TDS exceed values for suitability for livestock (> 5,000 mg/L). Additionally, the groundwater found in this layer contains fluoride in concentrations > 3 mg/L, a concentration that, when consumed, is harmful to the development of the juvenile skeleton. The sulfate content is > 1,200 mg/L and has a laxative effect on humans. Thus, groundwater does not play a role in the water supply of Outapi and its surroundings.

Annual rainfall in the central north of Namibia ranges between 350 mm in the west and 550 mm in the east. In the Outapi region, average precipitation is between 350 mm and 400 mm per year. More than two-thirds of the rain falls between January and March, and only 4% from May to October. Precipitation may vary considerably in time (from year to year) and in space with a variation coefficient between 40% and 60%. This high variation, its limitation to a certain time of the year, and the lack of autochthonous water sources pose risks for water-dependent activities such as farming.

Following sufficient rainfall, seasonal water flows occur in a system of oshanas (shallow ephemeral rivers) that transport water from the Angolan headwaters to the north of Namibia. Most of these surface water bodies dry up after the end of the rainy season.

For its industrial, domestic and agricultural water supply, the region depends on the Kunene River, a border river between Angola and Namibia. It is a perennial river and forms a water source that is available year-round. Via a network of canals and pipelines whose length amounts to 2,600 km, water is transported to the densely populated area in the north of Namibia. In Outapi, this water is purified in a water treatment plant (run by NamWater) and delivered to households and water points via pipes. This Outapi network was created around 2000.

2.9.2 Soils

The soils in the eastern and western area of the central north of Namibia consist of deep Kalahari sands that were deposited and shaped by strong winds (Mendelsohn *et al.* 2000). The soils in the central area are dominated by clayey sodic sands in the oshanas and sodic sands on the surrounding higher ground (Mendelsohn *et al.* 2000). The soils in the area around Outapi are categorized as sands and loams (Mendelsohn *et al.* 2000). The soils in the Outapi region are highly suitable for crop cultivation when water, nutrients and organic matter are provided (Mendelsohn *et al.* 2000).

2.9.3 Population growth, urbanization and wastewater infrastructure

Namibia's population has grown from 1.8 million in 2001 to 2.1 million in 2011; half of the population lives in the central north (NSA 2011). In the same period, the percentage of people living in urban areas has increased from 33% to 43% (NSA 2011). The urbanization rate is expected to exceed 70% in 2030 (GRN 2004).

As an example, the development of the informal areas in Outapi is shown in Figure 10. The number of dwellings grew considerable between 2008 and 2011. Keeping pace with this rapid development of urban dwellings is a challenge for infrastructure planning. Increasing urbanization is accompanied by a decreasing number of people with access to sanitation in the urban areas of Namibia. The percentage of people with access to sanitation decreased from 89% in 1991 to 82% in 2001 (census data 1991 and 2001, NPA (2004)), to 59% in 2003 and 57% in 2010 (NPA 2013).

The Namibian government defines requirements for sanitation systems in the National Sanitation Strategy (MAWF 2009). In urban areas, waterless systems and central systems with

Figure 10 Development of the informal areas in Outapi between 2008 (left) and 2011 (right) (Google Earth (Version 7.0.2) 2011, 2008)

Figure 11 Available toilets in the project area in Outapi

Figure 12 Sheds for body cleaning in Outapi: exterior view (left) and interior view (right)

Figure 13 Public toilets at the Outapi open market

Figure 14 Oxidation ponds for collection of sewage in Outapi

waterborne sewage are possible. It also states "when water is available at household level, but high quantities of water for flushing toilets are unaffordable, a centralized vacuum system could be considered". Preference is given to individual solutions, compared to shared sanitation, but it is acknowledged that "for certain informal settlements, the provision of waterless and shared toilets is an appropriate solution" (MAWF 2009).

In the rural areas of North Namibia, simplest sanitary facilities are used or open defecation is practiced. In urban areas, this also applies to the informal settlements. Here, some dry toilets are available (Figure 11). They often collapse during the rainy season. Maintenance or regular cleaning is lacking. For personal hygiene, at best, self-made sheds are available (Figure 12). Collection or removal of feces and used water does not occur. In districts that are better developed, flush toilets are used (Figure 13). The wastewater is transported by a conventional waterborne sewage system to non-aerated oxidation ponds (shallow pools for collecting wastewater, Figure 14). Reclamation or reuse of this water does not occur.

2.9.4 Floods

In 2008 and 2009, heavy flooding occurred in the Namibian north, whereas, in 2013, Namibia experienced a severe drought (Haeseler 2013; Kazondovi 2013; NMS 2016; Thulkanam 2010). Floods are caused by heavy rainfall and the inflow of water from the south of Angola, via the shallow ephemeral rivers (oshanas).

The currently practiced water and wastewater management in Outapi is not adapted to flood events. Existing gravity sewers, pump stations, oxidation ponds, pit latrines, open defecation areas, and the tap water treatment plant are located in flood-prone areas. Figure 15 shows the situation during the dry season and during the rainy season in March 2008. Clearly visible are the oxidation ponds and the location of the treatment plant of the tap water supplier NamWater.

The oxidation ponds, as well as the area of the waterworks, are flooded. It must be assumed

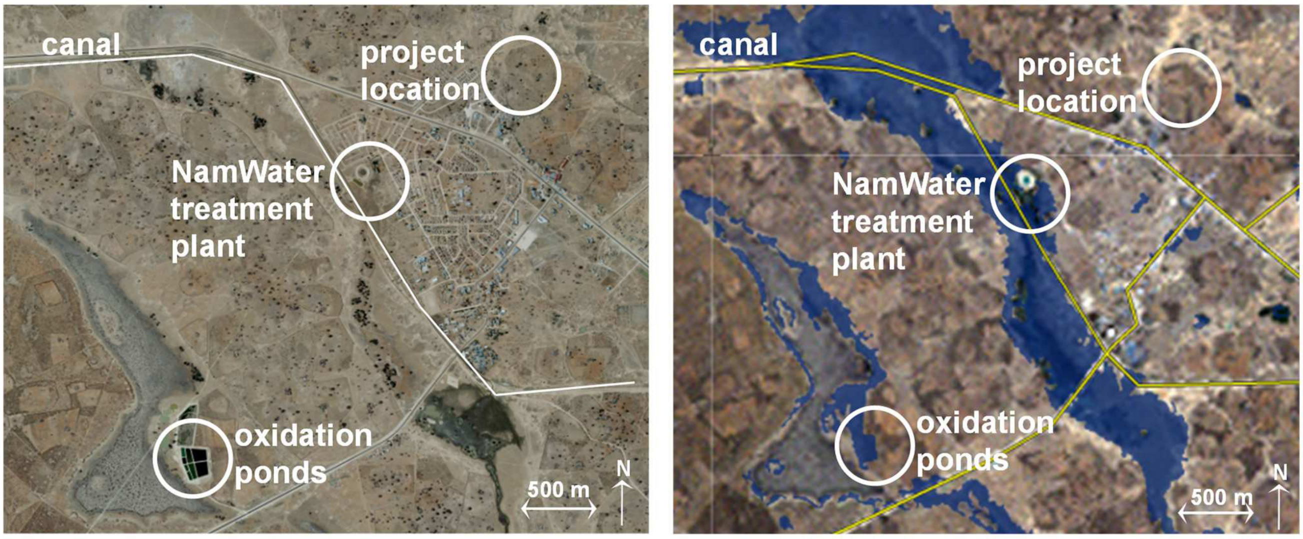

Figure 15 Situation in Outapi during dry season (left) and location of flooded areas on 16 March 2008 (DLR 2008; Google Earth (Version 7.0.2) 2008)

that rainwater, as well as run-off from the oshanas and the canal, spread feces and wastewater to surrounding areas. Pathogens contaminate stagnant water bodies and the canal water that is used as water source for the tap water supply of Outapi.

The capacity of the existing oxidation ponds was initially about 120,000 m³. Assuming a wastewater quantity of 120 L/(person×d), this pond capacity would be enough to store the annual wastewater of 2,740 people for one year (= 120,000 m³ × 1,000 L/m³ ÷ 365 d/a ÷ 120 L/(person×d)). The entire contents of the pond might be discharged into the environment during flood events and cause the spread of waterborne diseases (Filali-Meknassi *et al.* 2014; IFRC 2011, 2009).

2.9.5 The Namibian electricity sector

In Namibia, the Ministry of Mines and Energy (MME) is the supreme authority for the energy market (Vita *et al.* 2006). The MME was founded after Namibia gained independence in 1990 and is responsible for the use and regulation of energy resources (Vita *et al.* 2006). NamPower is a state-owned utility and is under supervision of the MME (Oertzen 2012a). As the sole energy supplier, NamPower has a monopoly in terms of power generation, transmission and import (Oertzen 2012a).

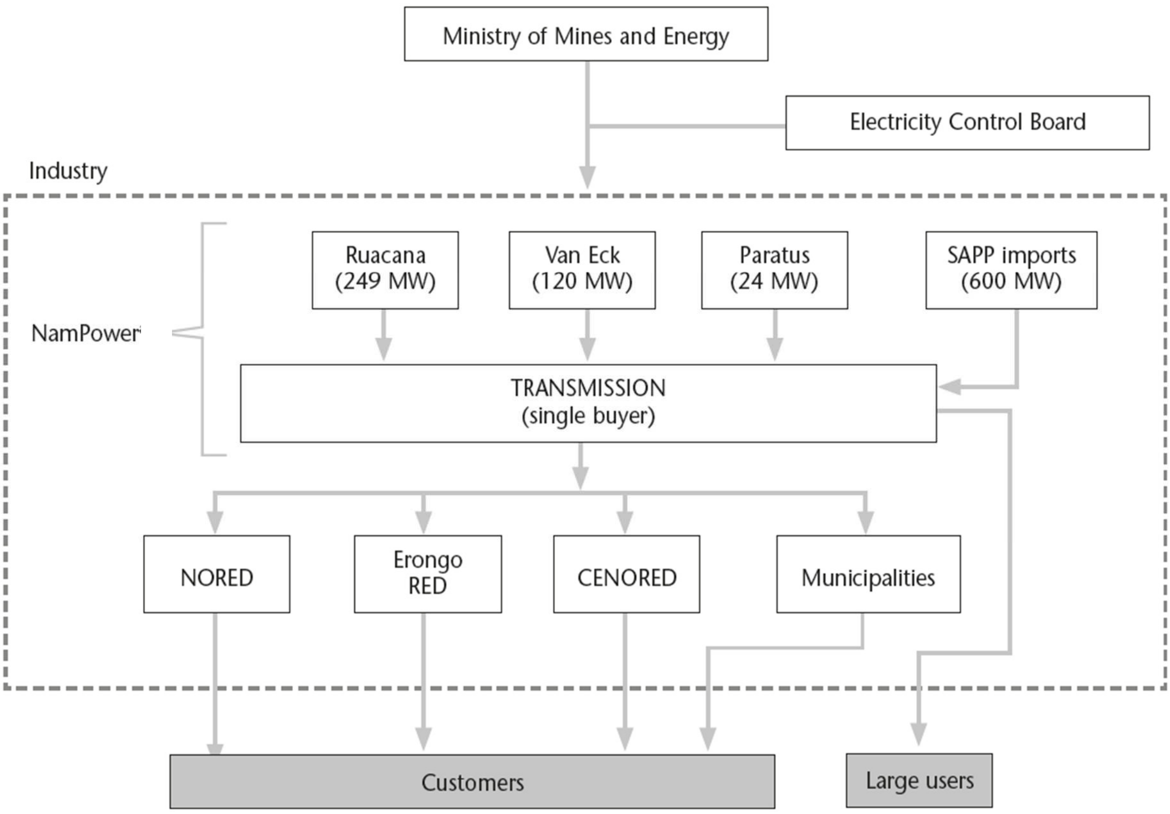

Figure 16 Overview of the Namibian electricity sector (Kapika and Eberhard (2013), modified), SAPP = South African Power Pool

Independent power producers have, so far, not yet been established, despite the efforts of the government (Vita *et al.* 2006). Based on the Energy White Paper that was adopted in 1998, an independent authority, the Electricity Control Board (ECB) has been created for the regulation of the energy market in 2000 (Asemota 2012). It acts as a statutory regulator for the generation, transmission, distribution, supply, import and export of electricity and also grants licenses (Vita *et al.* 2006).

Since the establishment of the ECB, the MME is only responsible for the development of energy policies, whilst the ECB has taken over the regulation of the energy sector (Kapika and Eberhard 2013). As can be seen in Figure 16, the Namibian electricity market is dominated by NamPower, acting as the owner and operator of all three power plants (Ruacana, Van Eck, Paratus), the transmission nets in the country and several separate power distribution facilities in rural areas in central and southern Namibia (Kapika and Eberhard 2013). A significant proportion of the electricity is imported from members of the Southern African Power Pool (SAPP) (Figure 17, Kapika and Eberhard (2013)).

The transmission system in Namibia is connected to various distribution stations, which transform the transmitted power from high voltage to medium voltage (Oertzen 2012a). The distribution network is operated by three regional electricity distributors (REDs), several local and regional town councils, and NamPower (Oertzen 2012a). They are responsible for the distribution of electricity to commercial, industrial, institutional and retail consumers in their respective area of activity (Oertzen 2012a).

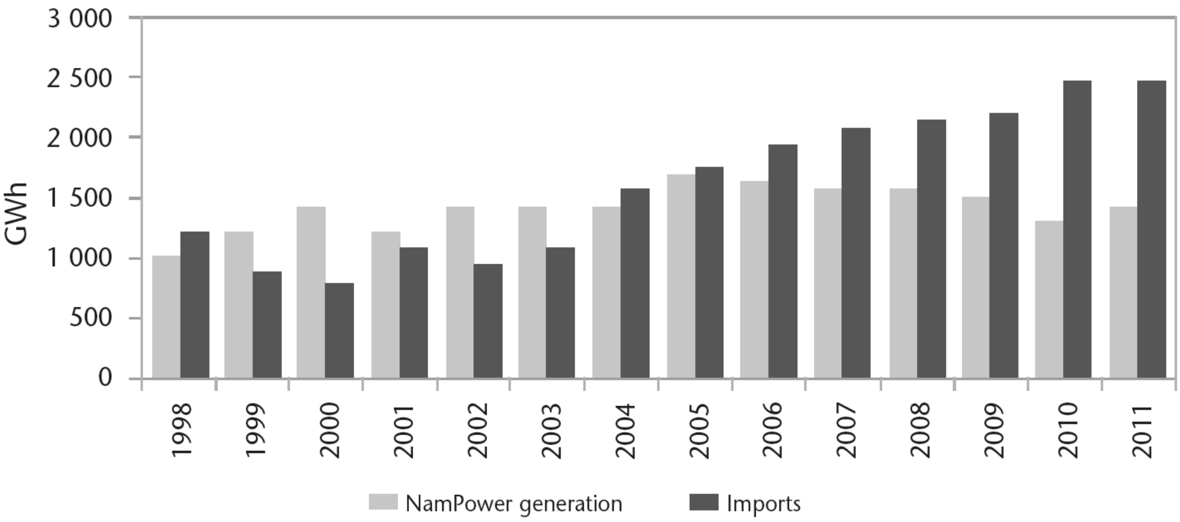

Figure 17 NamPower's generation capacity, compared with electricity imports, 1998-2011 (Kapika and Eberhard 2013)

The REDs are separate companies that buy the electricity from NamPower and resell it, according to their own pricing structure, to the final customer (Oertzen 2012a). Supervision of the tariffs raised by the REDs is provided by the ECB, which approved the tariffs (Kapika and Eberhard 2013).

Namibia has three REDs with clearly defined geographical areas of responsibility (Oertzen 2012a). The Erongo Regional Electricity Distributor (ERONGORED) is responsible for the central west. The Northern Electricity Distributor (NORED) covers the entire North of Namibia. The Central Northern Electricity Distributor (CENORED) services areas south of the area supplied by NORED (Kapika and Eberhard 2013).

3 Materials and methods

3.1 Study area and project description

This study was carried out the city of Outapi in North Namibia (Figure 18). In 2011, Outapi had a total population of 6,437 persons (NSA 2011). The population density is approximately 21.5 persons/ha (based on the approximate town area of 3 km², estimated using Google Earth (Version 7.1.2.2041) (2013)).

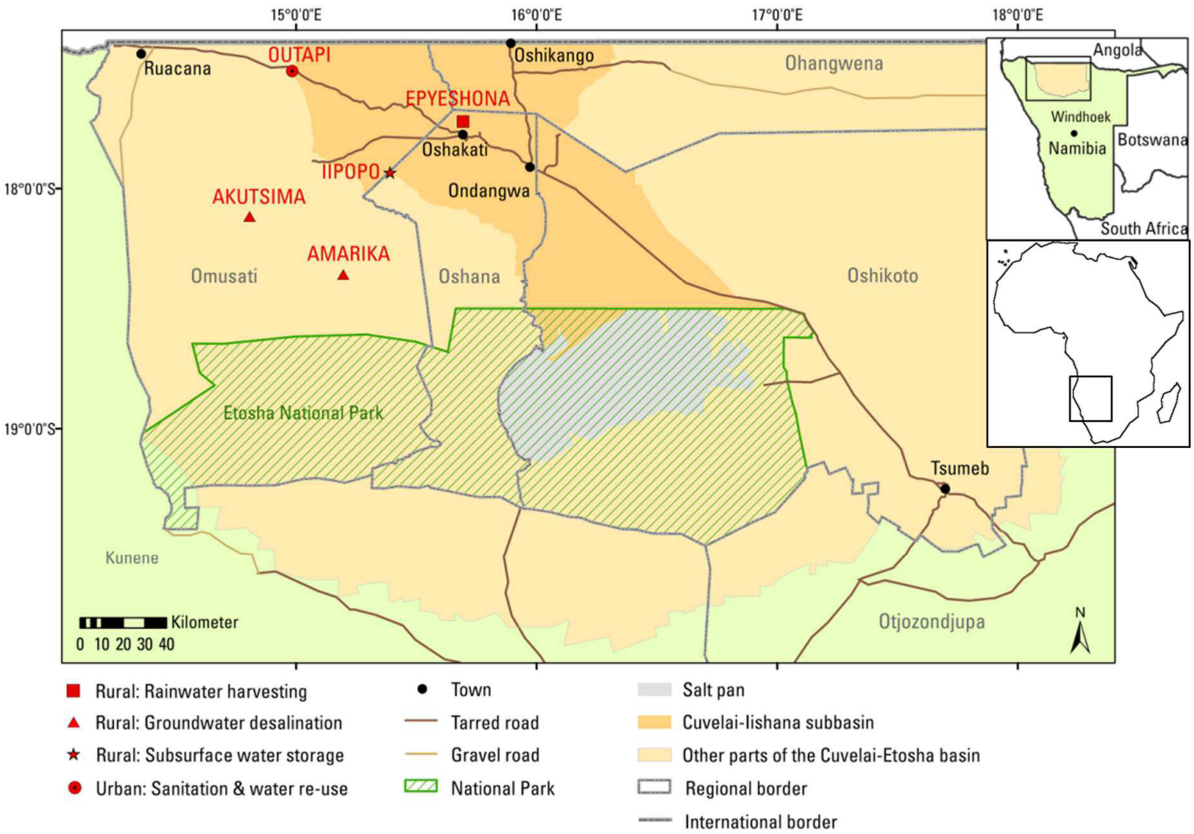

Figure 18 Overview on the study area of the CuveWaters project, the locations of the sub-projects and types of implemented facilities (ISOE (2013), Price and Hegnauer (2016), modified)

Together with local stakeholders, Outapi has been chosen as the location for a project on sanitation and water reuse (Deffner and Kluge 2013; Deffner and Mazambani 2010). This initiative is part of the interdisciplinary project “CuveWaters”. Its overall objective is the development and implementation of an integrated water resources management for the Cuvelai-Etosha Basin in the north of Namibia (Kluge *et al.* 2008). The CuveWaters project is a joint research project funded by the German Federal Ministry of Education and Research (BMBF). Facilities for rainwater harvesting, groundwater desalination, subsurface water storage, and sanitation and water reuse were implemented at five locations in several sub-projects (Figure 18).

Project partners of the sub-project on sanitation and water reuse are the Institute for Social-Ecological Research (ISOE, Frankfurt, Germany), the Technische Universität Darmstadt (TUDa, Darmstadt, Germany), Bilfinger Water Technologies (BWT, Hanau, Germany), the Outapi Town Council (OTC, Outapi, Namibia), the Desert Research Foundation of Namibia

(DRFN, Windhoek, Namibia) and the Ministry of Agriculture, Water and Forestry (MAWF, Windhoek, Namibia).

Planning of the overall concept for sanitation and water reuse started in 2009 as a close cooperation between the OTC, DRFN, TUDa, BWT and ISOE. To adjust the overall concept, and especially, the layout of the sanitation facilities to the needs of future users, several commu-

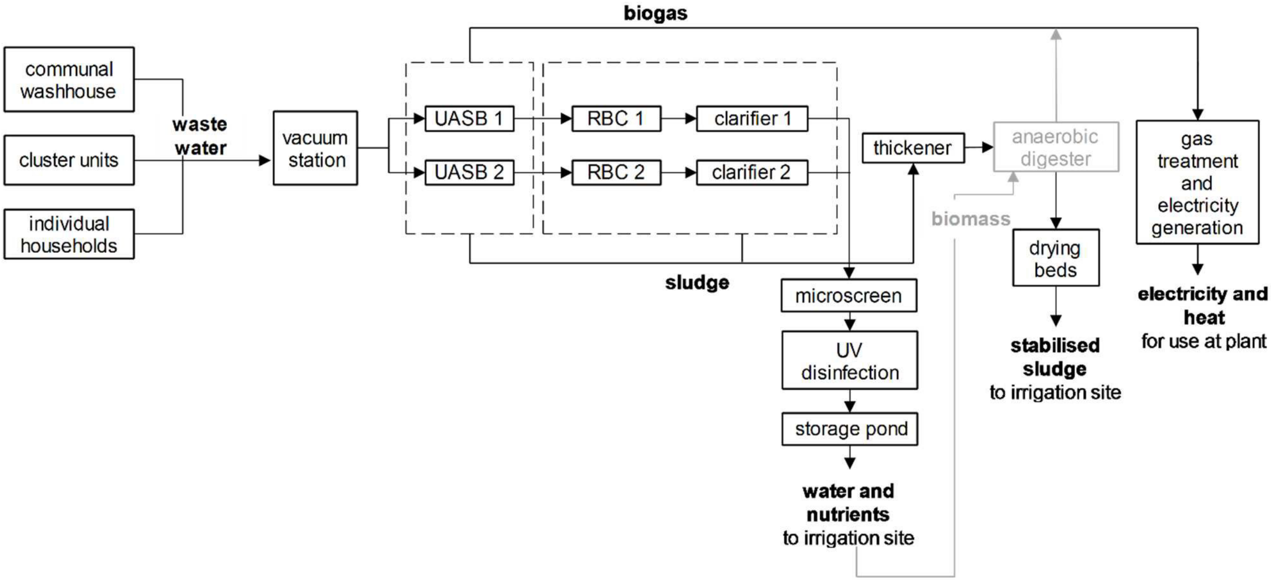

Figure 19 Schematic drawing of the concept for sanitation and water reuse in Outapi; elements in grey represent components that were not implemented as intended, UASB = upflow anaerobic sludge blanket, RBC = rotating biological contactor

Figure 20 Location of the components of the sanitation system: cluster units (1), individually connected households (2), communal washhouse (3), water reuse plant with storage pond (4) and irrigation sites (5 and 6) (Google Earth (Version 7.1.2.2041) 2013)

nity workshops were organized and run by DRFN and ISOE (Deffner and Mazambani 2010). A Namibian civil consultant was responsible for the detailed planning. Construction was carried out from 2011 to 2013 by a Namibian construction company. More details on the planning process, choice and dimensioning of wastewater treatment are provided in the results and discussion section (Chapter 4.1, page 53*ff.*). This section focuses on the description of what was finally implemented by the project partners.

Figure 21 Layout of the communal washhouse (top, drawing provided by Lund Consulting Engineers, Windhoek), after start of construction in December 2011 (bottom, left) and after completion in May 2013 (bottom, right)

The infrastructure includes various types of sanitation facilities, a vacuum sewer system (RoeVac, BWT, see Section 2.7, page 28), a wastewater treatment plant with sedimentation and anaerobic pretreatment (ROEDIGER UASB reactors – upflow anaerobic sludge blanket, BWT), aerobic treatment and secondary clarification (RBC – rotating biological contactors and lamella clarifiers, System S&P, Dr. Scholz & Partner), microscreening (a drum-type microscreen, 15 µm mesh width, PASSAVANT Micro Giant (MTSM) 1000 × 1000, PAN4-4711, BWT) and UV disinfection (low pressure UV lamps, LBX 50, WEDECO). The treated water is stored in a pond and applied to the agricultural fields via surface drip lines (Figure 19). The

Figure 22 Layout of one cluster unit (top, left, drawing provided by Lund Consulting Engineers, Windhoek), construction phase in December 2011 (top, right and bottom, left) and after completion in November 2013 (bottom, right)

reclaimed water is used for the production of vegetables for human consumption. An evaporation pond collects the drainage water from the fields, where it is dried by solar radiation. The sewage sludge is dried on sludge beds. After stabilization, the biosolids and nutrients are used on the agricultural fields.

Outapi is a fast-growing urban area with a very heterogeneous structure reflected in differently developed areas. It was clear from the beginning that one single type of sanitation facility could not serve the needs of all residents and suit all developmental stages. Thus, three different types of sanitation facilities were implemented in three areas at varying stages of development (Figure 20). Up to 1,500 residents can benefit from this infrastructure.

A larger, shared sanitation facility offers flush toilets, showers, hand wash basins and sinks for laundry washing on a pay-per-use basis to the community as well as to people from a nearby market place (Figure 21). It includes separate sections for male and female users and is located in a very recent, informal area of corrugated iron huts completely lacking any kind of water infrastructure at the time the project started. The communal washhouse was intended to serve up to 250 inhabitants. Operation started in April 2013 and security and maintenance staff was provided by the OTC. According to the definitions given in Norman (2011) (Figure 2), this facility can be characterized as a public facility because it is open to everybody on a pay-per-use basis. However, since it is targeted towards servicing the community, it is referred to as "communal washhouse" in this work.

Thirty smaller sanitation facilities ("cluster units") are shared by three to five families each since November 2013. The area has a pre-formal layout. It is a neighborhood with provisional streets and buildings. The cluster units are equipped with an indoor shower, toilet and hand wash basin and an outdoor laundry sink (Figure 22). They are managed by the allocated households. Showers and laundry sinks are equipped with prepaid water meters. Free access is given to toilets and the small hand wash basins. The costs for the water not billed for (toilets and hand washing) are cross-subsidized by the paid uses (showering and laundry washing).

Up to 66 households in a self-built neighborhood (a 'pre-formalized' area with brick houses) can be individually connected to water pipes and sewers (Figure 23). During the project duration, 42 households were included in the sanitation system.

Figure 23 Buildings with individual connections to tap water supply and vacuum sewers

3.2 Determination of water quantities

Water quantities used in the communal washhouse were measured with domestic multi-jet water meters (MNK, Zenner and Model M, Arad). The cumulative water use was recorded from the water meters by the project's laboratory assistant, usually at 8:00 am during weekdays (sometimes during weekends). Records were taken from six water meters that measured the total water use in the section for female users (including toilets, showers, hand wash basins, and half of the laundry sinks), the total water use in the section for male users (including toilets, showers, hand wash basins, and half of the laundry sinks), male toilets, female toilets, and from two separate water meters, each of which measured water use for half of the sinks.

The determination of the water quantities consumed by the cluster units and the individual households was also made with domestic multi-jet water meters (MNK, Zenner and Model M, Arad). Each cluster unit has one water meter for determining the water use of the toilet and hand wash basin and one water meter for the water use of the shower and the laundry sink. In the individual households, the total water use was monitored via water meters installed by the OTC. The values were recorded at irregular intervals (at least once per month) by the project's interns or the operators of the wastewater treatment plant. Daily means were calculated using the cumulative water use divided by the number of days per time interval.

At the communal washhouse, water meter readings were recorded more frequently than in the cluster units or individual households. The cumulative values of two consecutive days could be used to calculate daily values, which were then used for the calculation of standard deviations during each time period. The specific water uses were calculated using the daily water use divided by the daily number of uses.

Because data was collected more frequently at the communal washhouse than at the other sanitation facilities, more detailed data analyses could be carried out. The non-parametric Wilcoxon matched-pairs test was applied to identify significant differences between water use, the number of uses and specific water uses. The considered time intervals were the entire monitoring period (May 2013 to September 2015), the time period with a lower user fee (tariff 1: May 2013 to August 2014), and the time period with a higher user fee (tariff 2: September 2014 to September 2015). IBM SPSS Statistics, Version 20, was used to perform this test.

Water quantities in the influent and in the effluent of the wastewater treatment plant were measured online with electromagnetic flow meters (Promag 10W40, Endress+Hauser; see also Section 3.6).

Precipitation and evaporation were continuously recorded by an electronic weather station on the irrigation site (iMETOS, Pessl Instruments).

3.3 Sampling and analytical methods

Water quality data was obtained in an analytical laboratory that is located in the operator building of the wastewater treatment plant (Figure 24, Figure 25). Sampling and analyses were carried out by a laboratory technician who was trained by TUDa during the first weeks of operation and performed all required tasks during the project period.

The data presented here were collected between May 2013 and July 2015. Concentrations of wastewater constituents were measured as double determinations in volume-proportional, 10-hour mixed samples (8:00 am to 6:00 pm) or 12-hour mixed samples (6:00 am to 6:00 pm), depending on the opening hours of the communal washhouse or on the time period with the highest total wastewater quantities.

Sampling was usually performed once a week. Weekdays were shifted to also include weekends. Hach Lange cuvette tests were used for the analyses (Table 6; for working procedures, see www.hach.com). Concentrations of TCOD, TN and TP were determined in homogenized samples (homogenizer: T 25 digital Ultra-Turrax, IKA). Dissolved COD (DCOD), nitrate nitrogen (NO_3^--N), ammonium nitrogen (NH_4^+-N), nitrite nitrogen (NO_2^--N) and orthophosphate phosphorus (PO_4^{3-}-P) were determined in 0.45 µm filtered samples (Whatman membrane filters, ME 25). These parameters were also determined in three tap water grab samples. The tap water samples were analyzed without prior treatment (double determination).

Figure 24 Laboratory at the wastewater treatment plant in Outapi

Figure 25 Operational building (left), workplace of the operating personnel with computer for recording the online data (center) and sampling point in the effluent of the microscreen (right)

The chemical oxygen demand of particulate matter (PCOD) was calculated by subtracting the DCOD from the TCOD. Concentrations of organic N were obtained by subtracting NH_4^+-N concentrations and NO_3^--N concentrations from the TN concentrations. Concentrations of polyphosphate P and organic P were obtained by subtracting PO_4^{3-}-P concentrations from TP concentrations.

PH, EC, temperature and turbidity were measured in grab samples at the sampling points at the wastewater treatment plant (weekdays), the storage pond (mostly once per week), and in tap water (at irregular intervals) (pH and EC meter: Multi 1970i, pH electrode: Sentix 41-3, EC electrode: TetraCon 325, WTW; turbidity meter: 2100 Q IS Portable Turbidimeter, Hach Lange). The location of the sampling points is shown in Figure 26.

The determination of total dissolved solids (TDS) was not possible in all water samples, due to rapid clogging of the glass microfiber filters (Whatman 934-AH) and the relatively low TDS content in the samples; this required the evaporation of considerable amounts of water to obtain a sufficient quantity of weighable residues. Due to its simpler measuring procedure, a surrogate parameter, e.g., the electrical conductivity (EC), is often used instead (Eaton and Franson 2005) and was also used in this study.

Table 6 Hach Lange cuvette tests used for determination of the chemical oxygen demand, nitrogen and phosphorus compounds

testkit	parameter	range	unit
LCK303	NH_4^+-N	2.0 - 47	mg/L
LCK314	COD	15 - 150	mg O_2/L
LCK414	COD	5 - 60	mg O_2/L
LCK138	TN	1 - 16	mg/L
LCK338	TN	20 - 100	mg/L
LCK339	NO_3^--N	0.23 - 13.5	mg/L
LCK340	NO_3^--N	5 - 35	mg/L
LCK341	NO_2^--N	0.015 - 0.6	mg/L
LCK348	TP/PO_4^{3-}-P	0.5 - 5	mg/L
LCK350	TP/PO_4^{3-}-P	2.0 - 20	mg/L

Determination of TDS was only possible in the effluent of the wastewater treatment plant. The samples were filtered through glass microfiber filters (Whatman 934-AH) and dried at 105°C until they reached a constant weight. TDS was determined four times. For each determination, the mean value of multiple (2 or 3) measurements was calculated. Sample volumes between 8 L and 19 L were evaporated for these measurements.

Together with the EC measured in these samples (EC meter: Multi 1970i, electrode: TetraCon 325, WTW), a conversion factor of 0.62 (±0.03) (mg×cm)/(L×µS) was determined. For instance, 2.78 g solids were obtained from evaporation of 8.52 L sample volume with an average EC of 526 µS/cm. Thus, the conversion factor for this sample is 2.78 g × 1,000 mg/g ÷ 8.52 L ÷ 526 µS/cm = 0.62 (mg×cm)/(L×µS).

TS were measured weekly in grab samples (dry weight at 105°C). 5-day biochemical oxygen demand (BOD_5), Cl, B, Na, K, Mg and Ca were determined in 10- or 12-hour mixed samples by an external laboratory in Windhoek (Namibia Water Corporation, NamWater). Sampling and transport to Windhoek was carried out less frequently than routine analyses. Thus, the number of measurements for these parameters is much lower than for the other parameters.

Total coliforms, E. coli and thermotolerant (or fecal) coliforms were quantified using IDEXX Colilert-18 and Quanti-Tray/2000. Enterococci were identified using IDEXX Enterolert-E. Grab samples were taken approximately once a week in sterile vessels and immediately prepared for analysis. The detailed procedures can be found in IDEXX Laboratories (2011a) and IDEXX Laboratories (2011b). All microbiological parameters were reported as most probable numbers (MPN), which is “an index of the number of […] bacteria that, more probably than any other number, would give the results shown by the test” (Bartram and Ballance 1996).

Sampling for helminth eggs was carried out in the microscreen inlet and outlet (before UV disinfection), and in the irrigation water extracted from the storage pond. For the determination, wastewater samples were sieved (Retsch, 20 µm, 200 mm x 50 mm). The material retained by the sieve was recovered in a centrifuge bottle and concentrated by centrifugation at

660 g and the addition of a sodium chloride/sucrose solution (relative density = 1.28). Helminth eggs were optically counted using a microscope (Axio Lab A1, Carl Zeiss) and a counting chamber (Sedgewick Rafter).

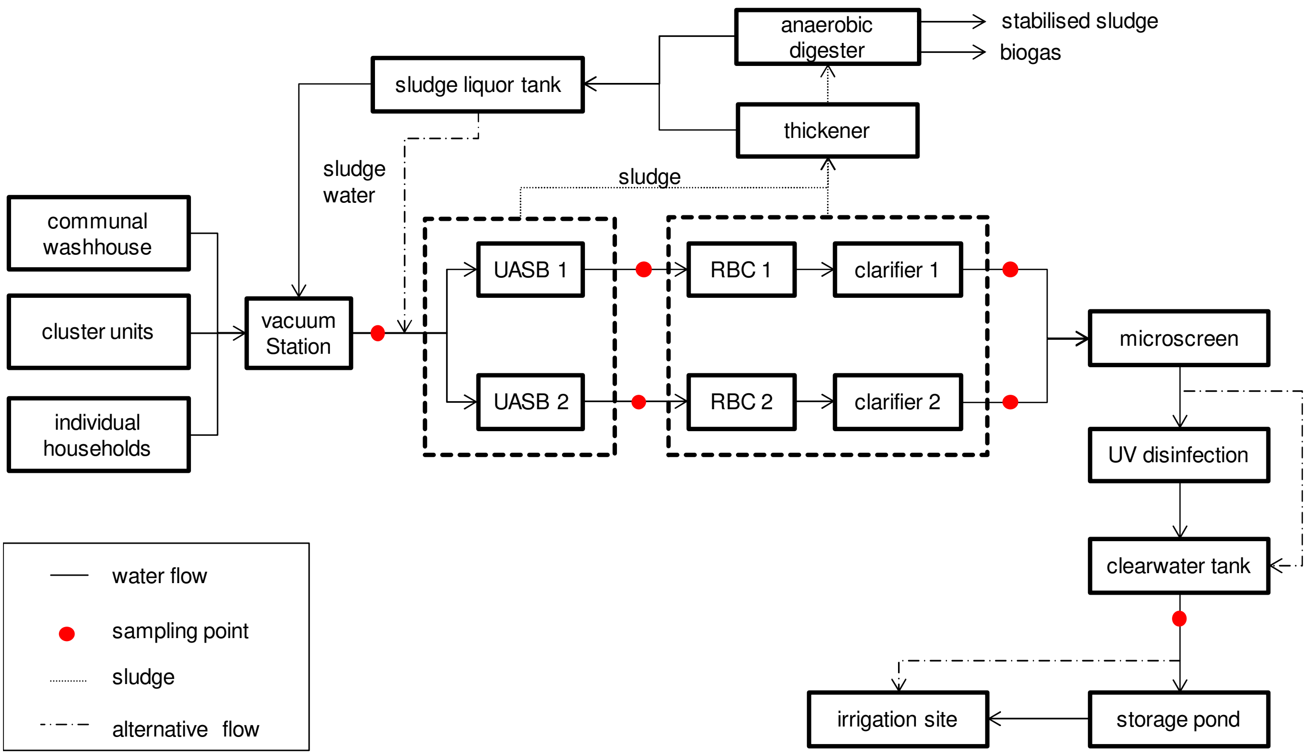

Figure 26 Flow chart of the sanitation system in Outapi with sampling points

3.4 Determination of utilization and the number of users

At the communal washhouse, billing is carried out on a pay-per-use basis, via a voucher system. Vouchers were sold at the OTC and at the sanitation facility. They were collected by the facility's caretakers and allowed the use of all the provided services during one visit.

The vouchers were used to determine the utilization. They were reclaimed from the caretakers by the laboratory assistant and then counted. From May 2014 to September 2015, utilization was determined from tally sheets. In the tally sheets, the number of visits for laundry washing and the number of visits for use of showers/toilets was recorded separately. The sheets were filled out daily by the caretakers at the communal washhouse and collected from time to time by the laboratory assistant.

For the individual households and the cluster units, data on the number of households and average household size were available from community workshops and socio-empirical surveys (Deffner and Mazambani 2010; Kramm and Deffner 2017, 2014).

3.5 Determination of specific loads

The water quantities can be determined separately for each type of sanitation facility via water meters installed in the tap water pipes. In contrast, water quality measurements are only possible in the influent of the wastewater treatment plant, which is a mixture of the wastewater received from all of the connected sanitation facilities.

TCOD, TN, TP and TDS loads were calculated using the water quantity data obtained from water meter readings and the concentrations measured in the influent of the wastewater treatment plant. When water use data were not available for a specific day, the water quantity was estimated using previous and subsequent data.

The sanitation facilities implemented in this project went into operation in several steps in 2013 and 2014. This is illustrated in Figure 27. First, operation of the communal washhouse started in May 2013. Then, the 30 cluster units were connected to the vacuum sewer system and became operational in November 2013. The individual households were connected starting from April 2014.

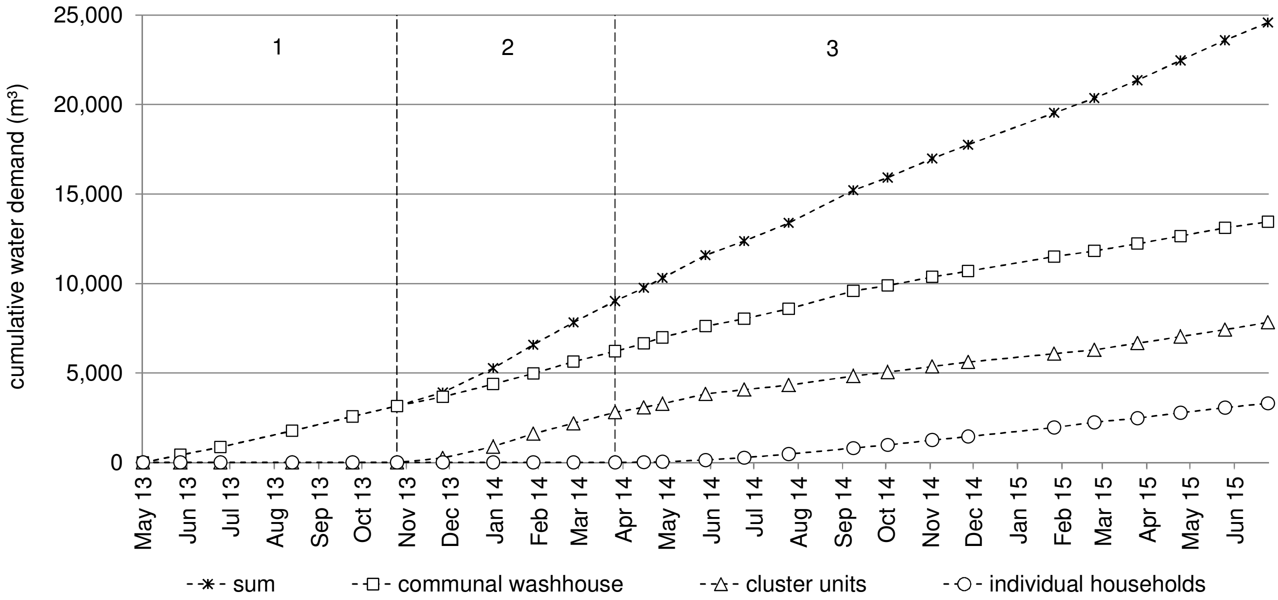

Figure 27 Cumulative water use in total, for the communal washhouse, the cluster units and the individual households; survey period 1: only communal washhouse (May 2013 to October 2013), survey period 2: communal washhouse and cluster units (November 2013 to March 2014), survey period 3: communal washhouse, cluster units and individual households (since April 2014)

The successive implementation made it possible to determine the loads separately. Initially, the determined loads represented only the loads from the communal washhouse (May 2013 to October 2013). After start-up of the cluster units, the determined loads originated from the communal washhouse and the cluster units (November 2013 to March 2014). Finally, the determined loads represented the loads from all sanitation facilities (since April 2014).

The measured TCOD, TN, and TP concentrations in the tap water samples were below the detection limit. Thus, when determining the loads per use for the communal washhouse, it was assumed that these concentrations in tap water are "zero". The TDS concentration in tap water was 40.2 (±2.7) mg/L (n = 3) and this was deducted from the total load; thus, loads per use correspond to input during water use.

The average specific load determined in this way was used to estimate the loads from the communal washhouse for the other time intervals. This means that, in the time interval from November 2013 to March 2014, the loads from the cluster units were determined as the difference between the total loads and the estimated loads from the communal washhouse.

The determined total loads originating from the cluster units were divided by the number of operational cluster units at that point in time. The number of cluster units in use was determined with the water meter records. Only the cluster units with a water use larger than zero were considered. The loads obtained for each cluster unit were divided by the average number of persons allocated to each cluster unit (average number of households per cluster unit = 3.5, average number of permanent household members = 3.4). In this fashion, the specific load per resident could be calculated.

For the last survey period (April 2014 to July 2015), the loads of the individual households were estimated using the average load of the communal washhouse and the average load of the cluster units. For the communal washhouse, total loads were estimated using the average number of uses per day and the determined average loads per use. For the cluster units, it was assumed that the loads are the same as determined for the time interval from November 2013 to March 2014. Both were deduced from the total loads. The loads from the individual households could thus be obtained.

3.6 Collection of online data

Online data was collected using the "Acron 7 – The Plant Chronicler" software. The collected process data mainly included level, flow, pressure and temperature measurements. The electricity and power demand was also registered for previously defined components (see appendix, page 230). In this work, data on the electricity and power demand, the effluent water quantities and water temperature, the relative pressure and water level in the vacuum tank were used.

3.7 Outliers and extreme values

Outliers and extreme values influence statistical key figures; parametric and nonparametric tests and should be accounted for prior to data analyses (Osborne and Overbay 2004). In this case, box plots were used for graphical identification using IBM SPSS Statistics Version 20. SPSS defines extreme values as values that deviate more than three times the interquartile range from the upper quartile value upwards or downwards from the lower quartile value.

Examples for factors causing extreme values and outliers in water quantity data are water meter reading errors and special events (such as leakages, maintenance works) or inappropriate use of the sanitation facility (for instance, taps left open). Water meter readings increment and can therefore be easily checked for consistency. Special events and inappropriate use of the facility are considered to be part of the usual operation and maintenance of the sanitation facility.

Thus, it was decided that outliers are to be considered as legitimate cases sampled from the correct population and they are included in the water quantity data. Only extreme values were deleted in the original data. To handle all data equally, this procedure was repeated for water quality data. Extreme values were deleted, outliers were not deleted.

Consequently, 23 values were excluded from water use data (0.7% of recorded values). The same procedure was repeated for the remaining variables. One value was excluded from data on total utilization rates (0.2% of recorded values) and 27 values (1.4%) were deleted from specific water uses. The data used for calculation of specific loads contained one extreme value (for EC or TDS), which was also deleted.

When data were further processed, e.g., when calculating loads or the water use per use, extreme values and outliers were deleted.

3.8 Definition of water quality criteria

The determination of water quality criteria consisted of two parts. One part focused on choosing and defining the parameters for monitoring the water quality for this specific project. Existing guidelines for irrigation water quality and water reclamation were reviewed. The project partners (TUDa and OTC) discussed and decided on the guidelines that were suitable for application in this specific case. Additional parameters and modifications were suggested by TUDa, based on information found in the literature.

The other part comprised sample collection, water quality analyses and the comparison of measured values with the chosen water quality objectives. Recommendations for the operation of the water reuse scheme were derived.

3.9 Costs of shared sanitation facilities

The capital and operation and maintenance costs of the communal washhouse and the cluster units were available from planning data or determined after implementation. A comparison with literature values was carried out.

3.10 Salt and nutrient management

Salt and nutrient balances were elaborated. Planning data was mainly taken from literature (TDS/EC conversion factor, sludge generation during wastewater treatment, algae concentration in the storage pond, evaporation, rainfall, irrigation demand, leaching requirement, alt and nutrient content of harvested crops). After implementation of the sanitation system, the literature values were replaced by monitoring data (EC values, TDS-, N-, P- and K-concentrations in the untreated and treated wastewater, precipitation and evaporation).

The quantity of salts leaving the system boundary via harvested crops was estimated using the average crop yields for Namibia (PWC 2005), the average length of their individual growth period (the time required for vegetation, flowering, yield formation and ripening, Doorenbos 1979), and the ash content of the raw crops as reported in nutritional data (USDA 2014). Time periods needed for preparation of the fields were not accounted for when calculating the potential number of harvests.

The average nitrogen content of the crops was calculated using the average yield for Namibia (PWC 2005), the average duration of the individual growth period (Doorenbos 1979) and the protein content of the fruits and vegetables (USDA 2014). The protein content of a food sample is determined from its Kjehldahl nitrogen content (Nielsen 2014). For different kinds of food, a specific ratio of protein to nitrogen is assumed (Nielsen 2014). Hence, it is possible to calculate the nitrogen content of the food from the protein content given in nutritional data (Nielsen 2014; USDA 2014). The same approach was applied for the calculation of phosphorus and potassium loads leaving the system boundary via harvested crops.

3.11 Energetic aspects

The potential contribution of co-digestion and co-generation to achieve energy self-sufficiency was evaluated using literature data. Barriers to its implementation were discussed based on the experiences made during implementation of the sanitation system in Outapi.

The electricity consumption of the sanitation system was assessed using monitoring data. Electricity costs and the significance of tariff structures for co-generation were evaluated using available information from several electricity supply entities in the region.

4 Results and discussion

4.1 Planning and implementation of wastewater collection, transport and treatment facilities

This chapter looks at the planning phase of the sanitation system and gives an overview of the main challenges experienced during implementation. It starts with a brief review of how water quantities and loads were estimated. The next section focuses on the design of the sanitation facilities and how transport of wastewater could be carried out. The importance of billing and its influence on the structural design and management of the shared sanitation facilities is outlined. The reasons for choosing a vacuum sewer system for wastewater transport and the main issues during its planning and implementation are presented in detail in the following section. It also anticipates some monitoring results from the operation of the vacuum sewers. Details on how wastewater and sludge treatment steps were chosen and details on water storage facilities are presented.

4.1.1 Input data

To design the sewers and wastewater treatment steps, figures on expected water quantities, concentrations and loads are required. Water and wastewater infrastructure is only weakly developed or non-existent in Northern Namibia (see Section 2.9.3, page 34). Hence, information that could be used as an evidence base for planning purposes in the region was not available among project partners or in the literature. For these reasons, assumptions regarding the future water use, loads, and concentrations had to be made in cooperation with the local experts, OTC and DRFN, based on their experience and the results of the community workshops (Deffner and Mazambani 2010).

Because the planned sanitation system had the objective of providing future users with a sufficient amount of water, the future water use was estimated at 60 L/(user×d), based on the recommendations given by Gleick (1996) and WHO and WEDC (2011) (see Section 2.2.1, page 12). This is more than the water use reported for informal settlements in Namibia (Deffner and Mazambani 2010; Uhlendahl *et al.* 2010). However, it was assumed that the specific water use would increase by improving access to water and sanitation.

The specific loads in Sperling (2007c) were taken as a basis for estimating loads and concentrations because they refer to developing countries and provide a complete dataset for all relevant parameters (see Table 2, page 16, Sections 2.2.1 and 2.2.3).

It was planned to connect 66 individual households, with an average household size of 4 persons, to the sanitation system. The 30 cluster units were intended to be used by 4 households each, with an average household size of 7 persons. The number of users for the communal washhouse was set at 250. Thus, the system was designed for up to 1,500 users (Table 7). Information regarding the household sizes was provided by ISOE and DRFN. The number of users of the communal washhouse, the number of cluster units and allocated households, and the number of individual households were discussed among the project partners and decided on by OTC (Deffner and Kluge 2013).

The project partners were aware that the chosen input data were subject to uncertainties. In view of the fact that better input data were not available, it was decided to proceed with the chosen assumptions. The risk of over- or underestimation of loads and water quantities was accounted for by installing two treatment lines and the possibility of cross-connecting and by-passing treatment steps (see Section 4.1.4, page 61*ff.*).

Table 7 Overview on the planned number of users

	communal washhouse	cluster units	individual households	safety margin	sum
households	-	120	66	-	
persons/household	-	7	4	-	
persons	250	840	264	146	1,500

4.1.2 Sanitation facilities

Collected wastewater can be percolated into the ground, stored and transported via human power or motorized vehicles, or be transported directly, without storage via sewers (Tilley *et al.* 2008). Available options for the collection of excreta include dry and flush toilets (Tilley *et al.* 2008). Sufficient water needs to be available for domestic and personal hygiene (Esrey *et al.* 1991). Hand washing after defecation, for instance, can considerably reduce incidences of diarrhea (Ejemot-Nwadiaro *et al.* 2015; Freeman *et al.* 2014). Thus, beside toilets, full sanitation provision needs to include facilities for laundry washing and personal hygiene (showers and wash basins).

Possible options for sanitation provision in Outapi were introduced during several community workshops in the project area (Deffner and Mazambani 2010). The first workshops in all three districts made it possible to analyze and understand the socio-technical system of water supply and sanitary conditions. Taking the situation analysis as a starting point, differing design requirements for new sanitation infrastructure were drawn up together with the OTC and inhabitants of the three informal settlements. This was an iterative process. The technical concept was adapted step-by-step, to meet actual neighborhood requirements, by applying empirical social research and different forms of interaction within district workshops (Deffner and Mazambani 2010; Kramm and Deffner 2017). The final design of the shared sanitation facilities was selected on the basis of the outcome of several meetings between OTC, DRFN, TUDa, BWT and ISOE, as well as from the findings of the community workshops.

An approach of community-based learning, targeting health-related behavior, was adapted and applied before the sanitation facilities became operational. The adaptation of the community health club (CHC) approach was developed, together with an NGO and local partners, to meet project and community needs (Deffner and Böff 2012; Waterkeyn 2010). The CHCs were a very important part of the implementation. The main objectives were long-lasting change of hygiene behavior, especially to reduce health risks; establishment of a routine and demand for using toilets, showers and wash basins; communication of benefits of sanitation facilities, to embed them in everyday life, and communication of adequate use of the new facilities, to prevent misuse and vandalism. Only both components together, the software (behavior and use) and the hardware (technology) can make such a project successful.

Some of the theoretically possible options for collection of excreta and sewage could be excluded right from the beginning. First of all, since the objective of this project was to reuse all of the water for agricultural irrigation, infiltration of the wastewater on the plots was not an option. In addition, the permeability of the soil in the project area is poor (Mendelsohn *et al.* 2000) and plot size was not always sufficient for infiltration of the water.

Due to the constraints imposed by the risk of floods, the centralized or decentralized storage of large amounts of untreated wastewater and excreta in, e.g., oxidation ponds, pit latrines, or similar facilities, is not acceptable. Furthermore, the generated greywater quantities would require very large storage facilities, not to mention a watertight design. Thus, sanitation options requiring storage facilities were excluded early in the planning process, leaving sewer systems as the method of choice for sewage transport. This also meant that flush toilets were required for collection of excreta. Implementation of flush toilets and sewers for collection and transport was also determined by other factors:

- The residents of Outapi pursue an urban lifestyle. Gardening or farming activities play a relatively minor role, resulting in a negligible demand for fertilizers and irrigation water on the household level (DRFN *et al.* 2010).
- Professional management of excreta and wastewater carried out by the municipality is seen as the key in protecting individuals from contagious material. Relying on the individual household's capabilities in excreta collection and disposal would probably not assist in improving the hygienic conditions of the informal settlements.
- The OTC already operates a gravity sewer system in the older parts of the city and operates public flush toilets at two open markets. Thus, trained staff is available. The residents are familiar with the concept of flush toilets.

The detailed design of the shared sanitation facilities was determined in close cooperation with the OTC. Regarding the communal washhouse, the OTC decided that a facility serving up to 250 residents should be implemented in a very young informal area. Assuming a utilization of 3 uses/(person×d) to completely cover personal hygiene and toilet use (assumption based on discussions with OTC and community members), the anticipated utilization rate would be 750 uses/d. Security and maintenance staff was provided by the OTC. The cluster units were designed so that they could provide sufficient services to four allocated households. The final design for the communal washhouse and the cluster units was elaborated by a Namibian civil consultant in compliance with the local regulations.

An important point during planning of these facilities was the system for revenue collection at the shared sanitation facilities. It should allow free access to some of the services provided and reliably collect revenues when using other services. The reason for this is that, in order to reduce open defecation in the informal settlements, the OTC opted for provision of free access to toilets and hand wash basins. The idea was to cross-subsidize the costs for the unbilled water by the paid uses (showers, laundry sinks).

Thus, prepaid water meters were chosen for revenue collection. In rudimentarily developed parts of Outapi, the OTC used prepaid standpipes for water supply. It therefore made sense to also use them for revenue collection in this project. The layout of the shared sanitation facilities was planned accordingly.

However, it turned out that the prices for prepaid water meters increased considerably between the planning phase and the intended installation date in 2013. Whereas the price for the supply and installation of one prepaid water meter was 3,145 NAD during concept design in 2009 (= 297 EUR including value added tax, average currency exchange rate in 2009, www.oanda.com), it increased to 6,876 NAD (= 477 EUR including value added tax, average currency exchange rate in 2013, www.oanda.com) until the time of installation in 2013. In terms of the price of tap water in Outapi in 2013 (10 NAD/m³), the budget required for the installation of the 26 prepaid water meters required for the communal washhouse (15 for showers, 11 for laundry sinks) would have corresponded to its presumed water use for more than 3 years ((6,876 NAD/meter × 26 meters ÷ 10 NAD/1000 L) ÷ (250 users × 60 L/(user×d) × 365 d/a × 100 L/m³) = 3.3 a).

In sum, prepaid water meters turned out to be too expensive for application in the communal washhouse and in the cluster units with respect to available funds and compared to the water quantities billed for. Furthermore, an external Namibian consultant was assigned a study on the development of suitable sanitation tariffs and billing modes for the infrastructure in this project (Oertzen 2012b). The consultant concluded that, "pre-payment water vending solutions have not yet established a satisfactory track record in Namibia". Instead, a voucher book system was recommended for billing at the communal washhouse. The final solution was decided on by the OTC. Considering the high prices for the prepaid water meters and the recommendations given by Oertzen (2012b), it opted for a voucher system for the communal washhouse and installation of prepaid water meters at the cluster units.

Thus, at the cluster units, the prepaid water meters were installed, as intended. A metal key is required for using the shower and the outdoor tap. The metal key can be topped up at the OTC pay kiosk.

At that time, the final layout of the communal washhouse was already decided and construction had already started. The spatial arrangement of the facility's sections, entrances, and exits fitted the original billing mode (prepaid water meters), e.g., separate control of visitor flows was not required and not accounted for. With the finally chosen billing mode (voucher system), it was no longer possible to create zones with services that are provided free of charge such as, for instance, free access to toilets and urinals, to maximize defecation and urination visits. Instead, an entrance fee had to be paid per visit, regardless the intended activities.

For the individual households, management issues that emerged during planning and implementation of the water supply and sewer connections were limited to decisions regarding the detailed procedure of how to connect to water supply and sewer system, e.g., if the connection of the households should be carried out by the technical department of OTC or by the residents themselves (with assistance) and whether charging and billing should be postpaid or prepaid. Connection to the sewer system was provided by OTC after payment of a connection fee. Pipes from the houses to the collection chambers had to be delivered by the households. To speed up the process of connection, OTC waived the connection fee. Following connection, the OTC's sole responsibility was to assure billing and conduct repairs, when needed.

4.1.3 Sewage transport

Sewer systems discussed for implementation in Outapi included gravity, pressure and vacuum sewer systems, small diameter gravity sewers and solids-free sewers. In North Namibia, floods may occur during rainy season and cause the spread of waterborne diseases, due to poor sanitary conditions in the entire region (Filali-Meknassi *et al.* 2014). Thus, implemented sewers need to be watertight, to prevent the intrusion of flood water in the sewer and, above all, to prevent contamination of flood water by mixing with wastewater.

The number of residents is increasing in Outapi. The population is growing in total and due to urbanization (see Section 2.9.3, page 34). This development is a challenge for sanitation infrastructure planning and implementation. Thus, sewer systems should be flexible in order to allow connection of additional users in the future.

These requirements can be met by vacuum or pressure sewers. Compared to gravity sewers, these sewer types provide advantages when installation takes place in flat terrain with a low population density and sandy soils, as is the case in the project area (Mendelsohn *et al.* 2000). Under such conditions, construction costs are usually cheaper than for gravity sewers, because pipes are installed at a smaller depth and because centrally located vacuum pumps replace the pumping stations required for lifting the wastewater when using a gravity sewer system (DWA 2008a; USEPA 2000).

For pressure sewers, a pump is required at every entry point (e.g., on the household level) for discharging the collected wastewater to the main (DWA 2008b; USEPA 2002). Compressed air flush stations are possibly required to prevent formation of H_2S in the sewers. Thus, when implementing a pressure sewer system, a number of technical components need to be installed within the network area. This requires strong institutional organization and needs to be considered for operation and maintenance (USEPA 2002).

For vacuum sewers, usually only one central pump station is required to maintain the low pressure. Vacuum sewers transport a mixture of air and wastewater; thus, oxygen depletion is not an issue in the sewers. In case of a leakage, water or air will enter the sewers while the wastewater is still transported inside the sewers and thus cannot contaminate the environment. Leakages causing an inflow of air into the sewers are detected immediately because they cause higher pressure inside the sewers and longer operating times of the vacuum pumps (Bowne *et al.* 1991; DWA 2008a).

Considering these preconditions, a vacuum sewer system was chosen as the most adequate technology option for collection and transport of wastewater in the city of Outapi. However, in Namibia and in South Africa, the reputation of sewer systems other than conventional gravity sewers has suffered due to misuse, misapplication and lack of operation and maintenance (Ashipala and Armitage 2011; DRFN 2011; Taing *et al.* 2011).

Despite the general technical suitability of vacuum sewers, some shortcomings or disadvantages exist (Bowne *et al.* 1991; GTZ 2005; Lens *et al.* 2001; Little 2004):

- Correct construction, operation and maintenance are required
- Additional energy is needed for the operation of a vacuum sewer system
- In case of a complete system failure, the houses connected to the drainage system cannot discharge their wastewater
- Limited durability and breakage of vacuum pipes (especially where they are connected) are possible when inadequate pipe material is used and installation is not carried out correctly
- Numerous valves (one at each collection chamber) can cause operational problems and require maintenance

However, some of these constraints are not unique to vacuum sewer systems but are problematic for any kind of technical infrastructure. Lack of proper management is a problem for any sanitation system. Salifu (1997) states, in his résumé of the condition of gravity sewer systems in Ghanaian cities, that a responsive operation and maintenance management scheme must be established in the long term. Other authors have also identified a lack of maintenance as the origin of deterioration of water-related infrastructure in Zimbabwe or in a township in Johannesburg, South Africa (African Development Bank Group 2010; Mangizvo 2009; The World Bank 2001).

To return to vacuum sewer systems, Bowne *et al.* (1991) summarize: “In short, many of the major objections to the use of vacuum systems are not well founded. These systems have been acceptable in a variety of applications and locations. Any hypothetical or abstract difficulty that can be applied to the vacuum system can also be applied to the more conventional systems.”

All in all, the challenges to be faced when implementing sanitation systems, in general, and sewer systems, in particular, seem not to originate from the technology itself but from the handling of installed facilities. Thus, despite the unfavorable preconditions, the implementation of vacuum sewers was continued for this project. Emphasis was placed on correct construction and sufficient training of the operators to enable and ensure long-term operation.

The vacuum pumps should maintain a pressure of at least 0.6 to 0.7 bar (DWA 2008a) or a relative pressure of -0.4 to -0.3 bar. Because the results section of this dissertation does not contain a separate chapter with the monitoring results for the vacuum sewer system, some results of the technical monitoring are anticipated at this point.

Figure 28 shows the mean relative pressure (points and dashed line) and the operating hours with a mean pressure below and above -0.3 bar (columns) for the vacuum sewer system in Outapi. Data from 1 August 2013 until 10 June 2015 are included. Thus, the total recorded operating time is lower for June 2015 than for the other months. The mean relative pressure for each month was always below -0.3 bar (average = -0.48 bar). The operating hours with a mean relative pressure above -0.3 represent only 3% of the total operating time. This percentage varies between 0% and 9%, except for May 2015. During this month, extensive maintenance of the wastewater pumps was carried out, which required frequent vacuum reliefs.

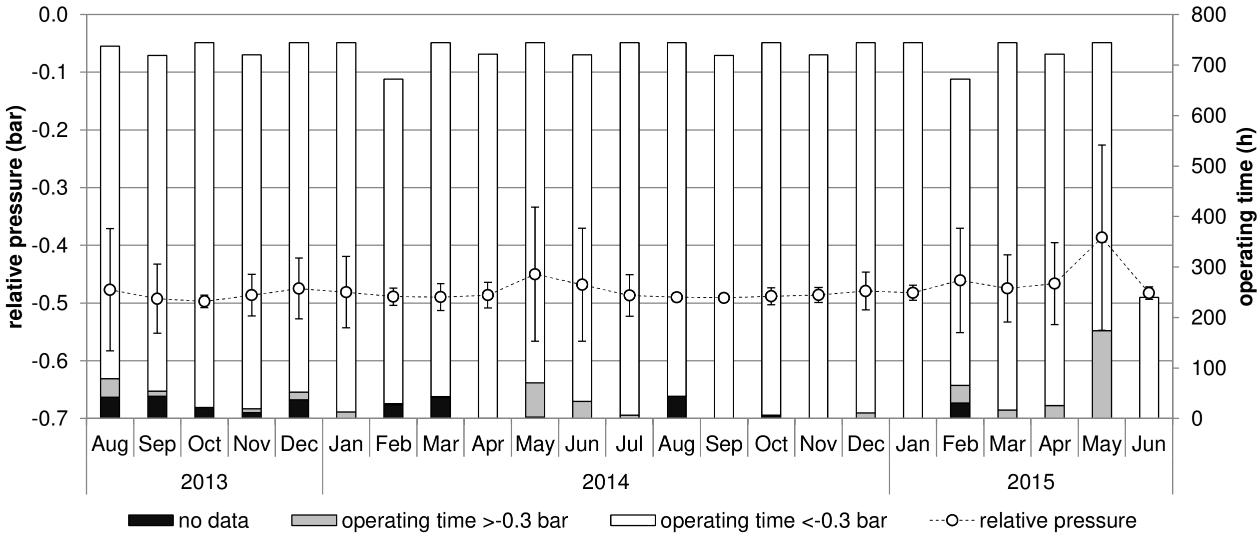

Figure 28 Monitoring results for the vacuum sewer system: mean relative pressure (bars represent standard deviation of the mean) and operating times for mean relative pressures >-0.3 bar and <-0.3 bar

Thus, the imperfections reported for previously installed vacuum sewer systems in South Africa and Namibia (Ashipala and Armitage 2011; DRFN 2011; Taing *et al.* 2011) did not occur in this case. Altogether, the vacuum sewer system that was installed in Outapi operated reliably. Malfunctions were detected by the operators or reported by residents and eliminated rapidly.

Adequate installation of the vacuum sewer system is a prerequisite for sustainable long-term operation but can be difficult to achieve under unfavorable conditions. Even though awareness regarding this issue existed for this project, some requirements were neglected by the construction company and the civil consultant responsible for supervision. This led to substantial delays in implementation. The following points turned out to be crucial for this project:

- Pipes and fittings must be certified for operation in vacuum conditions by the pipe manufacturer and approved by the vacuum system supplier. In this case, HDPE pipe material was delivered from Germany. The PVC pipes delivered from a South African supplier did not meet all requirements specified by Bilfinger Water Technologies.

- It is essential to perform pressure tests during pipe laying. Realization should be followed up by a reliable supervisor.
- Solvent welding of PVC pipes can be difficult when trenches are excavated in sandy soil. Careful cleaning of the surfaces is required. If it is not possible to assure this, electric welding of HDPE pipes is recommended instead.
- Electric welding is less susceptible to disturbances via, e.g., sand. Trained personnel is required. In this case, welding was performed under the supervision of trained staff of BWT. During installation, staff of the OTC was trained in electric welding to assure sufficient capacities for later repairs or extension of the vacuum sewer system.

Important points during planning were also the risks associated with misuse of the sewers for garbage disposal and use of problematic material for anal cleansing. This is always of concern when providing sanitation facilities to areas where official garbage disposal is non-existent and if residents are not (yet) familiar with sanitation facilities (Ashipala and Armitage 2011; DRFN 2011; Naranjo *et al.* 2010; Taing *et al.* 2013). Foreign objects transported in sewers can cause clogging and might damage pumps and other devices for wastewater transport and treatment.

The use of problematic material for anal cleansing and the use of toilets as a possibility for garbage disposal can be reduced to a certain degree by user involvement and supervision by the caretakers at the communal washhouse. However, complete prevention is not achievable. Thus, measures for protecting the wastewater transport and treatment devices were implemented in this project. A coarse screen was installed at the communal washhouse. Larger objects are retained in the sumps of the collection chambers before entering the vacuum sewers. Prior to wastewater treatment, a combination of stone trap, choppers and rotary lobe pumps was installed for removal and crushing of items such as paper, cloth and plastic.

After implementation, foreign objects such as razors, underwear, roll-on deodorants, etc. were found in the wastewater but did not cause major problems. However, the presence of larger amounts of hard stones of the eembe fruit (*Berchemia discolor*, size of the date-like stones ≈ 2 cm) challenged the wastewater pumps. The stones from the eembe fruit are added to the wastewater via feces because they are contained in fruits that are part of the local diet. They could not be removed reliably by sedimentation in the collection chambers or in the vacuum tank and then passed stone trap and crusher, leading to rapid wearing of the rotary lobes. As a first measure, the rubber rotary lobes were replaced by metal lobes. They were also blocked by the stones from the eembe fruit and re-start after standstill was sometimes difficult.

The pumps themselves also did not appear to be suitable for the pumping of sewage in Outapi. During the project period, one out of three installed pumps had to be replaced; in another one, a defect became apparent. Even after replacement of the rotors, they did not achieve the required flow rate.

Retrofitting was required to ensure removal of the stones from the eembe fruit to reduce spare parts consumption of the wastewater pumps and to make sure the required flow rate was met.

Therefore, in November 2015, a non-clogging impeller pump was installed behind the crusher, which fed the non-pressurized water to a bar screen. In this way, the stones from the eembe fruit and other foreign objects could be separated and the water could be fed to the treatment plant at sufficiently high flow rates.

4.1.4 Wastewater and sludge treatment

The kind of wastewater treatment to be applied depends on the treatment objectives (e.g., discharge or reuse) and input parameters. Depending on the wastewater composition and treatment objectives, different technology options are possible. Hence, individual solutions are required. Because Namibian water quality objectives did not exist, own objectives were developed during the course of this project (see Section 4.5, page 122*ff.*).

In this case, the water was intended to be used for irrigation of vegetables. Hence, pathogens needed to be reduced and nutrients had to remain in the water, for use as fertilizers. Based on these preconditions, it followed that a disinfection step was required, nutrient elimination was not needed, and a storage facility had to be planned to compensate for the gap between irrigation water supply and demand. Organics and particles needed to be reduced, to prevent anaerobic conditions in the storage facility and to protect irrigation infrastructures.

The expected concentrations of the main constituents were much higher than in usual domestic sewage (Table 9, page 65). Compared to the categories given by Tchobanoglous *et al.* (2004) and Henze (2008), the composition of the untreated wastewater in Outapi is considered “strong” or “high”.

Due to the expected high mean TCOD concentration, an anaerobic wastewater treatment step was planned, to convert the organics in the water into biogas. The wastewater was first pretreated in combined sedimentation/UASB (upflow anaerobic sludge blanket) reactors and then further treated, aerobically, with rotating biological contactors (RBCs) with downstream lamella clarifiers. Organic compounds were oxidized and nutrients were, intentionally, mostly retained in the water for fertilization purposes. Solids and helminth eggs were removed from the secondary effluent by lamella clarifiers and a drum-type microscreen. The effluent was disinfected by UV radiation before being stored in a pond for reuse as irrigation water (Figure 19).

Upflow anaerobic sludge bed reactors (UASB) were chosen for anaerobic pretreatment because they are easy to operate and can increase biogas production at the plant. Particulate organics are retained, hydrolyzed and partly converted into CH_4 and CO_2 (Chernicharo 2007).

Aerobic post-treatment is mandatory, because anaerobic pretreatment removes organic carbon to a maximum of around 70% (Chernicharo 2007); this is far away from being suitable for fulfilling reuse usability standards. In order to choose the appropriate technique, various aerobic processes (conventional activated sludge process, sequencing batch reactor, trickling filter, rotating biological contactor, biofilter, submerged fixed bed reactor) were compared and evaluated regarding their cleaning performance, sludge production, adaptability to fluctuations, capital costs, operating costs, emissions, process simplicity, robustness, suitability for relatively small catchment areas and energy consumption (Müller 2011).

Table 8 Evaluation of the considered aerobic processes regarding selected criteria that are relevant for the implementation in Outapi using information found in the literature (Müller 2011), 0 = does not fulfil the criterion, 1 = meets the criterion, 3 = meets the criterion to the highest degree, CAS = conventional activated sludge process, SBR = sequencing batch reactor, RBC = rotating biological contactor, SFB = submerged fixed bed reactor, o&m = operation and maintenance

criterion	**CAS**	**SBR**	**trickling filter**	**RBC**	**biofilter**	**SFB**
cleaning performance	3.0	3.0	2.0	2.0	2.5	2.5
sludge production	3.0	3.0	1.0	1.0	2.0	2.0
adaptability to fluctuations	2.5	3.0	1.0	1.0	0.5	2.0
capital costs	1.0	1.0	1.5	2.0	1.0	2.0
o&m costs	1.0	1.0	2.5	2.0	2.0	2.0
emissions	2.0	1.5	2.5	3.0	3.0	3.0
process simplicity	1.0	2.0	2.5	2.5	1.5	1.5
robustness	1.0	1.5	2.5	2.5	1.5	1.5
suitability for relatively small catchment areas	1.0	3.0	3.0	3.0	2.0	2.0
energy consumption	1.0	1.0	2.5	3.0	0.5	1.5
total	**16.5**	**20.0**	**21.0**	**22.0**	**16.5**	**20.0**

The considered criteria were evaluated by assigning a value ranging from 0 to 3 (0 = does not fulfil the criterion, 1 = meets the criterion, 3 = meets the criterion to the highest degree, Table 8). The robustness of this review was examined by a weighting of the criteria (variant 1: no weighting, variant 2: priority to robustness and low power consumption, variant 3: priority to energetically favorable variants, variant 4: costs, Figure 29). In all approaches, the rotating biological contactor achieved the best results (closely followed by trickling filters) and was therefore chosen as the aerobic treatment step for Outapi.

Soil-transmitted helminth infections are the most important health concern when reclaiming water for agricultural reuse (WHO 1989). For removal of helminth eggs and larvae, a drum-type microscreen was installed in the effluent of the lamella clarifiers, prior to the disinfection step. Helminth eggs that are relevant for wastewater treatment have a diameter between 20 µm and 80 µm (Jimenez *et al.* 2007; WHO 2004). Thus, a mesh size of 15 µm was chosen. In addition, the microscreen should ensure an effluent that is solids-free, to a large extent, and suitable for UV disinfection. After implementation, problems were experienced with the microscreen and UV disinfection; these are discussed in Section 4.5.2.2 (page 127*f.*) and Section 4.5.2.8 (page 133*ff.*).

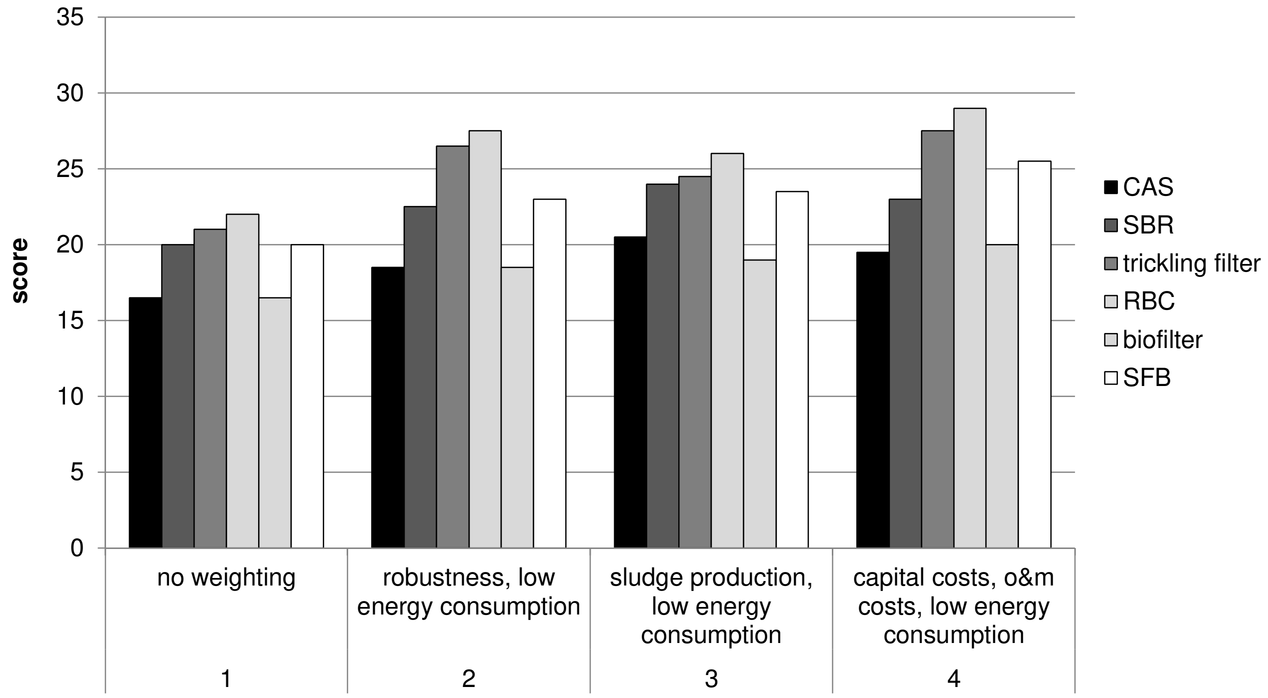

Figure 29 Overview on the results for the considered aerobic treatment processes after criteria weighting (Müller 2011), variant 1: no weighting, variant 2: double weighting of the criteria “robustness” and “energy consumption”, variant 3: double weighting of the criteria “sludge production” and “energy consumption”, variant 4: double weighting of the criteria “capital costs”, “o&m costs” and “energy consumption”, CAS = conventional activated sludge process, SBR = sequencing batch reactor, RBC = rotating biological contactor, SFB = submerged fixed bed reactor, o&m = operation and maintenance

During the planning for disinfection, membrane filtration, UV radiation, chlorination and application of chlorine dioxide or ozone were considered. Disinfection by membrane filtration appeared to be too energy consuming and too expensive in terms of operation (Norton-Brandao *et al.* 2013) and was not selected for Outapi. Ozone generation has to take place in situ (Tchobanoglous *et al.* 2004) and was therefore also eliminated as an option for disinfection. Because a denitrification step was not projected during wastewater treatment, the application of chlorine seemed too difficult to control; chlorine is initially consumed by ammonium ions and nitrite ions before free chlorine is available for disinfection (Tchobanoglous *et al.* 2004). Chloramines react further and produce nitrogen gas; however, nitrogen had to remain in the water for fertilization purposes (IWA 2012). In addition, chlorine, together with organic substances, produces odorous and harmful compounds in wastewater (IWA 2012); it is a highly toxic substance (IWA 2012) and was therefore not considered for wastewater disinfection in Outapi. As a result of these considerations, disinfection with UV radiation was chosen as the preferred technology, mainly due to safety concerns, its relatively easy handling and the ammonium concentration in the water.

The need for flexibility regarding changing wastewater quantities was also accounted for during planning of the wastewater treatment steps. Two identical treatment lines were installed, each consisting of a sedimentation/UASB reactor, RBC, and a lamella clarifier (Figure 19). When wastewater quantities are low (e.g., during start-up and the initial operation), the wastewater is treated in only one line. Once wastewater volumes increase, the second line can be put into operation. The set up also allows the bypassing and cross-connection of the components. For instance, it is possible to operate two UASB reactors for sedimentation/anaerobic pretreatment and only one RBC/lamella clarifier for aerobic treatment, or to carry out sedimentation/anaerobic pretreatment with only one UASB, and aerobic treatment with two RBCs/lamella clarifiers. Like this, maintenance and repairs are also easier to perform. One component can be bypassed while the second one is still available for treatment. Space requirements of future additional infrastructures (e.g., additional wastewater treatment lines) were also considered for installation of wastewater treatment facilities in the provided area.

Sewage sludge from the UASB reactors and the RBCs was collected in a thickener. Originally, its stabilization in an anaerobic digester (thermophilic digestion) and addition of crop residues from agriculture (biomass), to increase biogas production, was planned. The generated electricity could partly be used at the plant and the excess fed to the grid and be credited for by the local electricity supplier. The produced heat could be used on site for maintaining the required temperature for thermophilic digestion and increasing the temperature of the wastewater fed to the UASB reactors via heat exchangers. The biosolids originating from co-digestion of sewage sludge and biomass were supposed to be dried in the sun and then be used as fertilizer and bulking material on the agricultural fields.

A number of obstacles arose during the attempt to implement the thermophilic co-digestion of sludge and crop residues. In the end, the energetic concept was not fully implemented, due to limitations regarding material flows, as well as organizational and financial issues. These issues are discussed in more detail in Chapter 4.7.1 (page 160*ff.*).

4.1.5 Loads and concentrations during wastewater treatment

The detailed planning and process engineering was carried out by BWT. The loads and concentrations for each treatment step and the assumptions for reduction rates were provided by BWT (2011) (Table 9). Additional design data and process parameters are available in the appendix (Table 47, page 230). A comparison of planning data with monitoring data is presented in Section 4.2.10 (Table 11, page 89). Because references are not given in BWT (2011), a comparison with literature data is also included in Section 4.2.10. The actually determined specific loads for each type of sanitation unit are discussed in Section 4.3 (page 93*ff.*).

The expected loads and concentrations in the untreated wastewater were 150 kg/d and 1,667 mg/L for TCOD, 75.0 kg/d and 833 mg/L for BOD_5, 12.0 kg/d and 133 mg/L for TN, 1.5 kg/d and 16.7 mg/L for TP, and 90.0 kg/d and 1,000 mg/L for TSS (Table 9). The return flow from the anaerobic digestion unit is included in the UASB influent data in Table 9. Depending on the parameter, the contribution of the return flow to loads and concentrations was estimated at 5% or 15% of the untreated water.

The TCOD, DCOD and BOD_5 reductions during sedimentation/anaerobic pretreatment were estimated at 50% (Table 9, BWT (2011)). TN reduction was not anticipated. The reduction of TP was estimated at 20% and the TSS reduction at 70%.

Table 9 Planning data for loads and concentrations in the untreated water and in the effluent of the treatment steps considered for Outapi (BWT 2011), conc. = concentration, ww = wastewater

	untreated wastewater		untreated ww and return flow anaerobic digester		sedimentation, anaerobic pretreatment (UASB)			aerobic treatment (RBC), secondary clarification (LC)			microscreen, UV disinfection			storage pond	
	load	**conc.**	**effluent load**	**effluent conc.**	**effluent load**	**effluent conc.**	Δ	**effluent load**	**effluent conc.**	Δ	**effluent load**	**effluent conc.**	Δ	**effluent**	Δ
	kg/d	mg/L	kg/d	mg/L	kg/d	mg/L	%	kg/d	mg/L	%	kg/d	mg/L	%	mg/L	%
TCOD	150	1,667	158[a]	1,750[a]	78.8[e]	875[e]	-50	6.3[i]	70.0[i]	-92	4.5[i]	50.0[i]	-29	-	-
DCOD	105	1,167	110[a]	1,225[a]	55.1[e]	613[e]	-50	-	-	-	-	-	-	-	-
BOD_5	75.0	833	78.8[a]	875[a]	39.4[e]	438[e]	-50	2.7[i]	30.0[i]	-93	2.3[i]	25.0[i]	-17	-	-
TN	12.0	133	13.8[b]	153[b]	13.8[f]	153[f]	0	11.8[j]	131[j]	-14	11.8[f]	132[f]	0	-	-
NH_4^+-N	10.4	115	10.4[c]	115[c]	10.4[f]	115[f]	0	11.3[k]	126[k]	+10	11.3[f]	126[f]	0	-	-
org. N	3.5	38.3	3.5[d]	38.3[d]	3.5[f]	38.3[f]	0	0.0	0.5	-99	0.05[f]	0.5[f]	0	-	-
NO_3^--N	0.0	0.0	0.0	0.0	0.0	0.0	-	0.5	5.0[l]	-	0.5[f]	5.0[f]	0	-	-
TKN	13.8	153	12.0	133	13.8	153	0	11.4	127	-18	11.4	127	0	-	-
TP	1.5	16.7	1.7[b]	19.2[b]	1.4[g]	15.3[g]	-20	-	-	-	-	-	-	-	-
TSS	90.0	1,000	94.5[a]	1,050[a]	28.4[h]	315[h]	-70	2.7[i]	30.0[i]	-90	0.45[i]	5.0[i]	-83	-	-

[a] 5% of untreated wastewater
[b] 15% untreated wastewater
[c] 15% of untreated wastewater, NH_4-N = 75% of TN
[d] 15% of untreated wastewater, organic N = 25% of TN
[e] reduction: 50%
[f] no reduction
[g] reduction: 20%
[h] reduction: 70%
[i] water quality objective
[j] incorporation into biomass = 5% of BOD_5
[k] NH_4-N = 96% of TN
[l] NO_3-N = 3.8% of TN

TCOD = total chemical oxygen damand, DCOD = dissolved chemical oxygen demand, BOD_5 = 5-day biochemical oxygen demand, TN = total nitrogen, NH_4^+-N = ammonium nitrogen, org. N = organic nitrogen, NO_3^--N = nitrate nitrogen, TKN = total Kjehldahl nitrogen, TP = total phosphorus, TSS = total suspended solids

Reduction rates during the aerobic treatment and secondary clarification follow from the final water quality. The water quality objectives determined for planning were 70.0 mg/L for TCOD, 30.0 mg/L for BOD_5 and 30.0 mg/L for TSS. Thus, intended removal rates during the aerobic treatment were 92% of the TCOD, 93% of the BOD_5 and 90% of TSS.

Incorporation of TN into biomass during aerobic treatment was estimated at 5% of the BOD_5 in the influent. The TN concentration in the effluent would then be 131 mg/L (= 153 mg TN/L - 0.05 × 438 mg BOD_5/L). Because removal was not foreseen, incorporation of TP into biomass was not calculated for the aerobic treatment step.

For microscreening, effluent concentrations of 50.0 mg/L for TCOD, 25.0 mg/L for BOD_5 and 5,0 mg/L for TSS were anticipated. This corresponds to reduction rates of 29%, 17% and 83%, respectively.

These relatively high TN and TP concentrations in the effluent meant increased risks of salinization, overfertilization and eutrophication on the agricultural fields. These issues were a major concern during planning and implementation. Hence, a comparison of planned and monitored salt and nutrient contents and a discussion of possible management measures is provided in Section 4.6 (page 137*ff.*).

Sludge production of the UASB reactors was estimated at 90 kg TSS/d. 66.2 kg TSS/d are contributed by sedimentation of TSS (= 94.5 kg TSS/d × 0.7). 23.6 kg/d are contributed by a specific excess sludge production of 0.15 kg TSS per kg COD applied (= 0.15 TSS/kg COD × 158 kg TCOD/d).

Sludge production in the RBCs is due to the growth of cell biomass and to sedimentation of TSS. Sludge production of the RBCs and removal by lamella clarifiers and microscreen was estimated at 61 kg TSS/d. 27.9 kg TSS/d are contributed by removal of TSS (= 28.4 kg TSS/d - 0.45 kg TSS/d). 33.0 kg TSS/d are contributed by an assumed TSS production of 0.9 kg TSS per kg BOD_5 removed (= 0.9 kg TSS/kg BOD_5 × (39.4 kg BOD_5/d - 2.7 kg BOD_5/d)).

4.1.6 Water storage

The treated water was stored in a pond with a capacity of 3,712 m³, sufficient to store the effluent of the wastewater treatment plant during more than one month (e.g., during harvest seasons or the preparation of the irrigation area for the next growing phase; Zimmermann *et al.* 2017b). The storage pond consists of two basins. The water flows into the first basin, whose capacity is about 256 m³, and overflows into the second larger basin with a capacity of about 3,456 m³. The surface area of both ponds amounts to 1,855 m². The storage pond was planned and implemented by ISOE.

4.1.7 Conclusions

This chapter presented an overview on the main choices made and the challenges experienced during the planning and implementation phase of this project. The occurrence of floods and the dynamic population development were determining factors when choosing options for sewage collection, transport and treatment.

Regarding sewage transport, the requirements of a watertight and flexible system were best met by vacuum sewers. Among Namibian stakeholders, there were strong reservations regarding implementation of this kind of technology. However, close coordination with all partners led to successful realization in Outapi. Following implementation, the vacuum sewers worked reliably. Major threats arose from insufficient supervision during construction, especially regarding the tightness of the vacuum sewers. Facilitating regular inspections during construction and supporting the training of local staff for later repairs or extensions is therefore recommended.

The example of the vacuum sewers also demonstrated that the planning process has to intensively involve all project partners, in order to make sure that the most suitable technology is implemented, even though there may be reservations among stakeholders.

From the number of potentially available technology options for wastewater collection and transport, only a few remained that could meet the requirements of the specific local context and the project objectives. There was the need to avoid large centralized or decentralized storage volumes, due to the risk of floods, the requirement to prevent mixing of wastewater with flood water, the need for an adjustable system size, and the objective of collecting the water and nutrients for reuse purposes. In the end, flush toilets and vacuum sewers remained the only feasible options. Hence, the number of implementable technology options for sanitation systems can be very limited, due to specific local conditions.

A major obstacle and uncertainties arose from the limited knowledge regarding the wastewater characteristics. It was not possible to validate the assumptions about the projected number of users or uses, the specific water use, and loads. This demonstrates the necessity for collecting such data. When planning occurs under such uncertain conditions, the habits and preferences of the local population need to be analyzed as thoroughly as possible. It is also important to account for these uncertainties by planning flexible wastewater treatment, e.g., via installation of two treatment lines, space for extensions, or interchangeable machinery and equipment.

The TCOD concentrations were expected to be relatively high. Thus, an anaerobic pretreatment was planned to energetically utilize the organics contained in the water. High TN and TP loads and concentrations were also anticipated. Hence, measures to mitigate the risks of over-fertilization and soil salinization on the agricultural fields were foreseen. It was clear that the nutrient and salt content of the water needs thorough monitoring after implementation.

Because Namibian regulations regarding water quality for irrigation did not exist, water quality objectives had to be developed. This was an important step in order to provide suitable irrigation water and to support operation and maintenance at the wastewater treatment plant.

For the shared sanitation facilities, choice and implementation of an appropriate billing system was challenging. Users should be motivated to save water. Hence, volumetric billing systems (postpaid or prepaid) should be preferred. Additionally, there was the OTC`s wish to provide free access to sanitation and hygiene, meaning free access to toilets and hand washing facilities. It is obvious that this conflicts with the need for full cost recovery.

In Namibia, investment costs for setting up the infrastructure for, e.g., prepaid water meters, were very high, compared to the cost of the water volumes billed for. A solution would be to introduce postpaid billing. Postpaid billing of the users would be easy for individual households, but prone to conflicts when pursued for the jointly managed cluster units.

For maximum flexibility, the layout of a shared sanitation facility should allow modifications of the billing system. For instance, separate control of visitor flows is required if an operational concept includes free access for one or several uses and charges for other uses. This can be achieved via the separation of visitor flows or by prepaid systems (e.g., coin-operated showers and laundry sinks). If the operational concept cannot be agreed on before planning and construction starts, this needs to be considered when planning the spatial arrangement of the sections, entrances, and exits, because switching from one operational arrangement to another one might be very difficult once the permanent structures of the building are finished.

During planning, close attention was paid to the possibility of foreign object damage caused by misuse of the toilets and vacuum sewers, e.g., as a means of garbage disposal, and the utilization of problematic anal cleansing materials. Hence, measures for removal and shredding of such objects were implemented. In the end, foreign objects that were expected to enter the water (e.g., roll-on deodorants, sanitary towels, underwear, razors etc.) could be retained. In contrast, the stones from the eembe fruit challenged the sewage pumps in a way that made further retrofitting necessary. In general, possibilities for technical supplements or for exchange of units should already be provided during planning.

After implementation and start-up of the sanitation system, the wastewater characteristics, specific loads and water use, and the relatively high nutrient and salt content of the irrigation water needed to be addressed further. Both points were investigated during the technical monitoring. The results are presented in Chapter 4.3 (page 69*ff.*) and 4.6 (page 137*ff.*).

4.2 Water quantities and quality

This chapter provides an overview of the water quantities, the main chemical, physical and biological characteristics of the untreated water and the changes during wastewater treatment. The data described here were used as the basis for the results and discussions presented in later chapters. It represents the input used for the determination of the specific loads and water uses for each type of sanitation facility (Chapter 4.3, page 93*ff.*), the definition and control of water quality criteria (Chapter 4.5, page 122*ff.*), and for defining measures for salt and nutrient management (Chapter 4.6, page 137*ff.*).

Some of the monitoring results will be presented in a later section. For instance, the data on pH are contained in Chapter 4.5.2.3 (page 130*ff.*) and data on electricity consumption of the vacuum sewers and wastewater treatment steps are described in Chapter 4.7 (page 160*ff.*). The performance of the microscreen is discussed in detail in Chapter 4.5.2.2 (page 127*ff.*).

4.2.1 Water quantities

During the whole monitoring period, the effluent of the wastewater treatment plant was, on average, 30.3 (±11.8) m³/d (Figure 30). In August, September, and October 2013, only water from the communal washhouse was treated (in average 16.6 (±10.2) m³/d). In November 2013, the cluster units became operational (see Figure 27, page 49). The water use increased to 38.4 (±14.4) m³/d (November 2013 up to and including March 2014).

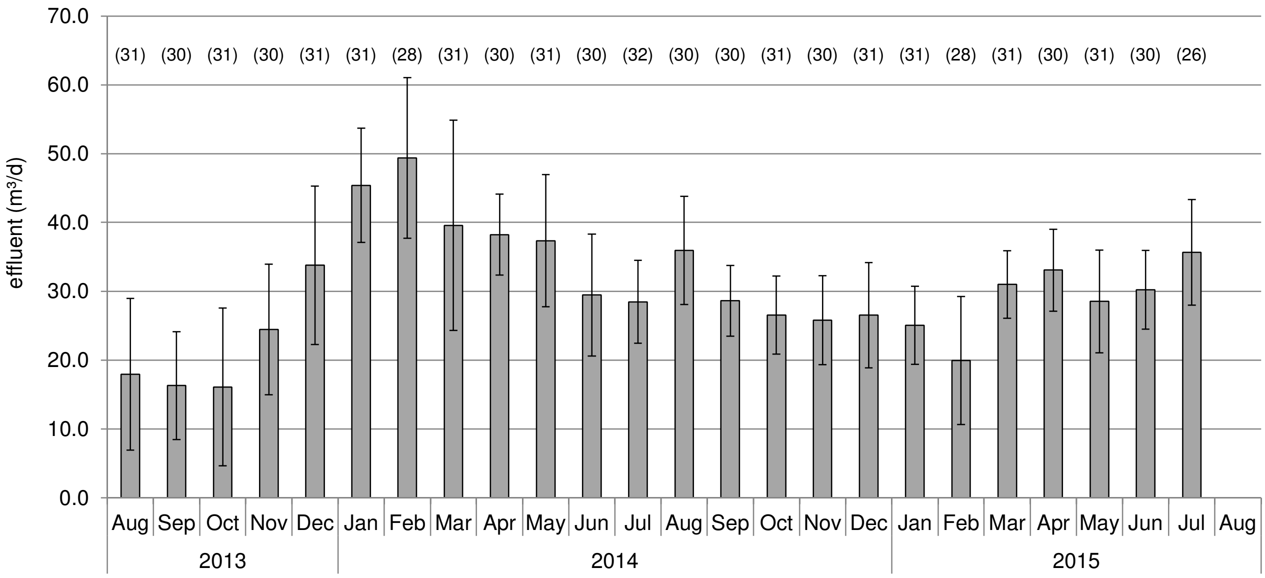

Figure 30 Treated water quantities in the effluent of the wastewater treatment plant; bars represent the standard deviation of the mean, the number of values is given in brackets

Starting from April 2014, the individual households were connected stepwise to the vacuum sewer system. Extensive repairs and improvements of the installations were carried out at the cluster units and at the communal washhouse in March 2014 and May 2015. Hence, the water use decreased from 37.8 (±8.0) m³/d (April and May 2014) to 28.8 (±7.8) m³/d (June 2014 to July 2015), despite the connection of the individual households starting from April 2014.

4.2.2 Water and air temperature

The daily average temperature of the water was 23.9°C (Figure 31). The lowest average monthly temperatures were measured from June to August. In these months, the average temperature of the water was 19.6°C (August 2013), 20.5°C (June and July 2014) and 20.4°C (June and July 2015). The highest average monthly temperatures were measured from November to March, with values up to 27.3°C in March 2015.

The average daily air temperature measured at the irrigation site was 23.8°C, with highest monthly averages from November to February (up to 27.6°C) and minima in June and July (down to 17.9°C). Air temperatures can reach up to 40°C during the hottest time of the year.

This is similar to the average temperatures reported for Ondangwa (a town located 130 km further east), where means are 17°C in June and July and up to 25°C in October, November, and December (Mendelsohn *et al.* 2000).

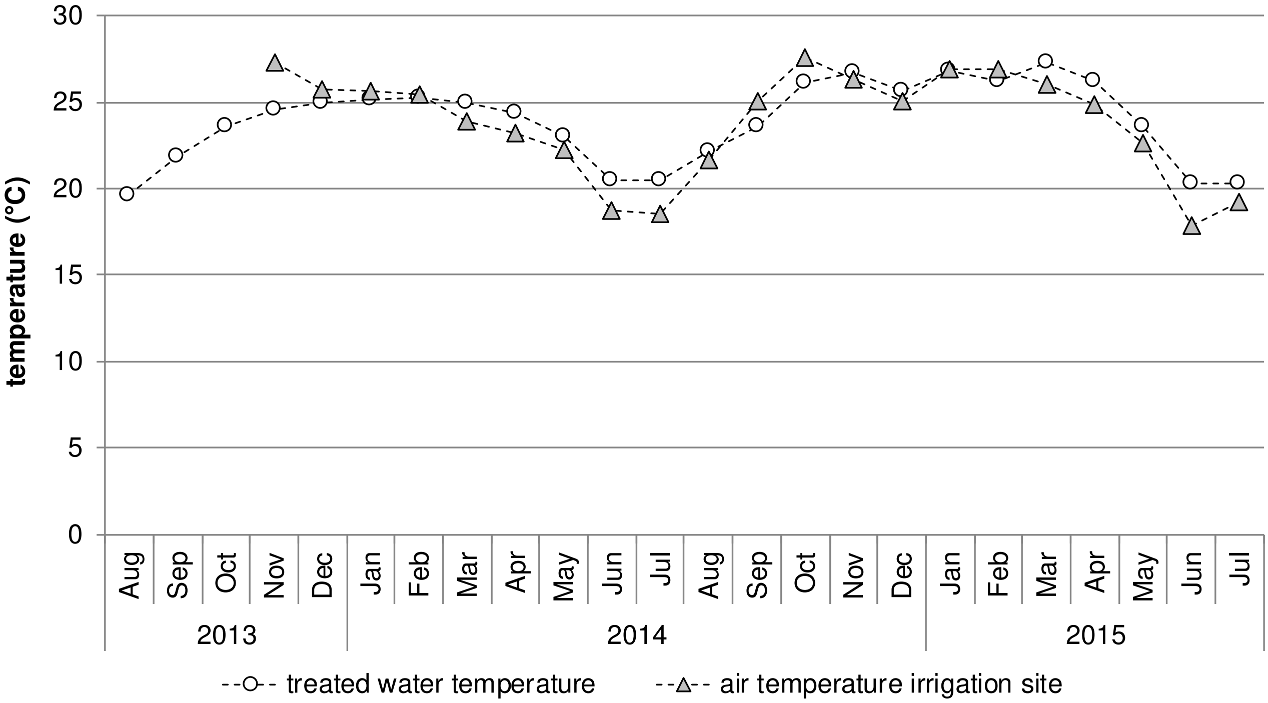

Figure 31 Average monthly temperature in the effluent of the wastewater treatment plant and average monthly air temperature measured at the irrigation site; bars represent the standard deviation of the mean, the number of values is given in brackets

In Outapi, tap water is provided by treating surface water from a nearby canal. This canal transports water from the Kunene River to the North of Namibia. Water is not heated at the sanitation facilities. Hence, the treated water temperature is almost the same as the air temperature. The standard deviation for the air temperature is slightly higher (±3.4°C) than the standard deviation of the water temperature (±2.5°C).

4.2.3 Electrical conductivity

During the monitoring period, the average EC was 612 µS/cm in the untreated wastewater. It increased to in average 689 µS/cm in the effluent of the UASB reactors. This increase can be explained by the increase of soluble substances during hydrolysis in the anaerobic reactors (Tchobanoglous *et al.* 2004). The EC decreased to 532 µS/cm in the effluent of the RBCs. This is almost the same as the value of 527 µS/cm measured in the effluent of the wastewater treatment plant (after microscreening and UV disinfection).

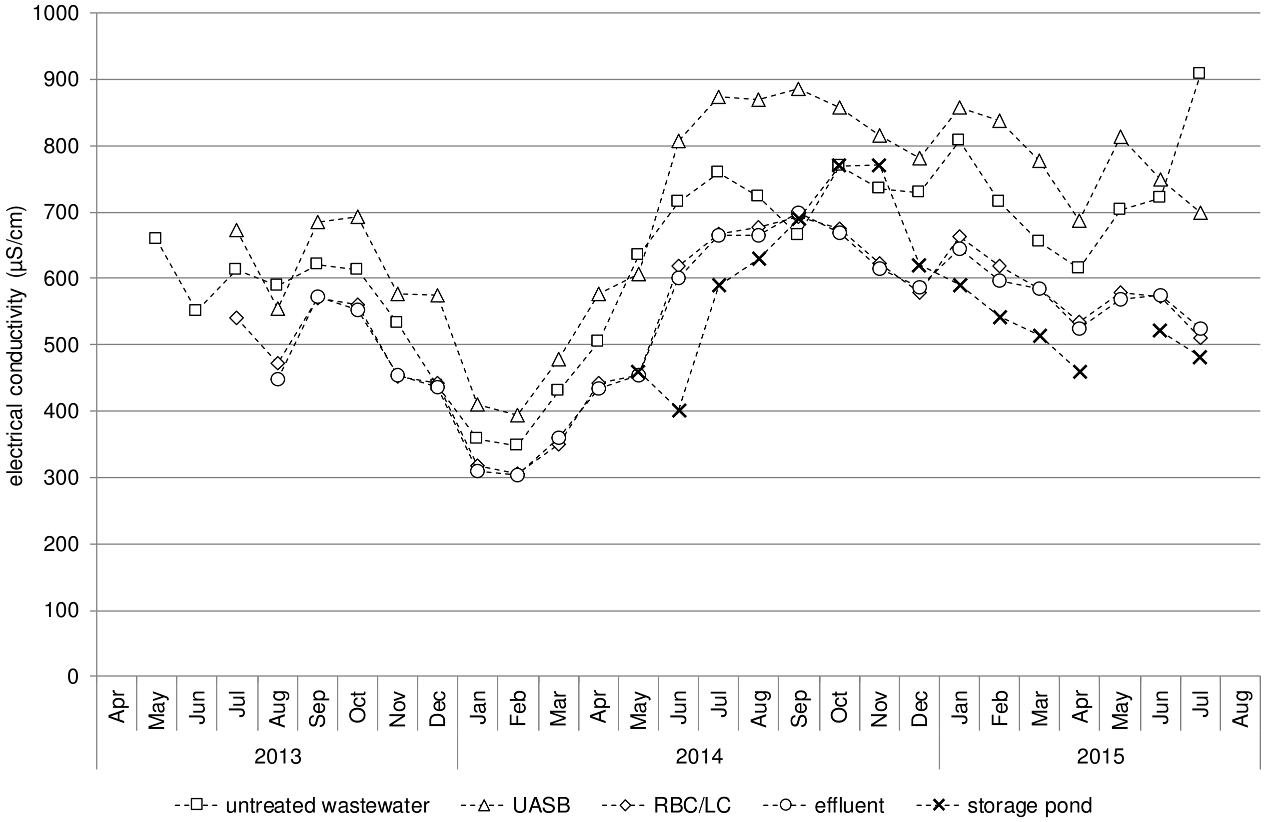

Figure 32 Electrical conductivity in the untreated wastewater, in the effluent of the UASB reactors, in the effluent of the RBCs and lamella clarifiers, in the effluent of the wastewater treatment plant, and in the storage pond

The decrease of 157 µS/cm (effluent UASB reactors minus effluent RBCs/lamella clarifiers) can be explained by the consumption of HCO_3^- and oxidation of NH_4^+ to NO_3^- during aerobic treatment. During nitrification, two moles of HCO_3^- are consumed per mole of oxidized NH_4^+ ($NH_4^+ + 2\ O_2 + 2\ HCO_3^- \rightarrow NO_3^- + 3\ H_2O + 2\ CO_2$). The monitored changes in ammonium and nitrate nitrogen concentrations were -26.4 and +22.9 mg/L, or 1.8×10^{-3} mol/L on average (Table 10). The calculated decrease of HCO_3^- was 3.3×10^{-3} mol/L. The equivalent conductivity of nitrate is slightly lower than the equivalent conductivity of ammonium but roughly compensates the change in electrical conductivity. The decrease in EC due to oxidization of ammonium nitrogen to nitrate nitrogen is only 22 µS/cm (= -139 µS/cm + 117 µS/cm). In contrast, consumption of HCO_3^- leads to a decreasing electrical conductivity of (calculated) 146 µS/cm. Hence the calculated conductivity decrease of 167 µS/cm fits to the measured decrease of 157 µS/cm.

Table 10 Measured ammonium nitrogen and nitrate nitrogen concentrations in the effluent of the UASB reactors and the RBCs/lamella clarifiers, calculated change in HCO_3^- concentrations, equivalent conductivities of the ions (Haynes *et al.* 2013) and calculated electrical conductivities

	monitored concentrations						**equivalent**	**change in electrical**
	UASB (effluent)		**RBC (effluent)**		**UASB-RBC**		**conductivity**	**conductivity**
	mg/L	mol/L	mg/L	mol/L	Δc (mg/L)	Δc (mol/L)	mS×m²×mol^{-1}	ΔEC (µS/cm)
NH_4^+-N	36.2	2.6E-03	9.8	7.0E-04	-26.4	-1.9E-03	7.35	-139
NO_3^--N	0.5	3.6E-05	23.4	1.7E-03	+22.9	+1.6E-03	7.142	+117
HCO_3^-	-	-	-	-	-	-3.3E-03	4.45	-146
total								**-167**

The EC of the tap water was low, with values between 46 and 66 µS/cm (Figure 33). The tap water is provided by NamWater. The treatment plant is located in Outapi and purifies canal water originating from the Namibian-Angolan border river Kunene.

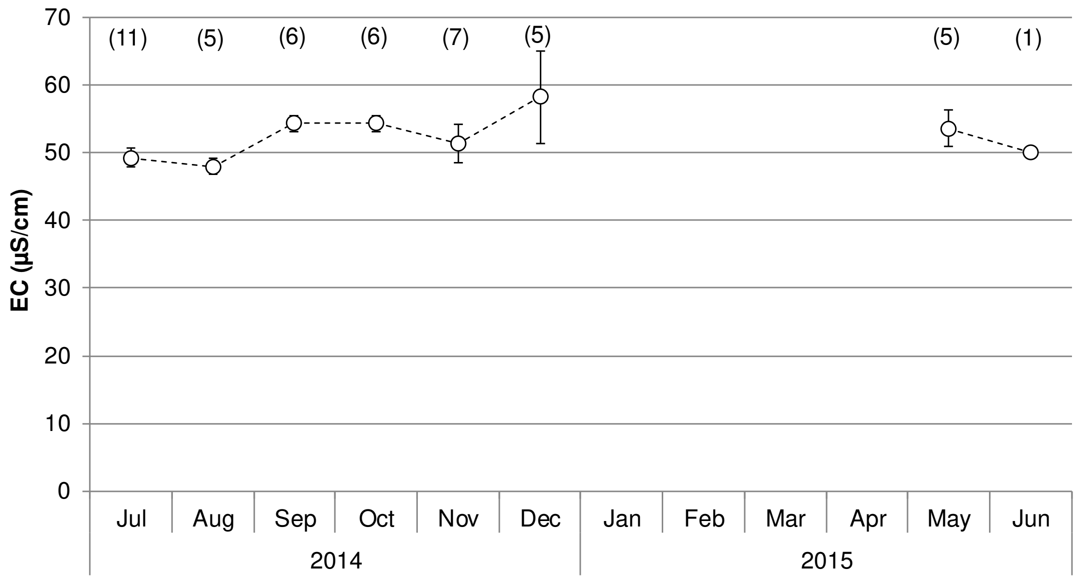

Figure 33 Electrical conductivity in the tap water; the bars represent the standard deviation of the mean, the number of measurements is given in brackets

4.2.4 Turbidity

The turbidity of the untreated wastewater was, on average, 507 NTU (nephelometric turbidity units) with a relatively high standard deviation of ±218 NTU (Figure 34). After sedimentation and anaerobic pretreatment in the UASB reactors, the average turbidity was 126 NTU with a standard deviation of ±47.1 NTU. Thus, in the UASB reactors, turbidity is reduced as particulate matter is removed and turbidity values are equalized. The turbidity values after the aerobic treatment and lamella clarifiers and after the microscreen are not addressed here. They are discussed in in detail in Section 4.5.2.2 (page 127*ff.*).

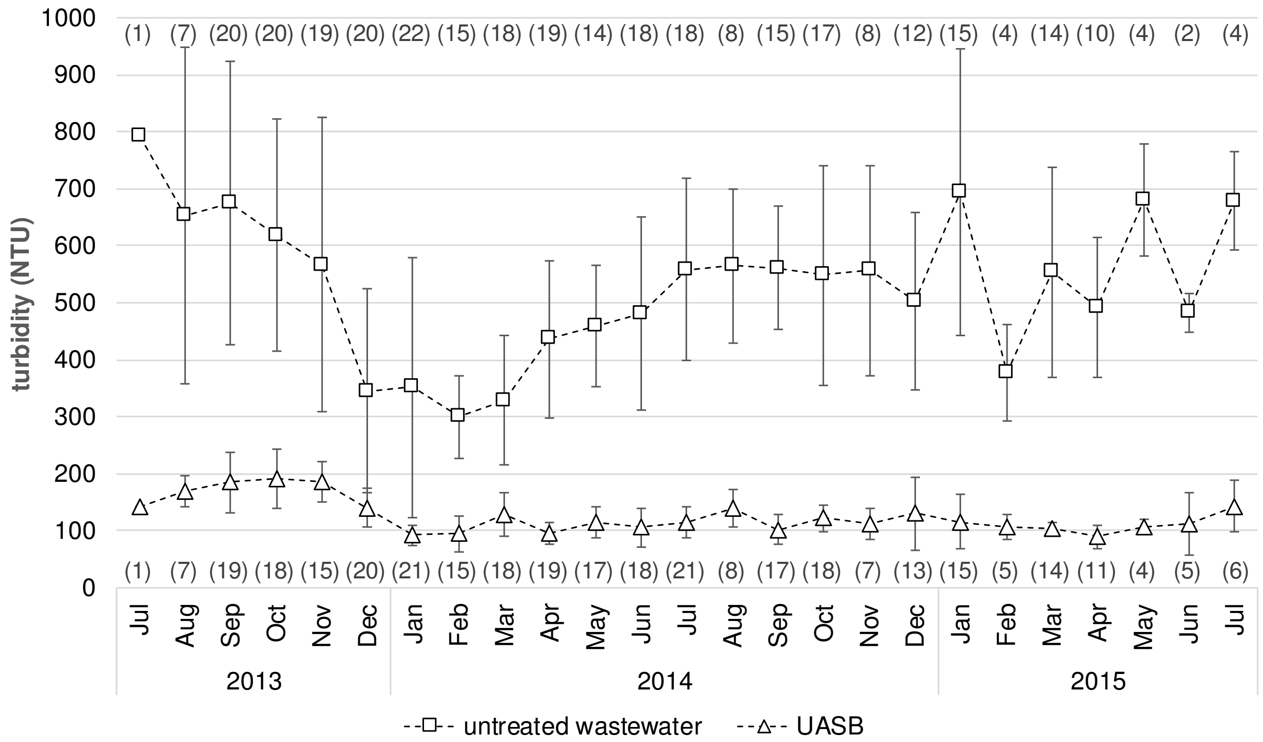

Figure 34 Turbidity in the untreated wastewater and in the effluent of the UASB reactors; bars represent the standard deviation of the mean, the number of values is shown in brackets (top: in untreated water, bottom: in the effluent of the UASB reactors)

4.2.5 Chemical oxygen demand

The TCOD in the untreated wastewater was, on average, 738 mg/L. Until April 2014, it was 557 mg/L and increased to, on average, 964 mg/L between May 2014 and June 2015 (+73%, Figure 35). The start-up of the cluster units in November 2013 did not influence the TCOD concentrations. However, the gradual connection of the individual households to the vacuum sewer system and the repairs carried out at the communal washhouse and the cluster units led to higher concentrations after April 2014.

The mean DCOD was 166 mg/L. This value remained relatively stable throughout the monitoring period. Until April 2014, it was, on average, 159 mg/L and increased to a mean value of 180 mg/l from May 2014 to June 2015 (+13%). The increase of the TCOD is mainly attributed to an increase in PCOD. Over the whole monitoring period, the PCOD was about 572 mg/L. Until April 2014, it was 398 mg/L and increased to 784 mg/L (+97%, May 2014 to July 2015).

The changes in concentrations are different for the effluent of the UASB reactors (Figure 36). The TCOD and DCOD concentrations were 369 mg/L and 197 mg/L until December 2013. They subsequently decreased to, on average, 168 mg/L and 75.3 mg/L (December 2013 to July 2015). In November 2013, the UASB reactors were taken into full operation. Before this date, they were only operated as a sedimentation unit. From November 2013, their top lids were closed and operation under the absence of atmospheric oxygen was initiated.

Hence, the decrease in concentrations after this date is attributed to the start of anaerobic digestion processes in the reactors. The PCOD also decreased from 172 to 92.7 mg/L. It constituted, on average, 52% of the TCOD concentrations. The percentage was 47% until December 2013 and then 55%.

In the effluent of the RBC and lamella clarifiers, the concentrations were, on average, 55.7 mg/L for TCOD, 36.2 mg/L for DCOD and 19.5 mg/L for PCOD (Figure 37). In the effluent of the wastewater treatment plant, the concentrations were 57.7 mg/L for TCOD, 40.8 mg/L for DCOD and 17.0 for PCOD (Figure 38). Thus, the concentrations increased slightly for TCOD and DCOD (+3.7% for TCOD, +12.6% for DCOD) and decreased slightly for PCOD (-12.9%).

However, the microscreen should lead to lower concentrations as particles are removed. Especially during the first months of operation (May 2013 to October 2013), the water quality deteriorated because dust and insects could enter the container of the microscreen and the

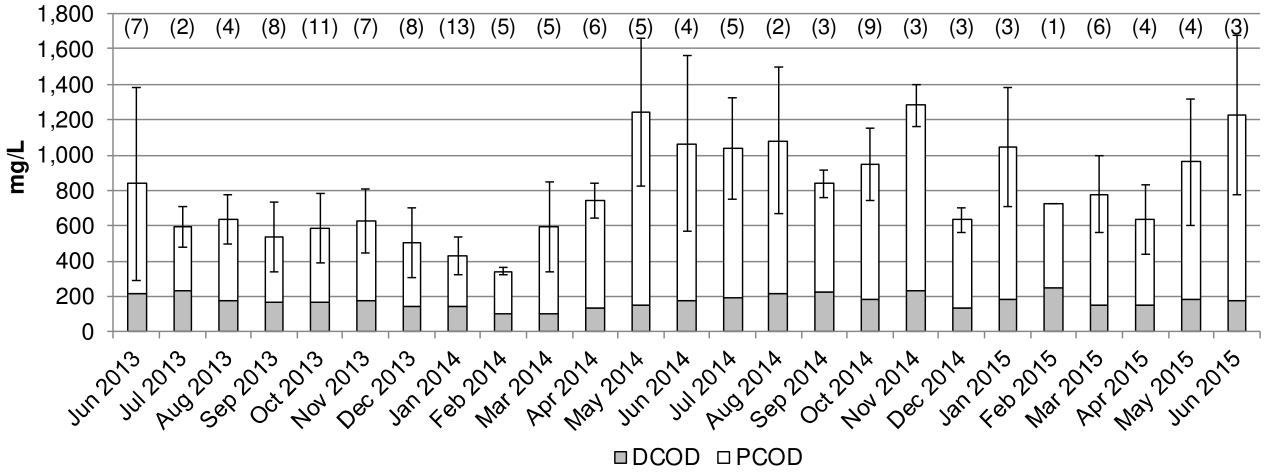

Figure 35 TCOD and fractions of DCOD and PCOD in the untreated wastewater; bars represent the standard deviation of the mean TCOD, the number of TCOD values is shown in brackets

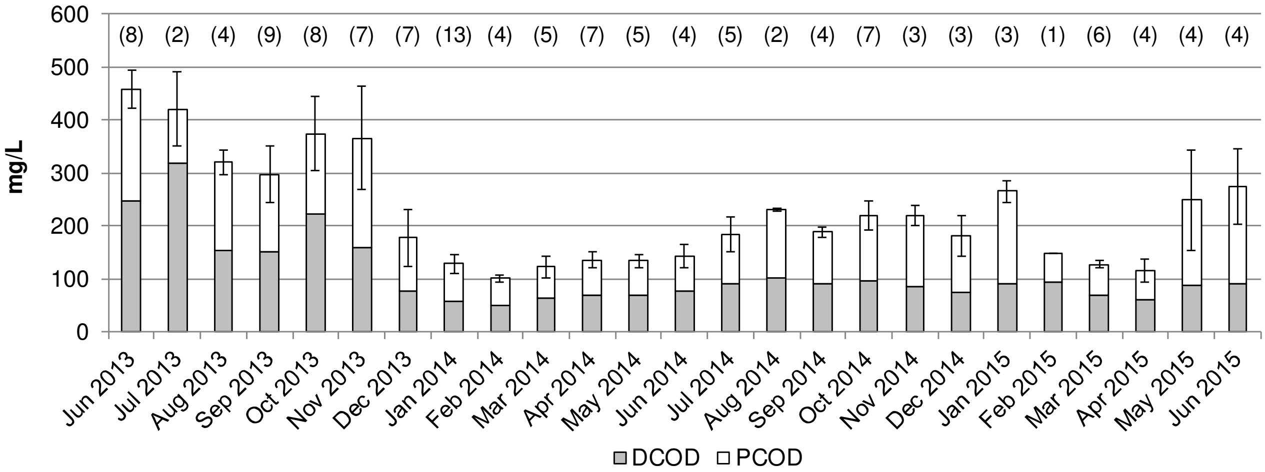

Figure 36 TCOD and fractions of DCOD and PCOD in the effluent of the UASB reactors; bars represent the standard deviation of the mean TCOD, the number of TCOD values is shown in brackets

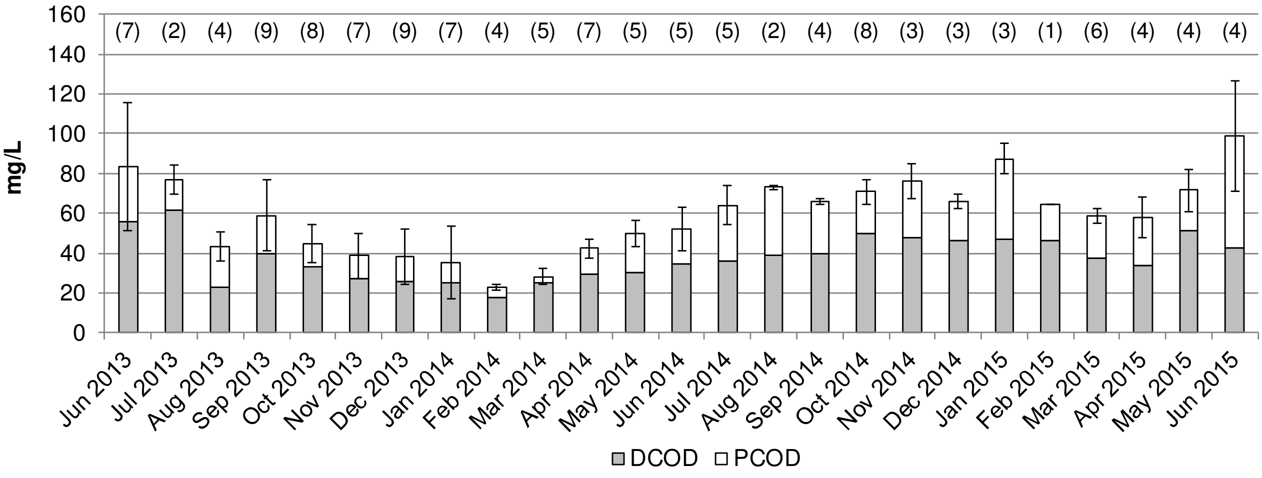

Figure 37 TCOD and fractions of DCOD and PCOD in the effluent of the RBCs and lamella clarifiers; bars represent the standard deviation of the mean TCOD, the number of TCOD values is shown in brackets

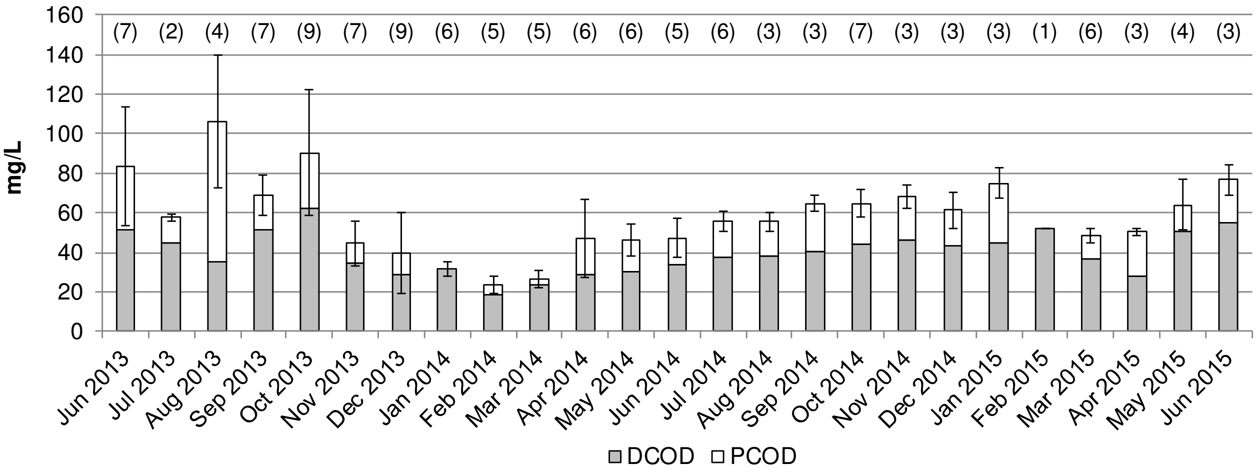

Figure 38 TCOD and fractions of DCOD and PCOD in the effluent of the wastewater treatment plant; bars represent the standard deviation of the mean TCOD, the number of TCOD values is shown in brackets

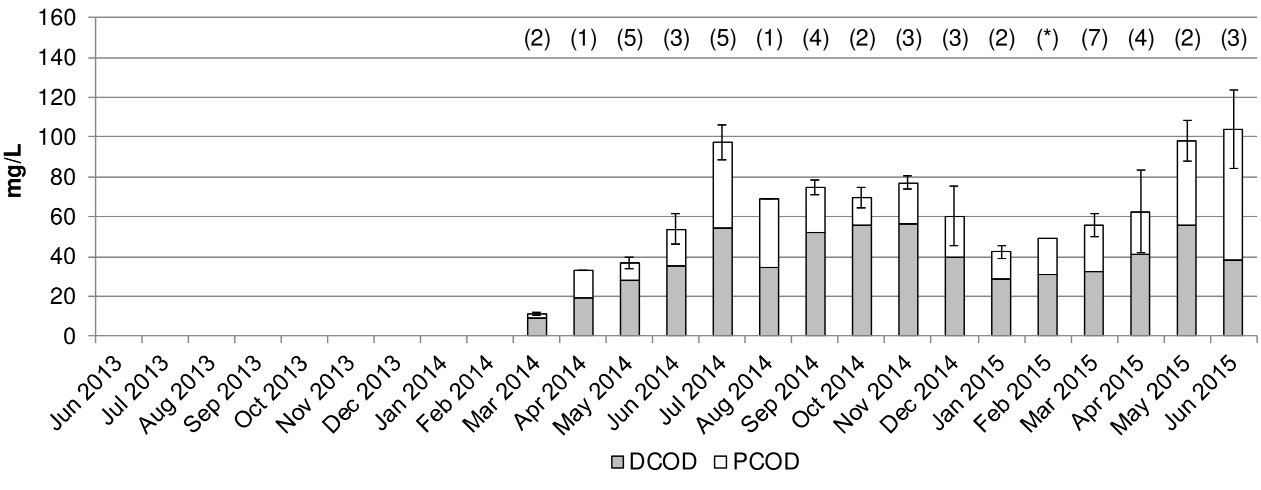

Figure 39 TCOD and fractions of DCOD and PCOD in the stored water; bars represent the standard deviation of the mean TCOD, the number of TCOD values is shown in brackets, an asterisk marks estimated data because measurements were not available for this month

container for storage of the treated water. The microscreen did not compensate this additional input. During this time period, the concentrations increased from, on average, 60.0 mg/L (TCOD), 41.5 mg/L (DCOD) and 18.5 mg/L (PCOD) in the effluent of the RBCs/LCs to 84.6 mg/L (TCOD), 56.4 mg/L (DCOD) and 28.2 mg/L (PCOD) in the effluent of the plant (TCOD: +41%, DCOD: +36%, PCOD: +52%).

During the project period, various operation modes of the RBCs were tested. During start up, the wastewater was treated with only one RBC until August 2013. From September 2013 to May 2014, it was treated with both RBCs. Because water quantities remained lower than planned, one of the RBCs was taken out of operation in June 2014. From October 2015 to April 2015, intermittent operation during the night was carried out as a measure for energetic optimization (see also Section 4.7.2.2, page 172).

After installation of additional covering plates at the microscreen and the container for storage of the treated water, and instruction of the operators, deterioration of the water quality during storage could be mitigated. Also, water quantities increased and throughput was higher. The average concentrations after November 2013 were 54.3 mg/L (TCOD) and 34.5 mg/L (DCOD) in the effluent of the RBCs/LCs to 49.2 mg/L (TCOD), and 35.1 mg/L (DCOD) in the effluent of the plant (TCOD: -9.4%, DCOD: +1.7%).

The storage pond became operational in March/April 2014. The average TCOD and DCOD concentrations were 64.9 mg/L and 39.6 mg/L, respectively (Figure 39). Thus, the PCOD was about 39%.

4.2.6 Biochemical oxygen demand

The BOD_5 measurements were carried out by an external laboratory in Windhoek. Hence, samples were collected less often and less values are available. All BOD_5 data and values for the TCOD/BOD_5 ratio are presented in Figure 40 and Figure 41.

The average BOD_5 in the untreated wastewater was 192 mg/L but varied considerably (standard deviation = 152 mg/L, n = 10, Figure 40). It went down to 53.3 mg/L in the effluent of the UASB reactors and decreased further to 8.6 mg/L in the effluent of the RBCs and lamella clarifiers. The average BOD_5 in the effluent of the wastewater treatment plant was 5.5 mg/L. Thus, the BOD_5 concentration decreased by 97% during wastewater treatment. During storage, it increased to 16.0 mg/L.

The BOD_5:TCOD ratio in the untreated wastewater was, on average, 0.3 (Figure 41). This is relatively low but still within the typical range of 0.3 to 0.8 (Tchobanoglous *et al.* 2004). However, the ratios spread over a large range of values, from < 0.1 to more than 0.8. After anaerobic pretreatment, the average BOD_5:TCOD ratio was 0.3. After aerobic treatment it was 0.1, on average, which is also a typical value (Tchobanoglous *et al.* 2004). During storage, the ratio increased to 0.3. More details on the BOD_5:COD ratio and its significance for the quality of the reclaimed water are given in Chapter 4.5.2.4 (page 131).

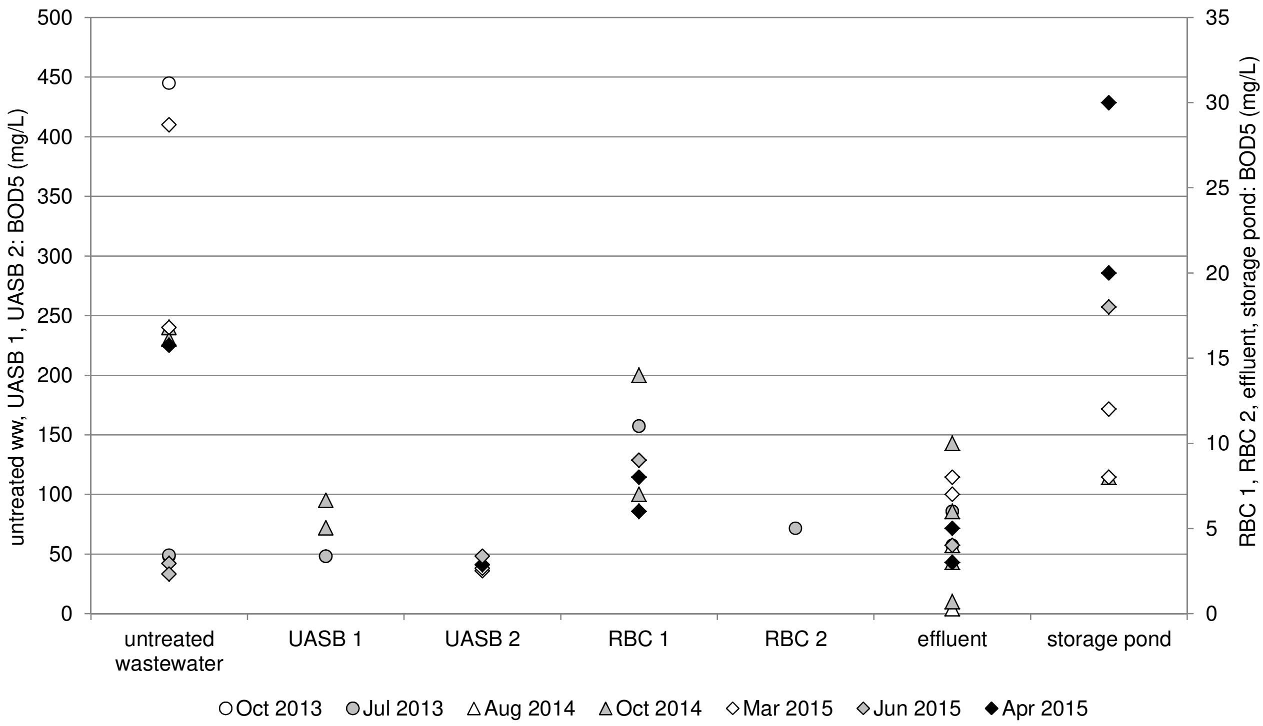

Figure 40 Overview on BOD_5 measurements in the untreated wastewater, in the effluent of the UASB reactors, in the effluent of the RBC/LC 1, in the final effluent, and in the storage pond; ww = wastewater

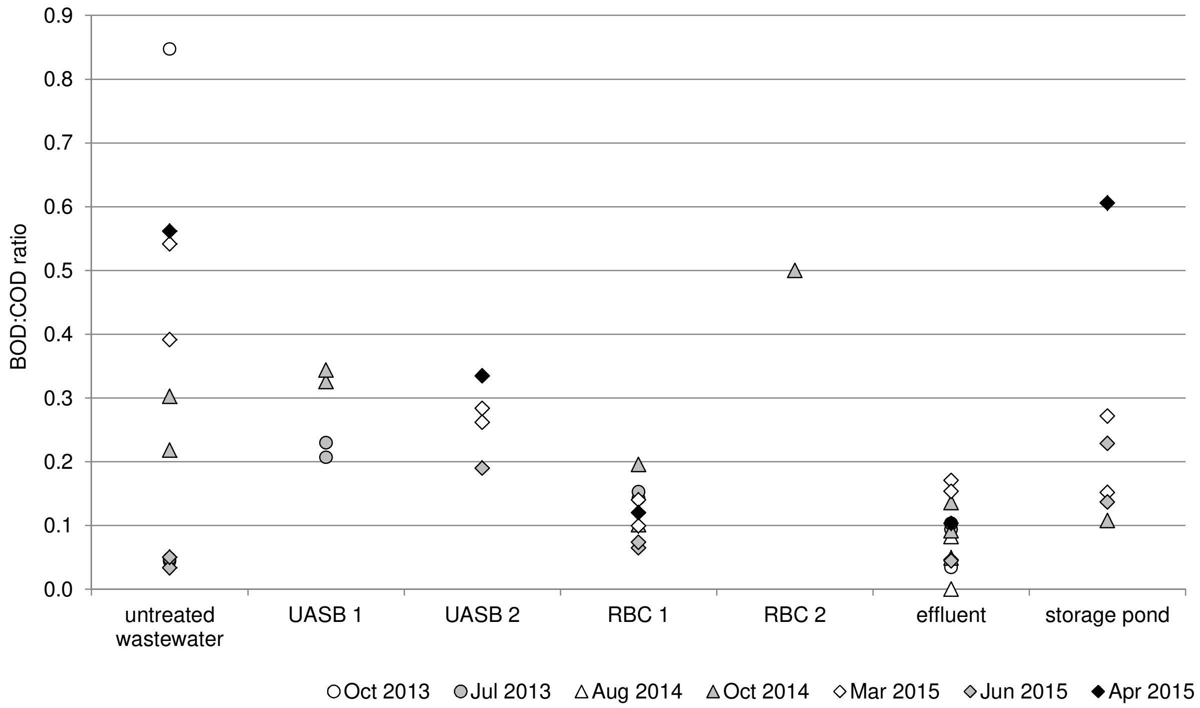

Figure 41 BOD:COD ratio of the untreated wastewater, in the effluent of the UASB reactors, in the effluent of the RBC/LC 1, in the final effluent, and in the storage pond; ww = wastewater

4.2.7 Nitrogen concentrations

The TN in the untreated wastewater consisted of, on average, 51% ammonium nitrogen, 48% organic nitrogen and 1% nitrate nitrogen. These percentages remained more or less constant during the monitoring period (Figure 42). They are in compliance with typical values in raw domestic sewage (Sperling 2007c). Until February 2014, the TN concentration in the untreated wastewater remained relatively low, at 38.8 mg/L. It increased from May 2014. The mean concentration between May 2014 and June 2015 was 79.5 mg/L.

This change in concentrations also coincides with the connection of the individual households to the vacuum sewer system that started in April 2014. In addition, maintenance measures at the communal washhouse and the cluster units (mostly repairs of leaking toilets and taps) reduced tap water use and, thus increased concentrations in the wastewater. The start of operation of the cluster units in November 2013 did not affect the concentrations of TN and the nitrogen forms.

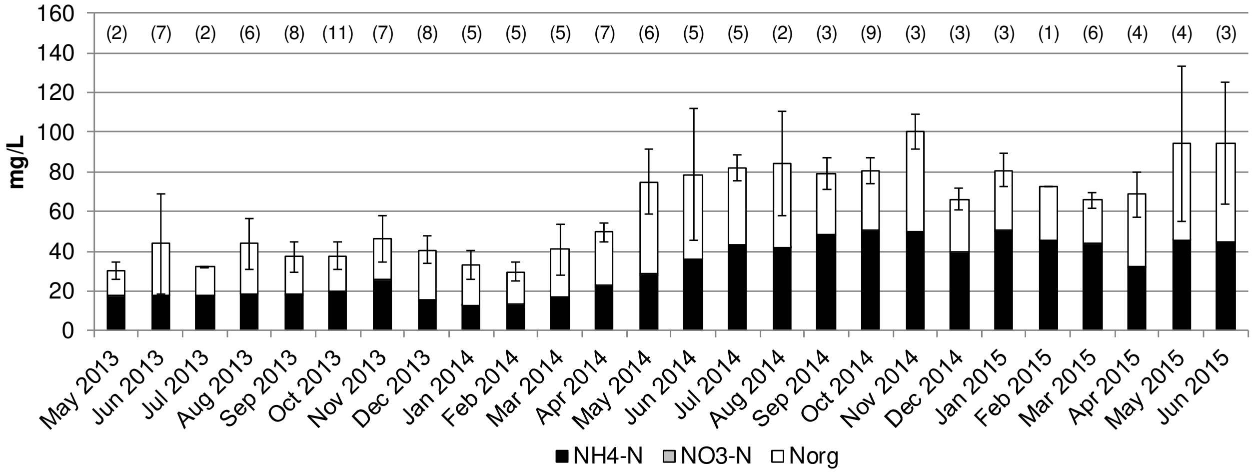

Figure 42 Ammonium nitrogen, nitrate nitrogen and organic nitrogen in the untreated wastewater; bars represent the standard deviation of the mean TN; the number of TN values is shown in brackets

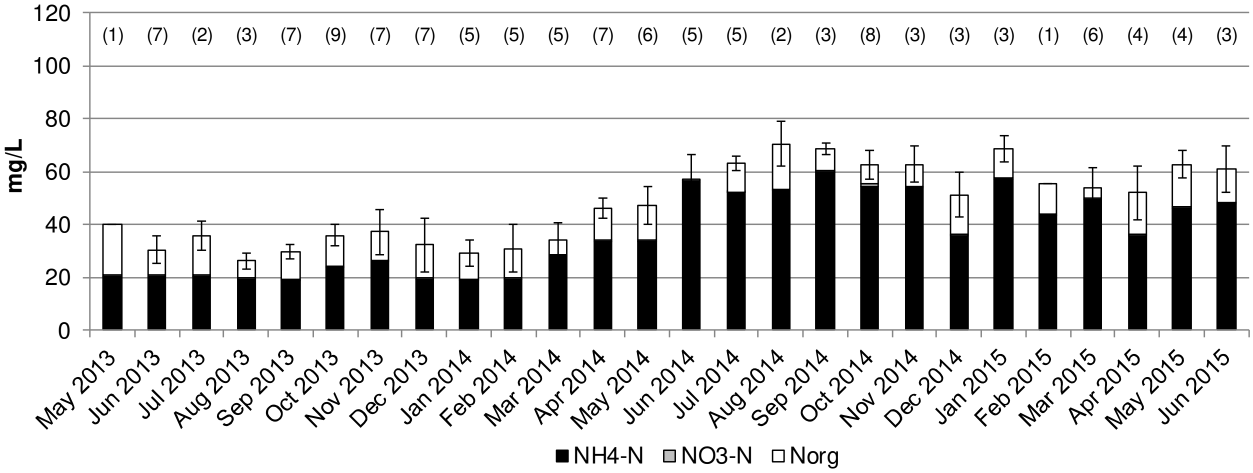

Figure 43 Ammonium nitrogen, nitrate nitrogen and organic nitrogen in the effluent of the UASB reactors; bars represent the standard deviation of the mean TN; the number of TN values is shown in brackets

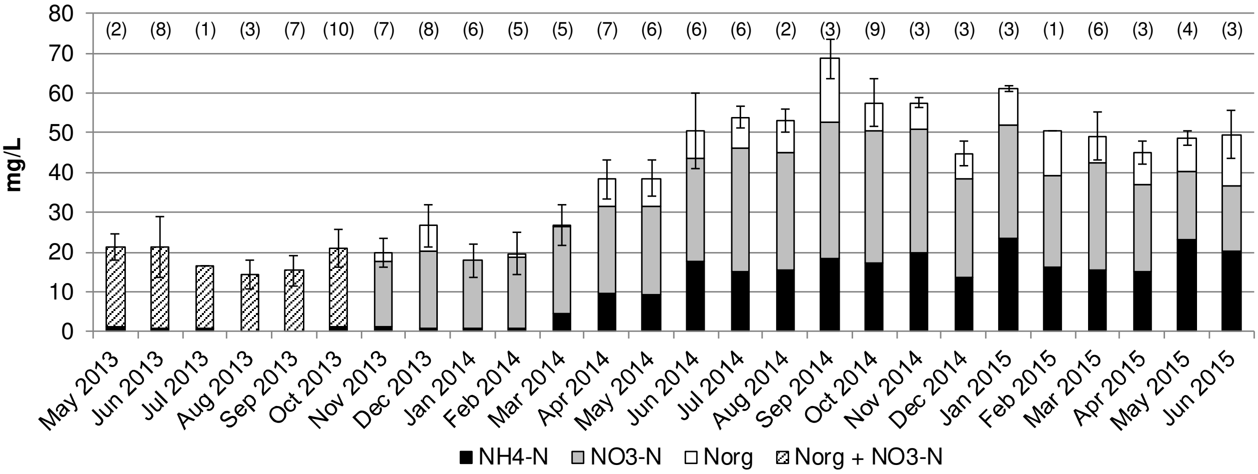

Figure 44 Ammonium nitrogen, nitrate nitrogen and organic nitrogen in the effluent of the RBCs and lamella clarifiers; bars represent the standard deviation of the mean TN; the number of TN values is shown in brackets

Following combined sedimentation/anaerobic pretreatment in the UASB reactors, the TN concentration in the water decreased to 45.6 mg/L in the effluent of the UASB reactors. Organic nitrogen concentrations decreased to 8.9 mg/L in the effluent. The ammonium nitrogen concentration increased to 36.2 mg/L. Thus, ammonium nitrogen represents, on average, 79% of the total nitrogen in the effluent of the UASB reactors. Degradation of organic nitrogen and production of ammonium nitrogen are part of the anaerobic digestion processes in the UASB reactors (Chernicharo 2007).

During aerobic treatment, ammonium nitrogen is oxidized to nitrate nitrogen. Throughout the entire monitoring period, the effluent of the RBCs and lamella clarifiers consisted of, on average, 9.8 mg/L ammonium nitrogen (27%), 23.4 mg/L nitrate nitrogen (65%) and 2.5 mg/L (7%) organic nitrogen. Nitrite was measured from November 2013 to July 2013 and was 0.2 mg/L, on average, (n = 19). Because nitrite nitrogen represents only a very small fraction of the total nitrogen (0.5%), it is not included in Figure 44. Between May 2013 and October 2013, only total nitrogen and ammonium nitrogen were measured.

Although the scales in Figure 43 and in Figure 44 differ, it is clear that TN concentrations are lower in the effluent of the RBCs and lamella clarifiers, compared to the effluent of the UASB reactors. Because no denitrification step is projected, the TN concentrations should decrease only to a minor degree, due to incorporation of nitrogen into cell biomass and removal of particulate nitrogen. This difference in TN concentrations was very high from May 2013 to November 2013, at, on average, 14.5 mg/L or 43% of the UASB effluent concentration. It decreased to, on average, 8.0 mg/L or 16% of the UASB effluent concentration from January 2014 to July 2015. The decrease in TN concentrations is attributed to unintended denitrification in the first months of operation, due to sludge bulking in the lamella clarifiers. When water quantities increased (due to the start-up of the cluster units in November 2013) denitrification rates declined (December 2013 to January 2014).

The ratios of the nitrogen compounds changed during the monitoring period. From November 2013 to February 2014, nitrate nitrogen was the dominant nitrogen form in the effluent of the RBCs and lamella clarifiers (71% to 91% or 16.5 to 21.9 mg/L of the total nitrogen), whereas ammonium nitrogen was only 1.0 mg/L, on average. Starting from March 2014, total nitrogen concentrations and the percentage of ammonium nitrogen increased. From May 2014 to June 2015, the average nitrate nitrogen concentration was 26.3 mg/L (50%), the average ammonium nitrogen concentration was 16.7 mg/L (33%), and the average organic nitrogen concentration was 8.6 mg/L (17%).

These changes had no or only minor effects on nitrogen compounds in the water. The higher ammonium nitrogen concentrations starting from June 2014 may have been due to operation with only one RBC.

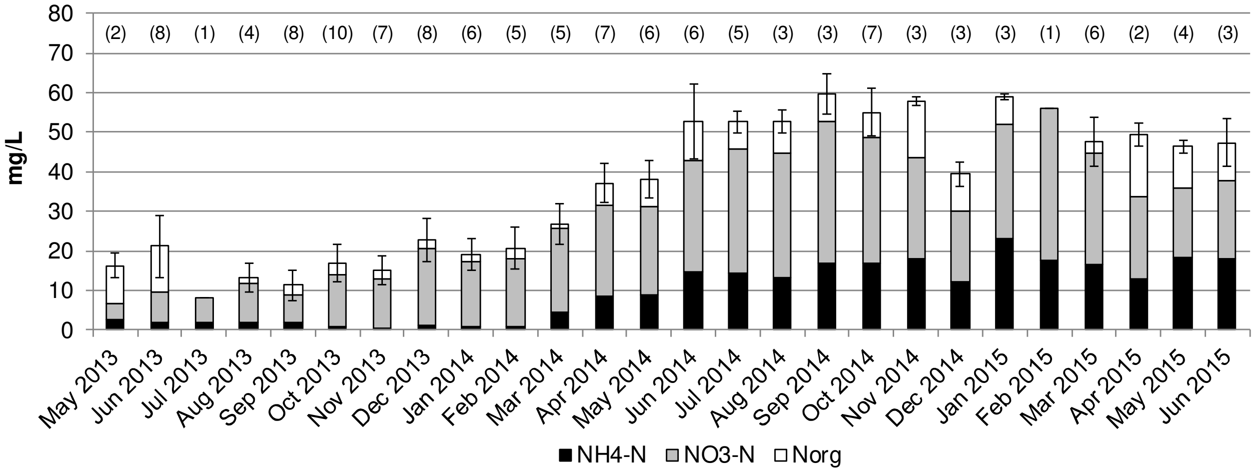

Figure 45 Organic nitrogen, nitrate nitrogen and ammonium nitrogen in the effluent of the wastewater treatment plant; bars represent the standard deviation of the mean TN; the number of TN values is shown in brackets

After the aerobic treatment and secondary clarification, the water passes the microscreen and UV disinfection. Thus, nitrogen concentrations and nitrogen forms are changed only to a minor degree (Figure 45).

The average TN concentration in the storage pond was 32.6 mg/L. However, concentrations were higher from May 2014 to November 2014 (average: TN = 42.2 mg/L, NH_4^+-N = 1.5 mg/L, NO_3^--N = 30.5 mg/L, organic N = 10.2 mg/L) and then decreased, starting from December 2014 (average December 2014 to June 2015: TN = 23.6 mg/L, NH_4^+-N = 1.1 mg/L, NO_3^--N = 18.1 mg/L, organic N = 4.4 mg/L).

In the storage pond, the TN concentrations decreased from November 2014 but remained approximately constant in the effluent of the wastewater treatment plant (Figure 45 and Figure 46). The decrease in TN concentrations coincides with the start of the rainy season (Figure 68, page 141) and the takeover of the irrigation site by a new farmer (Zimmermann *et al.* 2017b). Dilution of the upper layers of the pond water with rainwater and incomplete mixing with the more concentrated water underneath probably led to bias in the samples. In December 2014 and January 2015, precipitation was 119 mm and 81 mm, respectively (Figure 68, page 141).

The amount of rainwater collected in the pond is then 371 m³ (= 0.2 m × 1,855 m²; see Section 4.1.6, page 66 for the surface area of the pond). This is about 10% of the total volume of the pond (3712 m³ ÷ 371 m³ = 0.10) or 23% of the water collected in the pond during that time (25.7 m³/d × 62 d = 1,593 m³ and 371 m³ ÷ 1,593 m³ = 0.23). The rainwater could therefore be the reason for the lower concentrations in the samples taken from the storage pond.

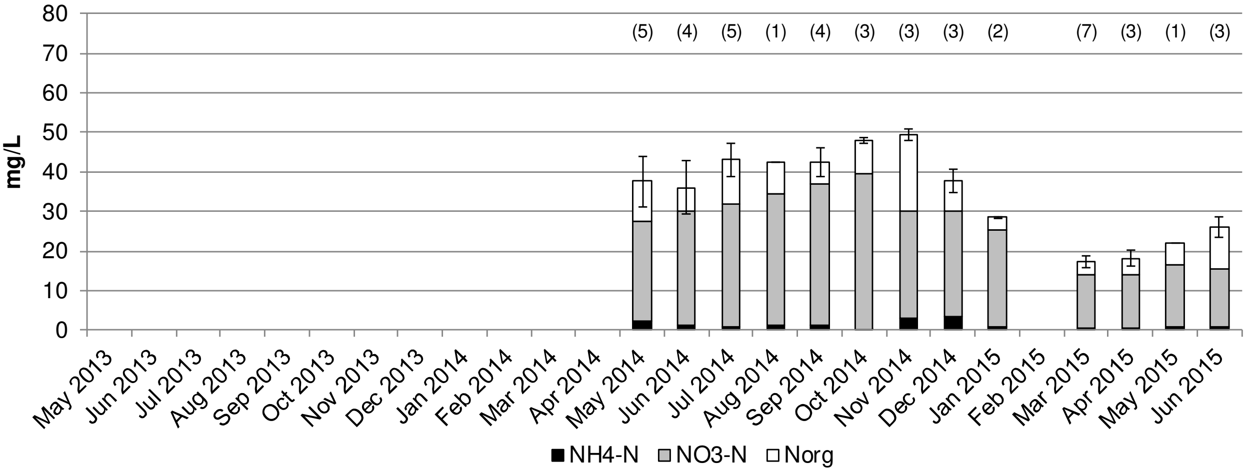

Figure 46 Ammonium nitrogen, nitrate nitrogen and organic nitrogen in the storage pond; bars represent the standard deviation of the mean TN; the number of TN values is shown in brackets

4.2.8 Phosphorus concentrations

An average of 10.3 mg/L TP was measured in the untreated wastewater. 6.4 mg/L or 62% was present in the form of orthophosphate P (Figure 47). The TP concentrations increased during the monitoring period. They were 8.2 mg/L until April 2014 and 12.7 mg/L from May 2014 to June 2015. The PO_4^{3-}-P concentrations were 5.2 mg/L and 7.7 mg/L, respectively (63% and 61% of TP). The increase in TP concentrations coincides with the connection of the individual households to the vacuum sewer system. The connection of the cluster units did not affect the TP and PO_4^{3-}-P concentrations.

After sedimentation and anaerobic pretreatment in the UASB reactors, the TP concentrations and PO_4^{3-}-P concentrations decreased slightly to 9.0 mg/L and 7.8 mg/L, respectively. Considerably less polyphosphate and organic P was contained in the effluent of the UASB reactors, compared to the untreated wastewater (Figure 48). As for the untreated wastewater, the concentrations in the effluent of the UASB reactors also increased after May 2014.

The TP and PO_4^{3-}-P concentrations were further reduced to 8.4 and 7.9 mg/L in the effluent of the RBCs and lamella clarifiers (Figure 49). The TP and PO_4^{3-}-P concentrations were almost identical in the effluent of the wastewater treatment plant (8.3 and 7.8 mg/L, Figure 50).

They increased slightly in the storage pond (9.9 mg/L and 8.8 mg/L, Figure 51). The concentrations increased until October and November 2014 and then decreased from December 2014 to April 2015, probably due to dilution with rainwater, as discussed in the previous section (Section 4.2.7).

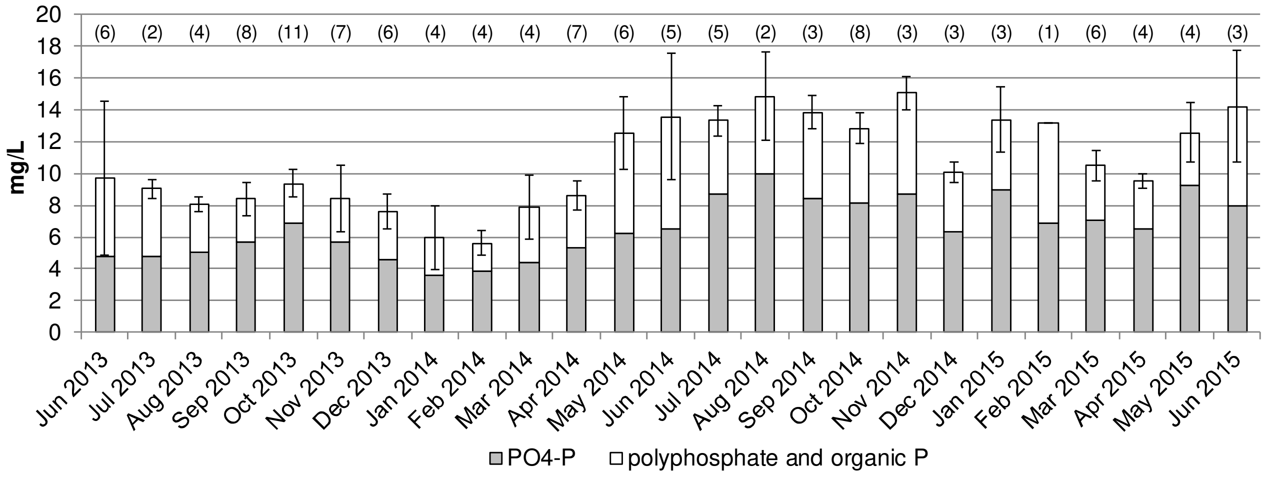

Figure 47 Orthophosphate, polyphosphate and organic phosphorus concentrations in the untreated wastewater; bars represent the standard deviation of the mean TP; the number of TP values is shown in brackets

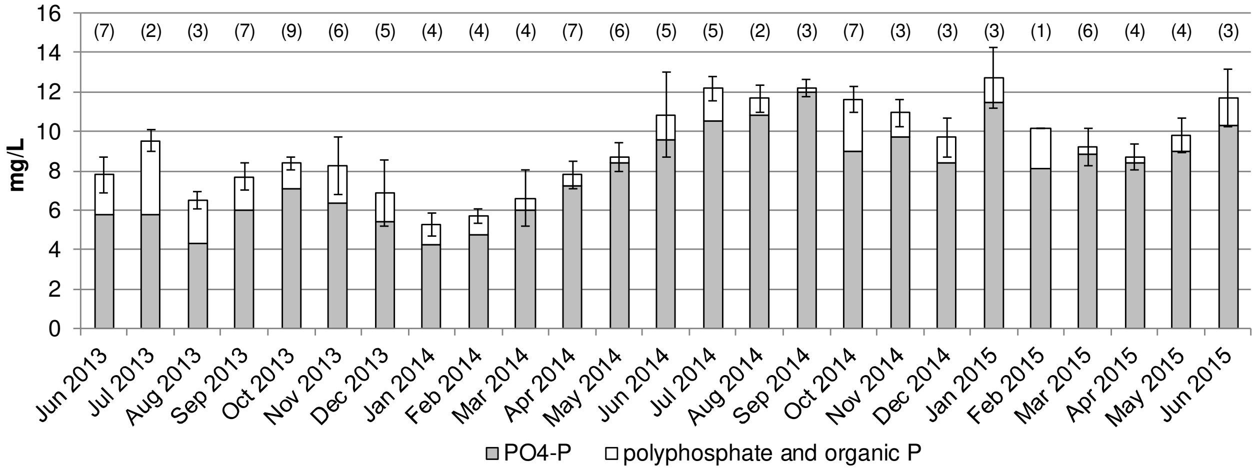

Figure 48 Orthophosphate, polyphosphate and organic phosphorus concentrations in the effluent of the UASB reactors; bars represent the standard deviation of the mean TP; the number of TP values is shown in brackets

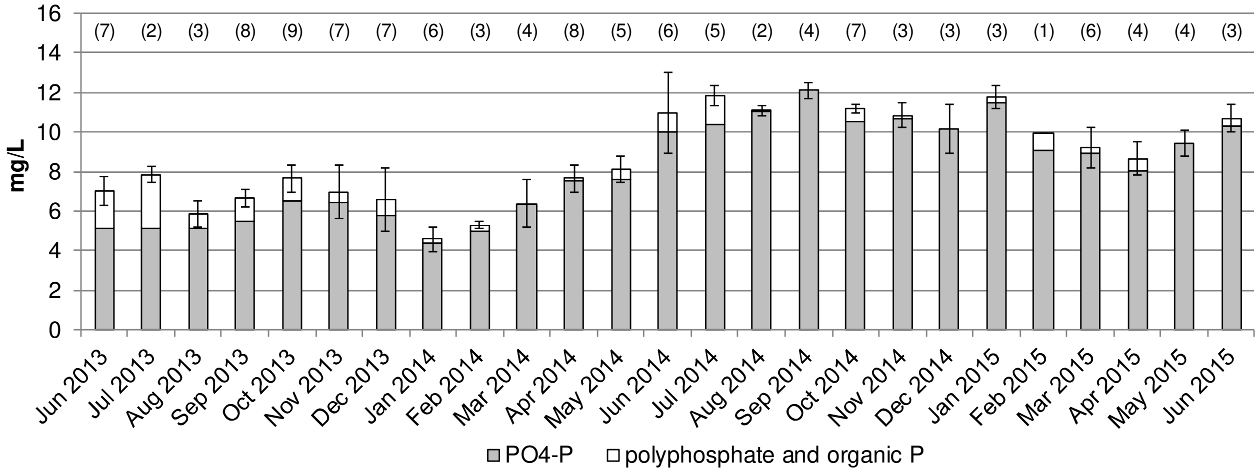

Figure 49 Orthophosphate, polyphosphate and organic phosphorus concentrations in the effluent of the RBCs and lamella clarifiers; bars represent the standard deviation of the mean TP; the number of TP values is shown in brackets

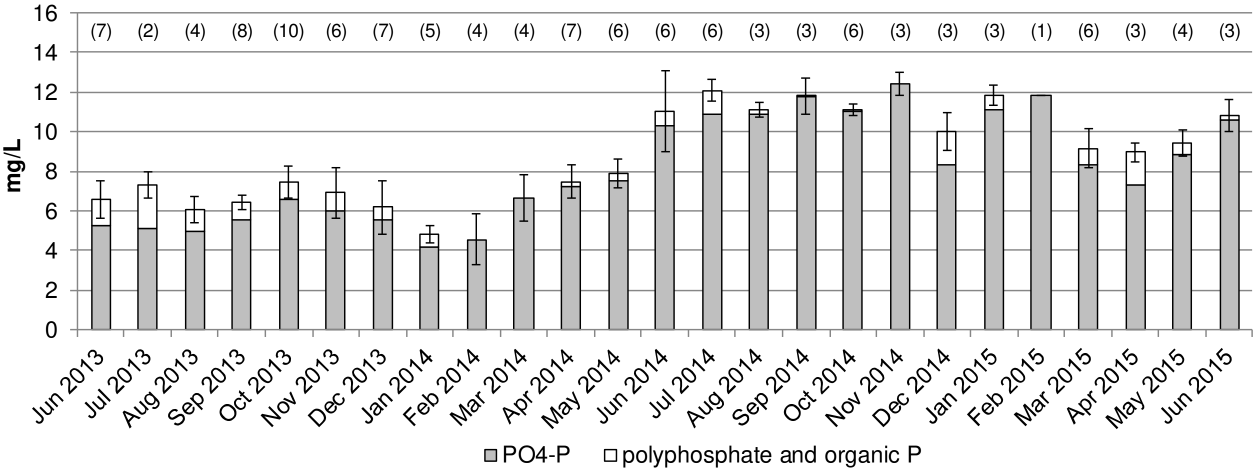

Figure 50 Orthophosphate, polyphosphate and organic phosphorus concentrations in the effluent of the wastewater treatment plant; bars represent the standard deviation of the mean TP; the number of TP values is shown in brackets

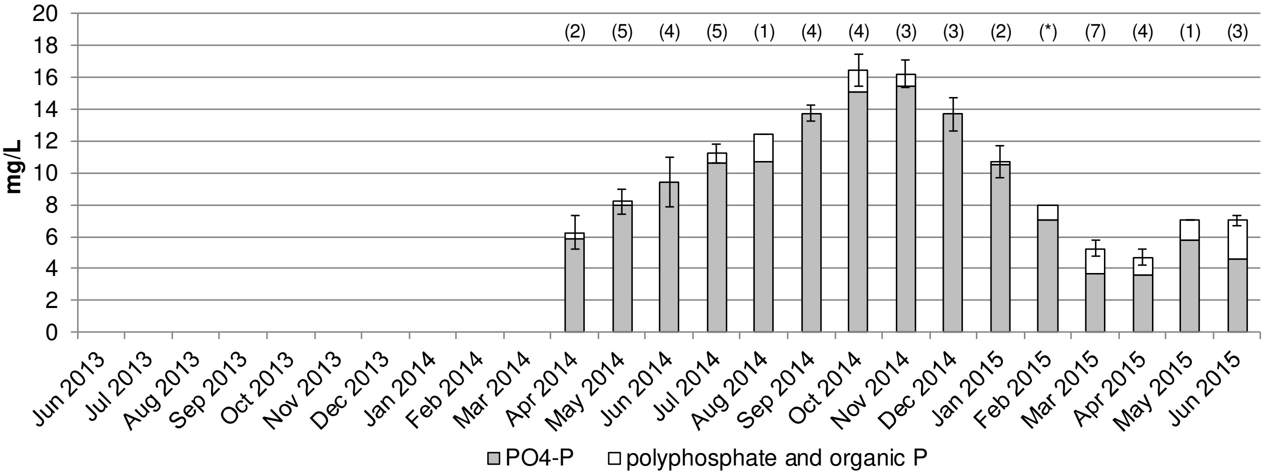

Figure 51 Orthophosphate, polyphosphate and organic phosphorus concentrations in the effluent of the storage pond; bars represent the standard deviation of the mean TP; the number of TP samples is shown in brackets; an asterisk marks estimated data because measurements were not available for this month

4.2.9 Microbial parameters

The UV disinfection device was installed in December 2013. Retrofitting was required in April and May 2014, which led to higher values of indicator organisms in the effluent during this time period (Figure 52 to Figure 55). During this interval, the water was supposed to be discharged into the existing sewers of Outapi and not reused for irrigation. However, the increasing levels of indicator organisms in April and May 2014 suggest that some of the water was discharged into the pond and not to the sewers.

Total coliforms in the untreated water had an average value of 6.3×10^7 MPN/100 mL (Figure 52). They decreased by 0.6 to 0.9 $\log_{10}$ units in the UASB reactors. The average concentrations were 1.7×10^7 MPN/100 mL in UASB 1 and 8.1×10^6 MPN/100 mL in UASB 2. The average values in the effluent of the RBCs and lamella clarifiers were 7.5×10^4 MPN/100 mL and

$7.8×10^4$ MPN/100 mL, respectively. Thus, total coliforms are further reduced by 2.2 log_{10} units during aerobic treatment and secondary clarification.

From August 2013 to November 2013, the average value for total coliforms in the effluent of the wastewater treatment plant was $2.6×10^5$ MPN/100 mL. Disregarding December 2013 and April and May 2014 (when installation and retrofitting of the UV disinfection were carried out), the average value of total coliforms in the effluent was $5.1×10^2$ MPN/100 mL. Because the average value in the effluent of RBC/LC 1 was $5.6×10^4$ MPN/100 mL, the average log_{10} reduction via microscreening and UV disinfection was 2.0 units. In the storage pond, total coliforms decreased to, on average, $4.5×10^3$ MPN/100 mL (all data from March 2014 to June 2015). This is a reduction of 0.3 log_{10} units, compared to the effluent value ($2.2×10^4$ MPN/100 mL from March 2014 to June 2015).

The average concentrations of *E. coli* were $2.3×10^7$ MPN/100 mL in the untreated wastewater, $5.7×10^6$ MPN/100 mL in UASB 1 and $2.3×10^6$ MPN/100 mL in UASB 2 (Figure 53). This corresponds to a reduction of 0.6 to 1.0 log_{10} units during sedimentation and anaerobic pre-treatment.

In the effluent of the RBCs and lamella clarifiers, the average values for *E. coli* were $1.6×10^4$ MPN/100 mL (RBC/LC 1) and $4.1×10^4$ MPN/100 mL (RBC/LC 2), respectively. The average reduction from the effluent of UASB 1 to RBC/LC 1 was 2.5 log_{10} units.

When the UV disinfection was functioning properly, effluent *E. coli* concentrations were 85 MPN/100 mL. The log_{10} reduction was 2.1 log_{10} units (*E. coli* in RBC/LC 1 = $1.1×10^4$ MPN/100 mL).

The average E. coli concentration in the storage pond was 40 MPN/100 mL (all data); thus, *E. coli* was reduced by about 1.7 log_{10} units (*E. coli* in the effluent including all values from January 2014 to June 2015: $2.2×10^3$ MPN/100 mL). More details regarding the performance of UV disinfection and microbiological water quality objectives are provided in Sections 4.5.2.2, 4.5.2.8 and 4.5.2.10 (page 127*ff.*, 133*f.* and 135).

Thermotolerant coliforms and enterococci were monitored between August 2013 and September 2014. Monitoring was put on hold in favor of monitoring *E. coli* and total coliforms. The main reason was that *E. coli* is the indicator organism suggested by WHO (2006) and was therefore used for monitoring in this project.

The average number of thermotolerant coliforms was $2.1×10^7$ MPN/100 mL in the untreated water (Figure 54). $3.1×10^6$ MPN/100 mL was monitored in the effluent of UASB 1 and $2.5×10^6$ MPN/100 mL was monitored in the effluent of UASB 2. Thus, thermotolerant coliforms were reduced by 0.8 to 0.9 log_{10} units during sedimentation and anaerobic pretreatment.

In the effluent of the RBCs and lamella clarifiers, they were further reduced to $4.1×10^4$ MPN/100 mL (RBC/LC 1) and $7.5×10^4$ MPN/100 mL (RBC/LC 2), respectively. This corresponds to an average reduction of 1.7 log_{10} units.

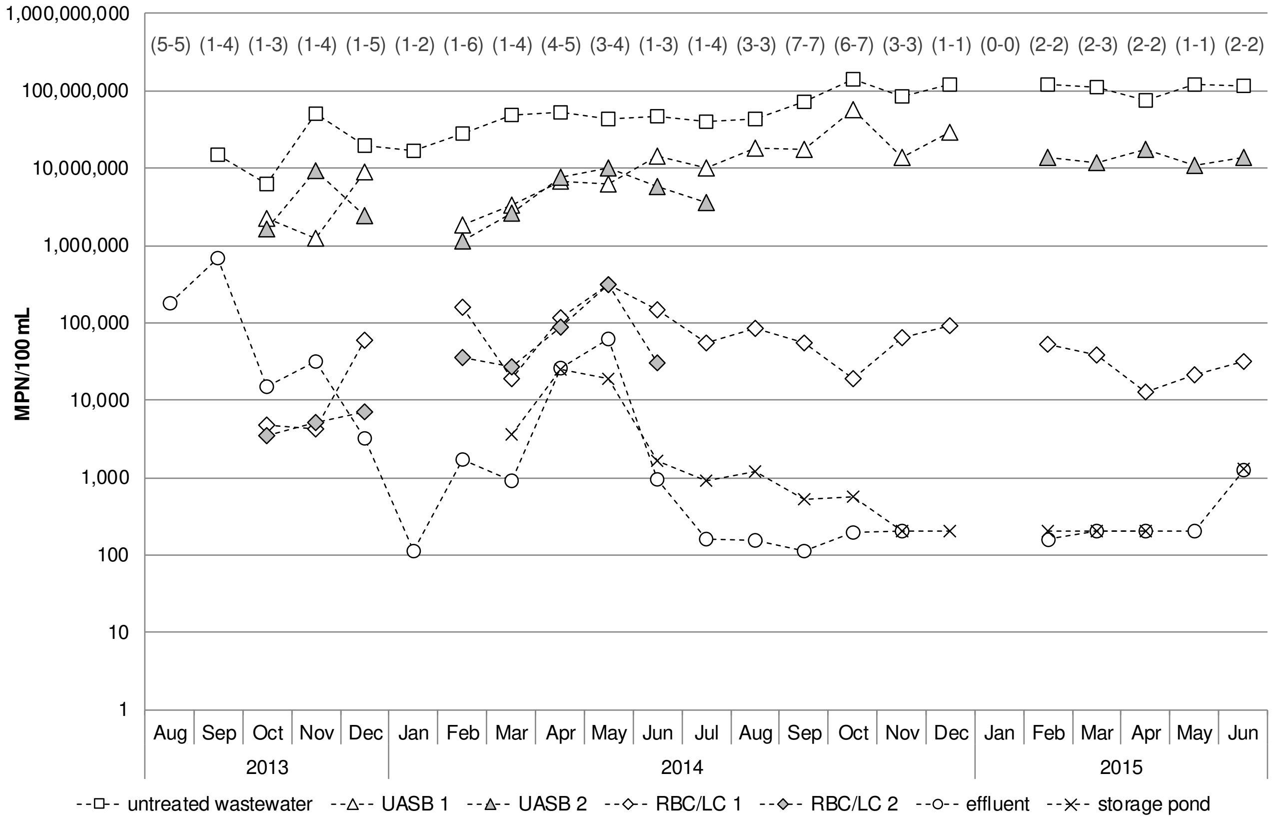

Figure 52 Most probable number of total coliforms; the range of the number of measurements is given in brackets; e.g., (1-4) in September 2013 signifies that the number of values per sampling point ranges from 1 to 4

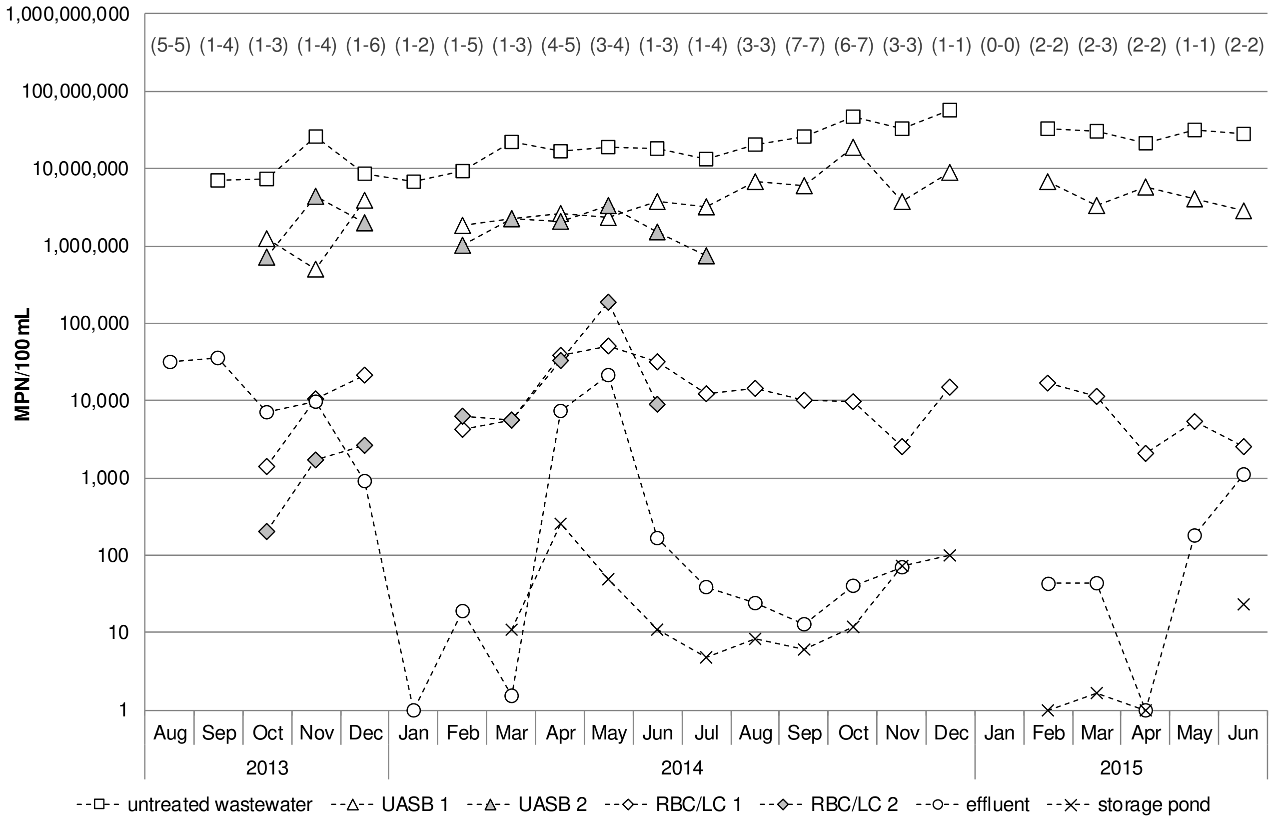

Figure 53 Most probable number of E. coli; the range of the number of measurements is given in brackets; e.g., (1-2) in January 2014 signifies that the number of values per sampling point ranges from 1 to 2

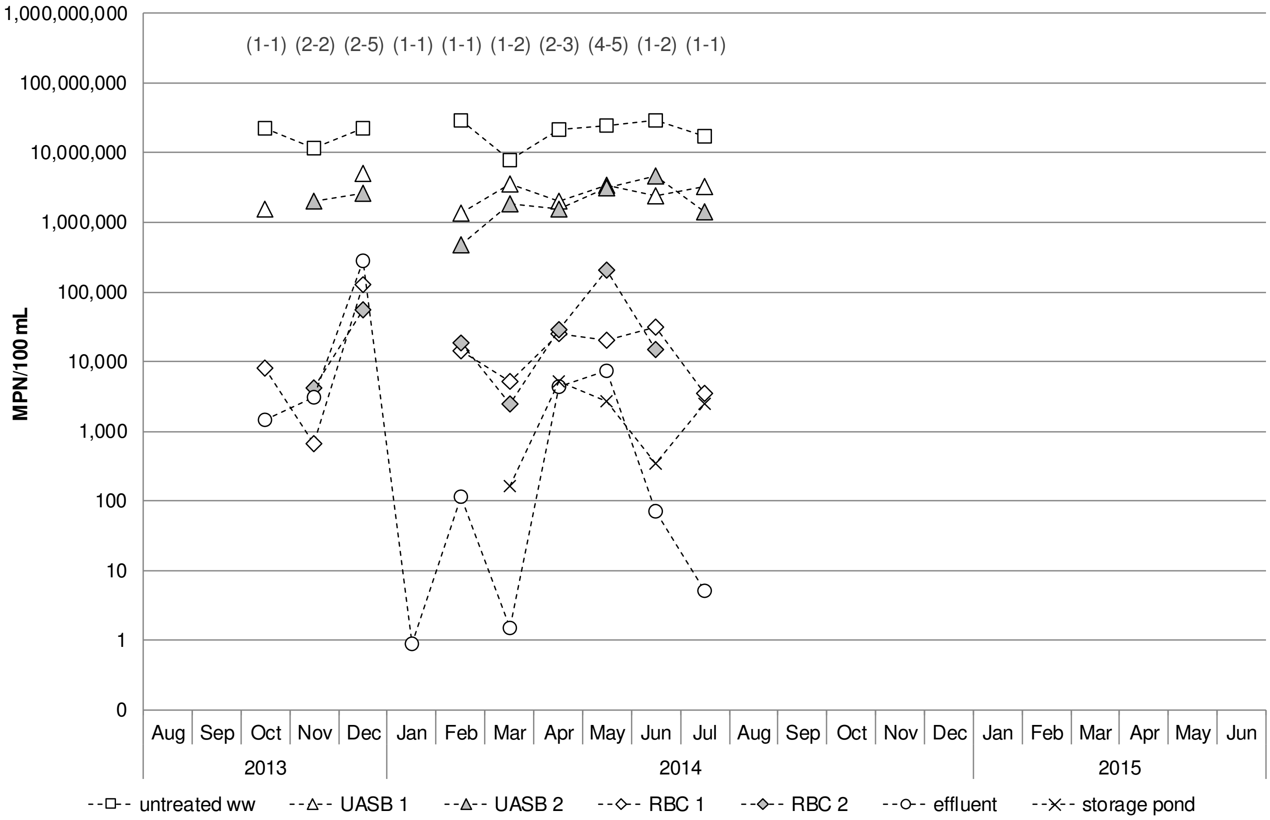

Figure 54 Most probable number of thermotolerant coliforms; the range of the number of measurements is given in brackets; e.g., (1-1) in October 2013 signifies that the number of values per sampling point was 1

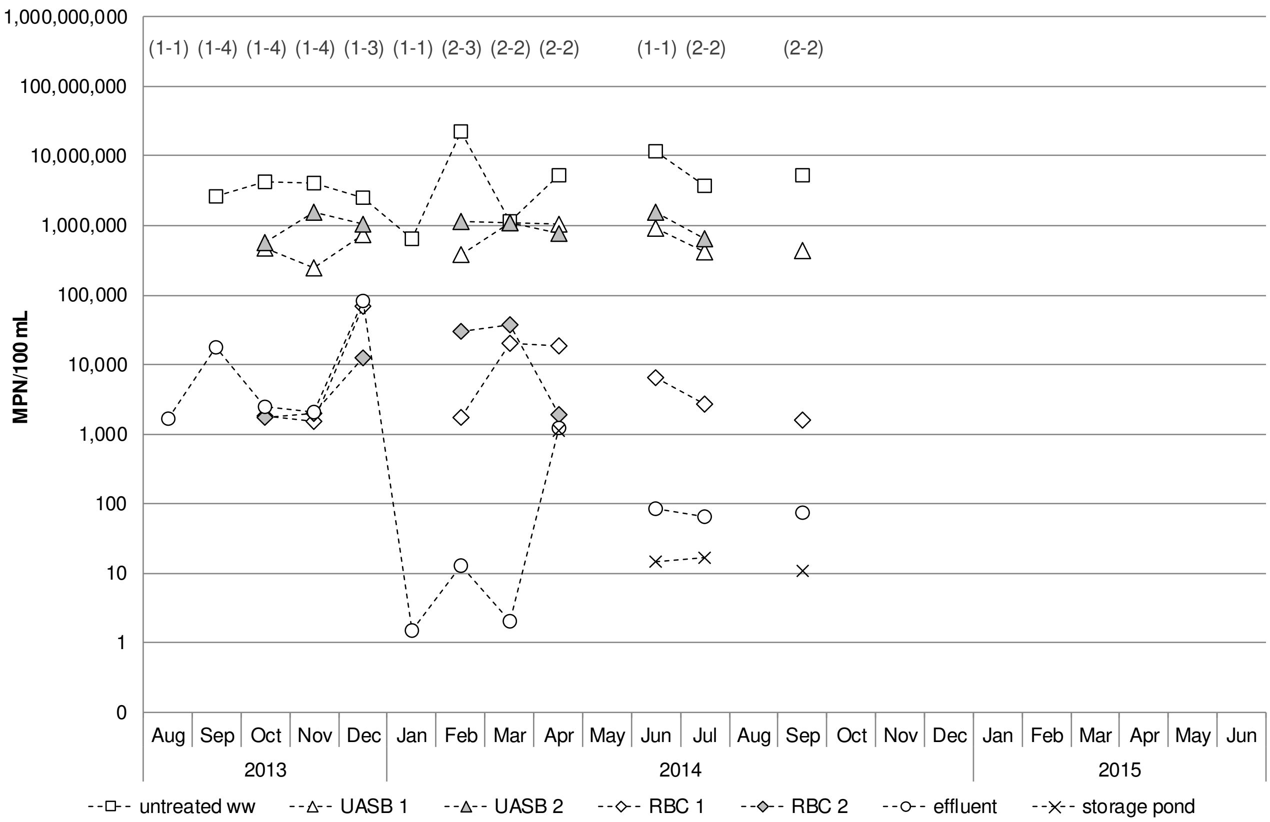

Figure 55 Most probable number of enterococci; the range of the number of measurements is given in brackets; e.g., (1-4) in September 2013 signifies that the number of values per sampling point ranges from 1 to 4

In the effluent, the average concentration was 38 MPN/100 mL during the months with correct operation (January to March 2014 and June/July 2014). The mean $\log_{10}$ reduction was 2.5 units (average value for RBCs effluent and lamella clarifiers = 1.2×10^4 MPN/100 mL).

Thermotolerant coliforms numbered, on average, 2.5×10^3 MPN/100 mL in the storage pond. The average $\log_{10}$ reduction during storage is 0.1 units (effluent: 3.3×10^3 MPN/100 mL).

Enterococci were present in concentrations of 5.6×10^6 MPN/100 mL in the untreated wastewater (Figure 55). The average concentrations were 6.4×10^5 MPN/100 mL and 1.1×10^6 MPN/100 mL in the effluent of UASB 1 and UASB 2, respectively, and 1.5×10^4 MPN/100 mL and 1.4×10^4 MPN/100 mL in the effluent of the RBC/LC 1 and 2, respectively. Reduction rates corresponded to 0.8 $\log_{10}$ units for sedimentation and anaerobic pretreatment and 1.8 $\log_{10}$ units during aerobic treatment.

UV disinfection reduced the enterococci concentration from 2.0×10^4 MPN/100 mL (effluent RBCs and lamella clarifiers) to 4.8×10^3 MPN/100 mL (effluent) or by 0.6 $\log_{10}$ units (January 2014 to September 2014, excluding values in April and May 2014).

The average concentration in the storage pond was 1.9×10^2 MPN/100 mL. The $\log_{10}$ reduction during storage was 1.8 units.

4.2.10 Comparison of planning and monitoring data

The planning data regarding wastewater quantities, loads and concentrations were presented and discussed in Section 4.1.1 (page 53*ff.*) and Section 4.1.5 (page 64*ff.*). After implementation, all loads and concentrations were considerably lower than estimated (Table 11). The mean TCOD, BOD_5, TN and TP concentrations in the untreated water were only 44%, 24%, 43% and 62% of the planned values, respectively. The loads were only 15%, 8%, 14% and 20%, respectively.

Because no removal steps were implemented for TDS, TN, and TP, the lower concentrations and loads were not problematic for the operation of the wastewater treatment plant. For irrigation, the lower loads and concentrations meant lower risks of overfertilization and soil salinization.

In contrast, sufficient TCOD concentrations are crucial for the effectiveness of anaerobic treatment steps. Anaerobic treatment is a reasonable alternative to aerobic treatment for TCOD concentrations above 1,500 to 2,000 mg/L (Tchobanoglous *et al.* 2004). For TCOD concentrations below 1,300 mg/L, aerobic treatment should be preferred (Tchobanoglous *et al.* 2004). According to the planning data, TCOD concentrations in the influent should have been sufficient for anaerobic treatment. Following implementation, concentrations were below the recommended range.

The monitored TCOD and PCOD reduction during sedimentation/anaerobic digestion was higher than the assumed value of 50%. 69% of the TCOD and 79% of the PCOD were removed

(Table 11). The DCOD reduction was lower (34%). The BOD_5 reduction during sedimentation and anaerobic digestion was estimated at 50%. The observed reduction was 73%.

According to the literature, the TCOD reduction is roughly 65.6% during anaerobic digestion (hydraulic retention time = 7 hours, 65.6% = $100 \times (1-0.68 \times 7\ h^{-0.35})$, Chernicharo (2007)). In the literature, values for TCOD removal rates in UASB reactors are mostly between 70% and 80%, ranging from 45% to 90% for a similar temperature range as in Outapi (see Seghezzo *et al.* (1998) and Figure 31, page 70). The DCOD reduction ranges between 20% and 60%, whereas most reduction rates lie between 40 and 50% (all temperature ranges, Seghezzo *et al.* (1998)).

According to an empirical formula in Chernicharo (2007), the BOD_5 reduction during anaerobic pretreatment is 73.5% (= $100 \times (1-0.7{\times}7h^{-0.5})$). Seghezzo *et al.* (1998) give reductions between 64% and 93% for a similar temperature range as in Outapi. Altogether, the observed TCOD-, PCOD-, DCOD- and BOD_5-reductions during sedimentation and anaerobic pretreatment observed in Outapi comply with the values given in the literature.

TN reduction was 21%, compared to an assumed value of 0%, and TP reduction was 13%, compared to the assumption of 20%. Because TN and TP removal steps were not projected, planning data did not consider the reduction of TN during anaerobic pretreatment and the reduction of TP during aerobic treatment.

In the influent of the RBCs, the TCOD loads constituted only 26% of the originally planned loads; thus, unintended nitrification occurred during the first months of operation, until water quantities increased. Because a denitrification step or pH adjustment was not projected, the pH decreased during aerobic treatment, because the buffering capacity of the water became exhausted (see Chapter 4.5.2.3, page 130*f.*). This could cause corrosion of the downstream infrastructure.

The mean TCOD concentration in the effluent of the RBCs/lamella clarifiers was 55.7 mg/L and, thus, lower than the planning objective of 70.0 mg/L. The percentual reduction was lower (75% compared to 92%).

The same applies to the BOD_5- and P-concentrations. The mean concentrations in the effluent of the RBCs/lamella clarifiers were lower after implementation (TP: 8.4 mg/L, BOD_5: 9.0 mg/L) than the planning values (TP: 15.3 mg/L, BOD_5: 30.0 mg/L). The relative reductions were higher than the planning data (TP: -6% versus 0%, BOD_5: -83% versus -93%. The BOD_5 and TP-loads were also lower. The mean loads in the effluent of the RBCs/lamella clarifiers after implementation were 30% (BOD_5) and 55% (TP) of the planning value.

The microscreen did not reduce the average TCOD value to the intended degree. Despite the relatively low TCOD concentration in the effluent of the RBCs/lamella clarifiers, the target concentration of 50.0 mg/L was exceeded by 7.7 mg/L. Hence, the intended TCOD reduction of 29% was not achieved. Further details about the performance of the microscreen are presented in Section 4.5.2.2 (page 127*ff.*).

Table 11 Planning data and monitoring results for loads and concentrations (conc.) during wastewater (ww) treatment (planning data provided by BWT (2011), poly. P + org. P = polyphosphate and organic phosphorus

	untreated wastewater		untreated ww and return flow anaerobic digester		sedimentation, anaerobic pretreatment (UASB)			aerobic treatment (RBC), secondary clarification (LC)			microscreen, UV disinfection			storage pond	
	load	conc.	effluent load	effluent conc.	effluent load	effluent concentration		effluent load	effluent concentration		effluent load	effluent concentration		effluent concentration	
	kg/d	mg/L	kg/d	mg/L	kg/d	mg/L	Δ%	kg/d	mg/L	Δ%	kg/d	mg/L	Δ%	mg/L	Δ%
planning data (untreated wastewater: 90 m³/d)															
TCOD	150	1,667	158[a]	1,750[a]	78.8[e]	875[e]	-50	6.3[i]	70[i]	-92	4.5[i]	50[i]	-29	-	-
DCOD	105	1,167	110[a]	1,225[a]	55.1[e]	613[e]	-50	-	-	-	-	-	-	-	-
PCOD	45	500	47.3	525	23.6	263	-50	-	-	-	-	-	-	-	-
BOD_5	75.0	833	78.8[a]	875[a]	39.4[e]	438[e]	-50	2.7[i]	30[i]	-93	2.3[i]	25[i]	-17	-	-
TN	12.0	133	13.8[b]	153[b]	13.8[f]	153[f]	0.0	11.8[j]	131[j]	-14	11.8[f]	132[f]	0.0	-	-
NH_4^+-N	9.0	100	10.4[c]	115[c]	10.4[f]	115[f]	0.0	11.3[k]	126[k]	+10	11.3[f]	126[f]	0.0	-	-
org. N	3.0	33.3	3.5[d]	38.3[d]	3.5[f]	38.3[f]	0.0	0.05	0.5	-99	0.05[f]	0.5[f]	0.0	-	-
NO_3^--N	0.0	0.0	0.0	0.0	0.0	0.0	-	0.45	5.0[l]	-	0.45[f]	5.0[f]	0.0	-	-
TKN	12.0	133	14	153	13.8	153	0.0	11.4	127	-18	11.4	127	0.0	-	-
TP	1.5	16.7	1.7[b]	19.2[b]	1.4[g]	15.3[g]	-20	1.4	15.3	0.0	1.4	15.3	0.0	-	-
TSS	90.0	1,000	94.5[a]	1,050[a]	28.4[h]	315[h]	-70	2.7[i]	30[i]	-90	0.45[i]	5.0[i]	-83	-	-
monitoring results (untreated wastewater: 30 m³/d)															
TCOD	22.1	738	22.1	738	6.8	227	-69	1.7	55.7	-75	1.7	57.7	+4	64.9	+12
DCOD	5.0	166	5.0	166	3.3	109	-34	1.1	36.2	-67	1.2	40.8	+13	39.6	-3
PCOD	17.2	572	17.2	572	3.5	118	-79	0.6	19.5	-83	0.5	17.0	-13	25.2	+49
BOD_5	5.9	196	5.9	196	1.6	53.3	-73	0.3	9.0	-83	0.2	5.5	-38	16.0	+189
TN	1.7	57.5	1.7	57.5	1.4	45.6	-21	1.1	35.9	-21	1.0	33.5	-7	32.6	-3
NO_3^--N	0.0	0.5	0.0	0.5	0.0	0.5	-7	0.7	23.4	+4,586	0.6	19.8	-15	25.4	+28
NO_2^--N	-	-	-	-	-	-	-	0.0	0.2	-	0.0	0.4	+117	0.1	-78
NH_4^+-N	0.9	29.2	0.9	29.2	1.1	36.2	24	0.3	9.8	-73	0.3	9.3	-5	1.4	-85
org. N	0.8	27.7	0.8	27.7	0.3	8.9	-68	0.1	2.7	-69	0.1	4.4	+64	5.8	+30
TP	0.3	10.3	0.3	10.3	0.3	9.0	-13	0.3	8.4	-6	0.2	8.3	-1	9.9	+18
PO_4^{3-}-P	0.2	6.4	0.2	6.4	0.2	7.8	20	0.2	7.9	2	0.2	7.8	-2	8.8	+14
poly. P+org. P	0.1	3.9	0.1	3.9	0.04	1.2	-69	0.02	0.5	-59	0.0	0.6	+11	1.0	+80

[a] 5% of untreated wastewater; [b] 15% untreated wastewater; [c] 15% of untreated wastewater, NH_4^+-N = 75% of TN; [d] 15% of untreated wastewater, organic N = 25% of TN; [e] reduction: 50%; [f] no reduction; [g] reduction: 20%; [h] reduction: 70%; [i] water quality objective; [j] incorporation into biomass = 5% of BOD_5; [k] NH_4^+-N = 96% of TN; [l] NO_3^--N = 3.8% of TN

Most constituents displayed declining concentrations in the course of wastewater treatment. NH_4^+-N and PO_4^{3-}-P concentrations increased during anaerobic pretreatment in the UASB, due to hydrolysis. NO_3^--N increased during aerobic treatment because NH_4^+-N was oxidized.

In the storage pond, NH_4^+-N is further oxidized and NO_3^--N concentrations increase. DCOD concentrations stay approximately the same. All other concentrations increase due to evaporation, algae growth, and input via animal organisms in the water.

The $\log_{10}$ reductions of the monitored microbiological parameters largely comply with the literature values (Table 12). Reduction during secondary treatment is slightly higher than reported in the literature. This could be due to relatively high hydraulic retention times caused by smaller water quantities. The monitored average reduction for microscreening and UV disinfection ranges from 0.6 to 2.5 log10 units (dose > 100 mJ/cm²). WHO (2006) gives reduction rates > 3 $\log_{10}$ units only for UV disinfection. Hijnen *et al.* (2006) report $\log_{10}$ reductions up to 5.6 at lower doses for bacteria. Thus, the achieved average reduction of 1.8 $\log_{10}$ units is very low.

Table 12 Log unit reduction for several wastewater treatment processes in the literature (WHO 2006) and monitored reductions in this study

	removal of indicator organisms ($\log_{10}$ units)					
	this study					WHO 2006
treatment process	**total coliforms**	**thermotolerant coliforms**	***E. coli***	**Enterococci**	**average**	**bacteria**
primary sedimentation + UASB	0.8	0.9	0.8	0.8	0.8	0 to 1 0 to 1
secondary treatment	2.2	1.7	2.5	1.8	2.1	0 to 1
UV disinfection	-	-	-	-	-	> 3
microscreening and UV disinfection	2	2.5	2.1	0.6	1.8	-
storage	0.2	0.1	1.7	1.8	1.0	1 to 4

4.2.11 Conclusions

This section provided an overview of the monitored water quantities and the main physical, chemical, and biological characteristics of the untreated wastewater and the changes during wastewater treatment. The monitored data were compared with planning data.

The treated water quantities were only one third of those expected. They increased due to leaking sanitary installations at the shared sanitation facilities, decreased after repairs and then remained relatively stable. The increasing water quantities from November 2013 to January 2014 mainly originated from tap water and from leaking taps and toilets.

During this time period, the monitored concentrations of the wastewater parameters also decreased. This dilutive effect was observed in the effluent of all treatment steps. Decreasing concentrations could therefore be used as an indicator for leaking installations at the shared sanitation facilities. Hence, in addition to regular inspections of the sanitation facilities, it was recommended to the operators of the wastewater treatment plant to use the EC values of the untreated wastewater as an additional means for detection of leakages. The EC is especially suitable to be used as an indicator for increased tap water use due to its ease and rapidity of determination.

Start-up of the cluster units increased the water quantities but had no distinct effect on the quality of the untreated water. Leaking sanitary installations led to decreasing concentrations of all parameters. In contrast, the connection of the individual households to the vacuum sewers led to increasing TCOD, TN and TP concentrations. The increase in TCOD was mainly due to an increase in PCOD. DCOD remained more or less at the same level. With respect to TN and TP, the monitored N and P forms increased to approximately the same extent. This suggests that the wastewater generated by the individual households is more concentrated than that from the shared sanitation facilities.

The EC increased and decreased considerably during wastewater treatment. This was mainly caused by hydrolysis during anaerobic treatment and consumption of HCO_3^- during nitrification. In this case, EC increased by 76 µS/cm or 12% during anaerobic digestion and by 157 µS/cm or by 23% during nitrification. Because neither salts nor chemicals are added or removed and dissolved substances are only removed via incorporation into cell biomass or adsorption to sedimented particles, one would expect the EC to remain more or less at the same level during wastewater treatment. However, this example shows that variations during anaerobic digestion and nitrification can be considerable. This needs to be considered when using the EC as a surrogate for balancing TDS in the water.

The water quality in the effluent of the UASB reactors was mainly influenced by the start of the anaerobic digestion process. The TCOD and DCOD concentrations decreased within a few weeks and remained relatively stable afterwards. The increasing TCOD and PCOD concentrations in March and April 2014 did not influence the effluent concentrations. Shutdown of one of the UASB reactors in July 2014 led to only a minimal increase in TCOD effluent values. Hence, start-up of the anaerobic processes in the UASB reactors occurred within a short time period. Thereafter, the UASB worked reliably.

The TN and TP concentrations in the effluent of the UASB reactors remained approximately the same from May 2013 to March 2014, increased from April 2014 to June 2014 and then remained relatively constant. The increase was caused by increases in ammonium N and orthophosphate P. Organic N, polyphosphate P and organic P concentrations did not increase. The increases in ammonium N and orthophosphate P were caused by changes in the water quality that resulted from connection of the individual households to the sewer system.

Unintended denitrification occurred during the first months of operation with lower water quantities. If this has to be prevented, additional maintenance of the lamella clarifiers is required.

The microscreen did not always compensate for the additional material input that led to deterioration of the water quality after the aerobic treatment step. The $\log_{10}$ reductions of the monitored microbial parameters were also lower than those reported in the literature even though

UV disinfection was working properly at very high doses (> 100 mJ/m²). These findings indicate that the performance of the microscreen might be insufficient. This is an important point, because the microscreen was installed to remove helminth eggs and particulate matter, in order to achieve optimal UV disinfection. Thus, it influences three parameters (concentration of helminth eggs and TSS, $\log_{10}$ reduction of *E.* coli) that are used for setting water quality objectives (Chapter 4.5, page 122).

For the wastewater characteristics, the most important point for this project was that the water quantities, loads and concentrations were considerably below the values assumed for planning. They constituted only between 8% and 20% (loads) and 23% to 61% (concentrations) of the planning data. This was of significance for subsequent treatment steps and reuse options. The planned hydraulic and organic loading capacities of the wastewater treatment plant were not fully used. Operation remained on a low level of efficiency. Less water and stabilized sludge were available for irrigation and fertilization of the agricultural fields. Lower quantities of sewage sludge and biomass residues were available for co-digestion. More details on the consequences for the sanitation system are discussed in the following sections.

4.3 Wastewater characteristics and utilization of shared and individual sanitation facilities

As outlined in the introduction, shared sanitation facilities are often the only feasible option for sanitation provision in settlements with sub-standard housing structures. Information regarding quantities and constituent loadings of such facilities were not available for planning. In order to provide pioneer data regarding shared sanitation, this chapter presents the wastewater characteristics and utilization rates of each type of sanitation facility. The main wastewater constituents are quantified, to derive the specific water uses and specific loads.

The first section focuses on the communal washhouse. Data on water use, concentrations and utilization rates collected from May 2013 to October 2013 are used to determine the total and specific loads and the specific water use. Regarding water use and utilization, the influence of the two tariffs applied during the project period is emphasized. Because the relatively low utilization of the communal washhouse was an important issue during implementation, the possible reasons and measures to increase utilization are further explored.

The next parts are devoted to the number of potential users, total and specific water use, water quality and loads of the cluster units and the individual households. The last section compares the specific values determined for each type of sanitation facility, both with each other and with planning data.

4.3.1 Communal washhouse

4.3.1.1 Daily water use

The mean water use of the communal washhouse was 16.7 m³/d (Figure 56). This is only slightly higher than the planned mean water use of 15.0 m³/d.

During the project period, two tariffs were tested at the communal washhouse. Initially, a fee of 0.5 NAD (Namibian dollars) per use or 0.04 euros (EUR) per use (= tariff 1, average currency exchange rate from May 2013 to August 2014, www.oanda.com) was introduced. In September 2014, the OTC decided to introduce a fee of 2 NAD/use or 0.13 EUR/use (= tariff 2, average currency exchange rate from September 2014 to September 2015, www.oanda.com).

Prior to the introduction of the higher fee, the mean water use was 19.2 m³/d (May 2013 to August 2014). Thereafter, the mean water use decreased to 14.5 m³/d (September 2014 to September 2015). The decrease was observed for all possible uses. It decreased from 11.1 m³/d (tariff 1) to 8.6 m³/d (tariff 2) for showers and hand wash basins, from 5.4 m³/d (tariff 1) to 4.4 m³/d (tariff 2) for toilet flushing and from 2.6 m³/d (tariff 1) to 1.5 m³/d (tariff 2) for laundry sinks. The differences are statistically significant at the 1% significance level for the total water use, for showers/hand wash basins and laundry sinks (Wilcoxon matched-pairs test, p-value in each case = 0.000). The decrease in water use for toilet flushing is not statistically significant (p-value = 0.387). The significance level is the "probability of falsely rejecting H_0" (Jarman

2015). In this case, the p-value of 0.000 is below and the p-value of 0.387 is above the significance level of 0.01 or 1%. Hence, H_0 (the median difference between pairs is zero) is rejected (Jarman 2015).

Increasing tariffs led to an average decrease in water use by 24%, varying from a decrease of 19% for toilets, 22% for showers and hand wash basins and 44% for laundry sinks. Thus, the effect of tariff changes on water use varied among uses. The differing impact may be caused by the priority of the activities for the users (showering is more important than laundry washing).

The water use of comparable South African shared sanitation facilities is 8.2 m³/d (ranging from 4.4 to 12.0 m³/d, Crous *et al.* (2013)). These facilities are mainly used for laundry washing (4.8 m³/d or 58% of the total water use), followed by showering and hand wash basins (1.8 m³/d or 22% of the water use), toilet flushing (1.3 m³/d or 16%) and urinal flushing (0.2 m³/d or 2.7%).

The daily water demand of the facility in South Africa is lower than the daily water use of the facility in Outapi. One similarity between the two sites is the large fraction of greywater. In Outapi, 71% of the wastewater is greywater (mainly from showers); in the South African case, this value is 81% (mainly from laundry sinks).

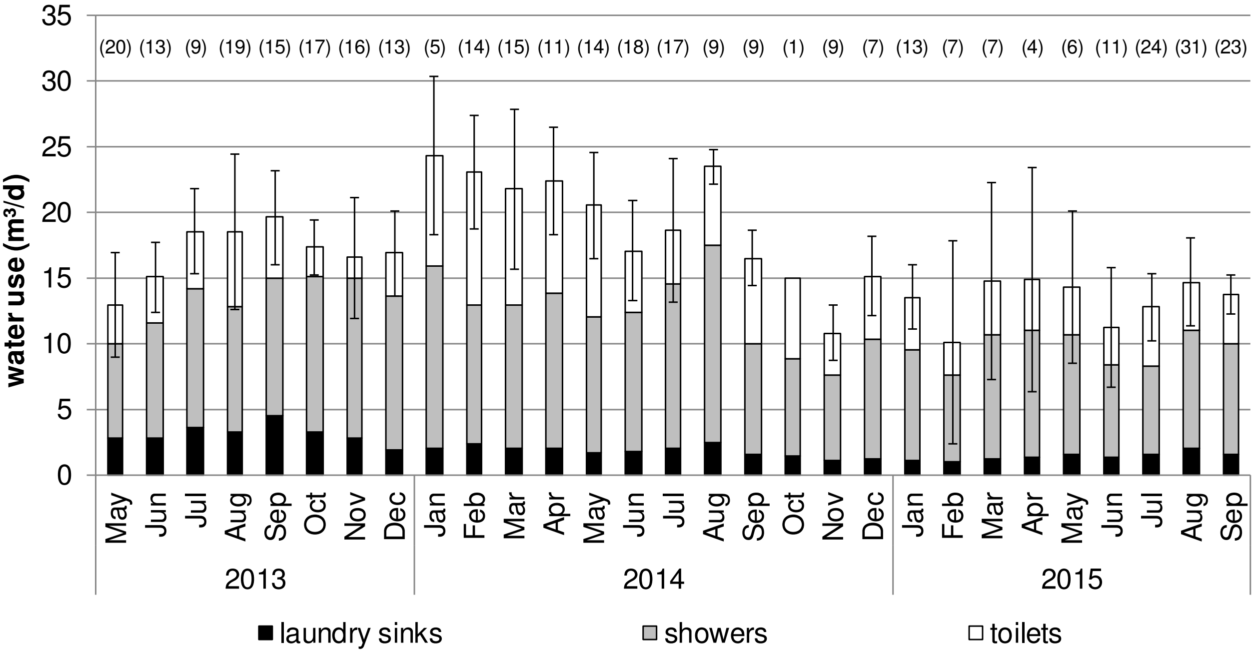

Figure 56 Average daily water use at the communal washhouse; bars represent the standard deviation of the mean total water use, the number of values is given in brackets, showers also include water from hand wash basins

Water use of urinals plays a minor role in the South African example. Urinals are also only used to a limited extent in Outapi (0.14% of the total water use). In both cases, water use for toilets is much lower than for the other purposes. It can be assumed that utilization of toilets is low in Outapi due to the necessity of payment. However, operating costs of the South African facilities are covered by the municipality and households are not charged for their use (Crous

2014). Nevertheless, water use for toilets is low. This means that other factors lead to the low utilization of toilets. Possible barriers to toilet use could be the general preference of open defecation, the cleanliness of the toilets, or the distance to be covered for using the facilities.

4.3.1.2 Utilization

During the entire monitoring period, the average utilization rate of the communal washhouse was 166 uses/d (Figure 57 and Table 13). This was far below the expected utilization rate of 750 uses/d (see Section 4.1.2, page 55). Before the introduction of the higher tariff, the average utilization rate was 208 uses/d. Subsequently, it was 120 uses/d. The difference is statistically significant at the 1% significance level (Wilcoxon matched-pairs test, p-value = 0.000).

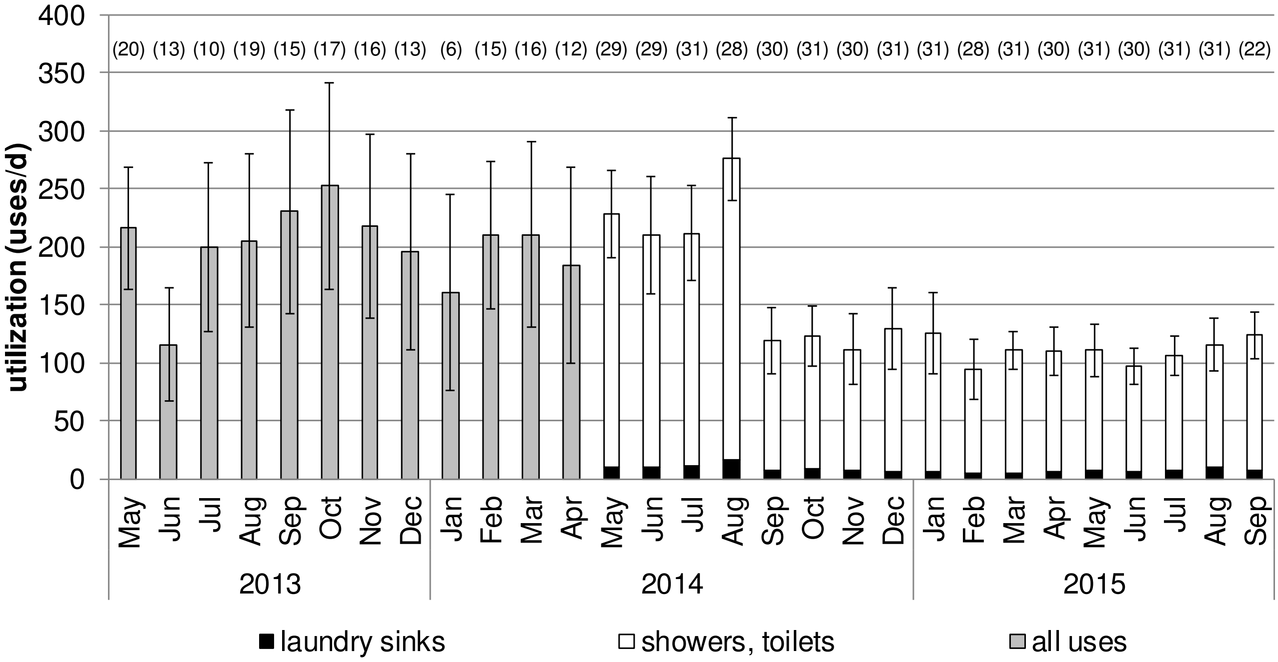

Figure 57 Average daily utilization of the communal washhouse; bars represent the standard deviation of the mean total utilization, the number of values is given in brackets

Both before and after the tariff change, utilization was much lower than planned. The facility was constructed to provide sanitation services to 250 persons or 750 uses/d. The actual utilization rates represented only 28% (tariff 1) or 16% (tariff 2) of the assumed value.

On average, 7.8 uses/d were recorded for laundry washing and 133 uses/d for showering and toilet flushing (May 2014 to September 2015, Table 13). Thus, the majority of visits was related to showering/toilet use. A differentiation regarding the actual activities carried out (toilet use and/or showering) during one visit was not carried out.

Before the introduction of the higher tariff in September 2014, 11.3 uses/d were recorded for laundry washing and 218 uses/d for showers and toilets. Afterwards, 6.7 uses/d were recorded for laundry washing and 107 uses/d for showers and toilets. These differences are statistically significant at the 1% significance level (Wilcoxon matched-pairs test, p-value = 0.000).

Table 13 Planned and monitored utilization and water use of the communal washhouse, sd = standard deviation of the mean, n = number of measurements, pe = population equivalents

	planned	monitored								
		May 2013 - Sep 2015			May 2013 - Aug 2014 (tariff 1)			Sep 2014 - Sep 2015 (tariff 2)		
	mean	mean	sd	n	mean	sd	n	mean	sd	n
utilization										
	uses/d	uses/d	uses/d	-	uses/d	uses/d	-	uses/d	uses/d	-
all uses	750	166	72.1	676	208	68	215	120	27	109
showers, toilets*	-	133	56.4	112	218	44	218	107	25	106
laundry*	-	7.8	6.2	6.0	11	7	10	7	6	5
* data collection started in May 2014										
water use										
total	m³/d	m³/d	m³/d	-	m³/d	m³/d	-	m³/d	m³/d	-
	15.0	16.7	5.0	387	19.2	4.8	231	14.5	4.4	156
per use	L/use	L/use	L/use	-	L/use	L/use	-	L/use	L/use	-
	20.0	101	36.8	377	92.3	32.5	225	120	30.3	152

70% of the visits for showering and toilet use occurred between 6:00 am to 10:00 am and 4:00 pm to 7:00 pm (Figure 58). For laundry washing, peak time was between 6 am and 10 am. During peak times, the number of showers in the section for men was not always sufficient; queuing was sometimes observed.

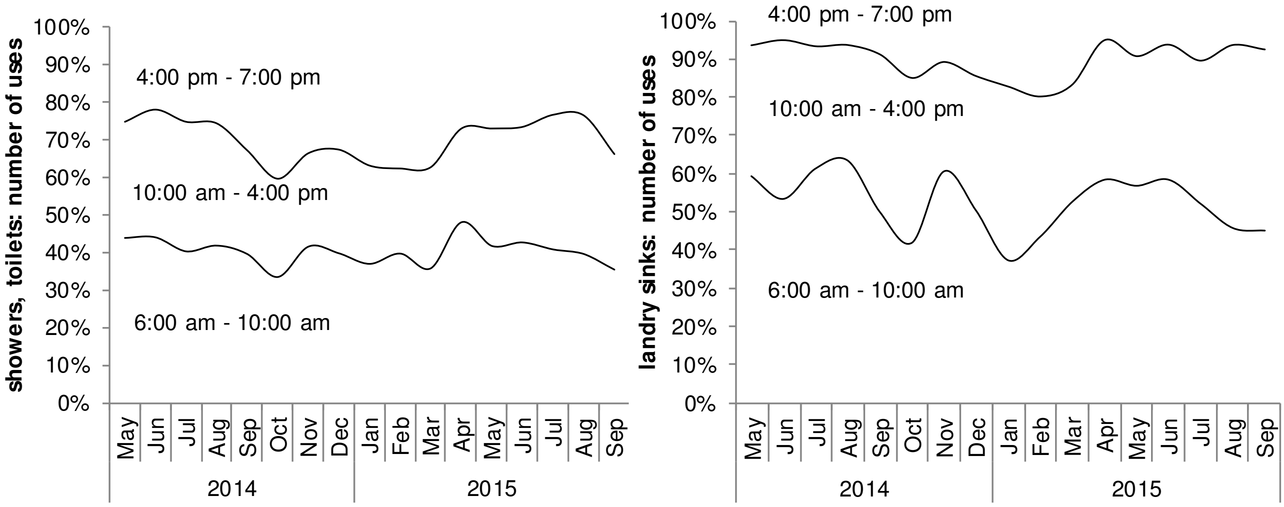

Figure 58 Distribution of the number of uses per time of the day for showers and toilets (left) and laundry sinks (right)

Examples of shared sanitation facilities located in Kenya and India (Table 3, page 17) are characterized by higher utilization rates than Outapi. Only three reported cases have similar or lower utilization rates than in Outapi. Even when the number of uses is referred to each toilet, the utilization rate in Outapi is much lower (only 16 (tariff 1) or 9 (tariff 2) per day).

In an ideal situation, all persons living in the surroundings of a shared sanitation facility would use it. Utilization would be relatively high in areas with a relatively high population density and relatively low in areas with a relatively low population density. Hence, a closer look is taken at the potential utilization in Outapi considering the local population density.

The population density in Outapi is approximately 21.5 persons/ha (see Section 3.1, page 40). This is a low-density area according to the categories in Kalbermatten *et al.* (1982) (up to 300 persons/ha). It is assumed that the catchment area of the communal washhouse has a minimum radius of 200 m and a maximum radius of 500 m (corresponding to a 5 to 10 minute walk). Thus, about 272 persons live within a distance of 200 m and about 1,699 persons live within a distance of 500 m from the communal washhouse (Table 14). The potential utilization rate is between 816 uses/d (= 272 persons × 3 uses/(person×d)) and 5,097 uses/d (= 1,699 persons × 3 uses/(person×d)). Thus, by calculation, the communal washhouse covers 4% to 25% (tariff 1) or 2% to 15% (tariff 2) of the potential number of uses in the assumed catchment area.

Table 14 Estimated number of persons, potential and monitored utilization rates for several perimeters around the communal washhouse and the two different tariffs, t 1 = tariff 1 (0.5 NAD/use), t 2 = tariff 2 (2 NAD/use)

		perimeter around the communal washhouse							
	unit	**200 m**		**300 m**		**400 m**		**500 m**	
settlement characteristics									
area	ha	13		28		51		79	
population density	persons/ha	21.5		21.5		21.5		21.5	
number of residents	persons	272		611		1,087		1,699	
utilization		**t 1**	**t 2**	**t 1**	**t 2**	**t 1**	**t 2**	**t 1**	**t 2**
a) potential, assumption: 3 uses/(person×d) = planning data	uses/d	815		1,834		3,261		5,096	
b) potential, assumption: available funds = 2 % of income	uses/d	121	30	272	68	484	121	756	189
c) observed	uses/d	tariff 1: 208, tariff 2: 120 (perimeter: not known)							

With regard to the examples of shared sanitation facilities in other locations (Table 3, page 17), Biran *et al.* (2011) mention that the study site in India is situated in "discrete areas of very high density". Schouten and Mathenge (2010) report a population density of "2,000 people per hectare" for their study site. Considering this population density of 2,000 persons/ha, the reported utilization rates of between 50 and 896 uses/d for the Indian and Kenyan cases correspond to up to 1.2% of the utilization potential of the local population within a radius of 200 m around the shared sanitation facility (0.012 = 896 uses/d ÷ (13 ha × 2,000 persons/ha × 3 uses/(person×d)). Kalbermatten *et al.* (1982) consider areas with a population of 500 to 600 persons per hectare high-density areas. Even if this lower population density is considered, the Kenyan and Indian examples only cover up to 4% of the potential number of uses in a 200 m radius (0,04 = 896 uses/d ÷ (13 ha × 550 persons/ha × 3 uses/d). For larger perimeters, the coverage is negligible.

Altogether, utilization rates and population densities in Outapi are low, but the communal washhouse covers the sanitation needs of the local residents to a much higher degree than the examples reported in literature.

The decrease in utilization rates after the introduction of tariff 2 further suggests that economic factors play an important role. The following sections therefore provide a closer look at the affordability of the communal washhouse services and the effect of distance or the size of the catchment area on the number of uses per person.

The available income in the area where the communal washhouse in Outapi was implemented is 4,064 NAD/(person×a) or 11 NAD/(person×d) (= 0.79 EUR/d) (Kramm and Deffner 2015). User fees for communal sanitation facilities implemented in Mozambique (Norman 2011) and India (Biran *et al.* 2011) correspond to up to 2% of the monthly income. Other figures on the available income for water and sanitation are, for instance, between 2% and 6% (Hutton 2012) and up to 3% (UNDP 2006) of the total income.

Assuming that the residents have 2% of their income available for the communal washhouse, they can spend 81 NAD/(person×a), thus 0.4 visits/d for tariff 1 or 0.1 visits/d for tariff 2. For the considered population density, the expected number of visits is then between 30 to 121 uses/d (radius = 200 m) and 189 to 756 uses/d (radius = 500 m) (e.g., calculation for tariff 1 and radius = 200 m: 4,064 NAD/(person×a) × 0.02 × 272 persons ÷ 0.5 NAD/use ÷ 365 d/a = 121 uses/d, Table 14).

The monitored utilization is not very different from the potential or expected utilization (Table 14). For a radius of 300 m, the monitored utilization corresponds to 76% (tariff 1) and 176% (tariff 2) of the potential utilization. For a radius of 400 m, the monitored utilization corresponds to 43% (tariff 1) and 99% (tariff 2) of the potential utilization.

For a tariff of 0.5 NAD/use, one utilization corresponds to 5% of the daily per capita income, and to 18% for a tariff of 2 NAD/use. If the communal washhouse tariffs should allow covering of all sanitation needs, with maximally, 2% of the available income, the entrance fee needs to be set at 0.07 NAD/use. A flat rate of 6 to 7 NAD per person and month would also be possible (81 NAD/a as available funds for three washhouse uses per day, 81 NAD/a ÷ 12 months/a = 6.75 NAD/month, 81 NAD/a ÷ 1,095 uses/a = 0.07 NAD/use).

4.3.1.3 Specific water use

The total water use per day and the total number of uses per day were used to calculate the water use per utilization. During the entire survey period, an average of 101 L was consumed per use. Following the introduction of the higher tariff, the specific water use increased from 92.3 L/use to 120 L/use. The observed differences are statistically significant at the 1% significance level (Wilcoxon matched-pairs test, p-value = 0.000).

A water use level of 60 L/(user×d) and 3 uses/(user×d) would result in a specific water use of 20 L/use. The monitored water use is much higher. This is attributed to the billing on a pay-per-use basis, which leads to a relatively high value for the specific water use, because users combine laundry washing, showering, and toilet use. Control of the specific water use is required to optimize the water use. This could be achieved via regular facility inspections and supervision of the users' activities by the caretakers.

A single utilization can correspond to one visit for only one activity (e.g., only laundry washing) or to one visit for several activities (e.g., showering, toilet use and laundry washing). To account for this, the number of uses was recorded separately for laundry washing visits and visits for showering/toilet use, starting from May 2014. The obtained actual specific water use was 107 L/use for showers/toilets and 218 L/use for laundry washing (May 2014 to September 2015).

After introduction of the higher tariff, the mean values decreased slightly for laundry washing, from 224 L/use to 215 L/use and increased from 80.8 L/use to 117 L/use for showers and toilets (Table 15). The p-values are 0.110 (showers, toilets) and 0.308 (laundry). Thus, the data supports the thesis that the specific water uses differ but the effect is not statistically significant.

Table 15 Specific water uses for the communal washhouse, referred to the total number of uses and referred to the actual number of uses

	planned	monitored								
		May 2013 - Sep 2015 both tariffs			May 2013 - Aug 2014 (tariff 1)			Sep 2014 - Sep 2015 (tariff 2)		
	mean	mean	sd	n	mean	sd	n	mean	sd	n
specific water use referred to the total number of uses										
	L/use	L/use	L/use	-	L/use	L/use	-	L/use	L/use	-
all uses	20.0	101	36.8	377	92.3	32.5	225	120	30.3	152
showers	-	59.0	47.1	511	53.5	58.7	298	71.7	20.2	213
toilets	-	29.2	32.1	516	26.1	37.9	302	36.5	20.3	214
laundry	-	12.3	10.2	522	12.4	11.6	307	12.1	7.8	215
specific water use referred to the actual number of uses										
		L/use	L/use	-	L/use	L/use	-	L/use	L/use	-
showers, toilets*	-	107	30.1	208	80.8	18.8	57	117	27.5	151
laundry*	-	218	123	208	224	127	56	215	121	152

4.3.1.4 Water quality and loads

Between May and October 2013 water quality and loads where examined in the wastewater originating from the communal washhouse. Average concentrations for TCOD, TDS (based on EC), TN, and TP were 579, 377, 38.6 and 8.8 mg/L, respectively (Table 16). This was much lower than expected (19% to 53% of planning data) and was most probably caused by the high percentage of greywater and the high specific water use. Average daily loads were 11.0 kg/d for TCOD, 6.5 kg/d for TDS, 0.7 kg/d for TN and 0.16 kg/d for TP. They constituted only between 22% and 66% of the planned total loads.

The determined mean loads per utilization were 45.0 g/use for TCOD, 3.0 g/use for TN, 0.69 g/use for TP, and 32.0 g/use for TDS. Compared to typical daily per capita loads for developing countries (Sperling 2007c), the specific loads per use corresponded to 45% of the anticipated daily per capita TCOD load, 27% of the TDS load, 38% of the TN load, and 69% of the TP load. Compared to the anticipated loads per use, these were lower than expected for TDS (80% of planning data) but higher for TN (113% of planning data), TCOD (135% of planning data) and TP (207% planning data).

Specific TS and TSS loads were estimated. TSS is mainly (> 95%) contained in feces and greywater (Jönsson *et al.* 2005). Only a very small fraction is contained in urine (approximately 2%) (Jönsson *et al.* 2005). Hence, the TSS distribution among urine, feces and greywater is comparable to the distribution of TCOD (Figure 4, page 13).

Thus, for the estimation of the specific loads, it was assumed that the TSS collection rate is the same as the TCOD collection rate (45% of the expected load per person per day). The total TSS load is usually 60 g/(person×d) (Sperling 2007c). The specific TSS load per use would be 27.0 g/(person×d) (= 0.45 × 60.0 g/(person×d)). The specific TS load would then be 27.0 g TSS/(person×d) + 32.0 g TDS/(person×d) = 59.0 g TS/(person×d).

Table 16 Data basis for the calculation of the specific loads from the communal washhouse and comparison with planning data; survey period: May to October 2013, sd = standard deviation, n = number of values, ci = confidence interval, TCOD = total chemical oxygen demand, TN = total nitrogen, TP = total phosphorus, TDS = total dissolved solids, EC = electrical conductivity, TSS = total suspended solids, TS = total solids

	planned	**monitored: May 2013 - October 2013**					**monitored ÷ planned**
	mean	**mean**	**sd**	**mean: 95 % ci**	**median**	**n**	
water quality							
	mg/L	mg/L	mg/L	mg/L	mg/L	-	%
TCOD	1,667	579	194	501 - 656	540	23	35
TN	133	38.6	9.6	34.7 - 42.4	39.0	23	29
TP	16.7	8.8	1.1	8.4 - 9.2	8.7	23	53
TDS	2,000	377	70.1	361 - 394	373	68	19
EC	-	µS/cm	µS/cm	µS/cm	µS/cm	-	-
	-	608	113	580 - 637	601	68	-
water use							
	m³/d	m³/d	m³/d	m³/d	m³/d	-	%
total	15.0	17.6	4.0	16.8 - 18.4	15.5	95	117
utilization							
	uses/d	uses/d	uses/d	uses/d	uses/d	-	%
total	750	204	76.0	188 - 219	212	95	27
loads							
a) total	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	25.0	11.0	4.8	9.1 - 12.9	9.0	23	44
TN	2.0	0.7	0.3	0.6 - 0.9	0.7	23	37
TP	0.25	0.16	0.03	0.15 - 0.18	0.16	23	66
TDS	30.0	6.5	2.0	6.0 - 7.0	6.1	68	22
b) specific	g/use	g/use	g/use	g/use	g/use	-	%
TCOD	33.3	45.0	17.4	38.1 - 52.0	37.3	23	135
TN	2.7	3.0	0.9	2.6 - 3.4	3.0	23	113
TP	0.3	0.69	0.19	0.61 - 0.77	0.65	23	207
TDS	40.0	32.0	17.6	27.9 - 36.2	28.4	68	80
TSS (estimate)	20.0	27.0	-	-	-	-	135
TS (estimate)	60.0	59.0	-	-	-	-	98

TN and TDS were collected to a much lower degree than the other parameters (37% and 22% of planning data, respectively). 80% of TN (Figure 4, page 13, Meinzinger and Oldenburg (2009)), 44% of the TDS load (Table 31, page 143, Jönsson *et al.* (2005)) and more than 95%

of excreted common salt (Schmidt *et al.* 2005; Sherwood 2006) is contained in urine. Hence, it seems that the users do not explicitly visit the facility for urination.

TDS is collected to a much lesser extent than TN. The mean sodium chloride intake per person per day is 9.2 g/d in the USA, 8.9 g/d in Germany and 6.6 g/d in Namibia (Powles *et al.* 2013). Thus, the Namibian intake of common salt is 27% lower than in Germany and the USA. Literature data on daily per capita TDS loads are sparse (see Table 2, page 16). The specific TDS load reported by Sperling (2007c) is supposed to be "a typical value for developing countries". However, it does not differ from the values for Central Europe (Meinzinger and Oldenburg 2009) and the USA (Tchobanoglous and Burton 1991). Assuming that the overall salt uptake of Namibians is 27% lower, the typical TDS load is 86.4 g/(user×d) instead of 120 g/(user×d). Then, the collection rate of TDS would be 37%, which is the same as for TN.

TCOD is mainly contained in feces (Figure 4, page 13, Meinzinger and Oldenburg (2009)). Its collection rate in Outapi was higher than for TDS and TN. This indicates that defecation visits were more frequent than urination visits at the communal washhouse.

The TP load per use was highest, compared to the daily per capita loads reported by Sperling (2007c). This was very probably caused by phosphates contained in detergents used for laundry washing and showering.

The actual number of users was not determined. For instance, a visitor could use the communal washhouse several times per day or only once per week. Considering the anticipated total per capita loads, the collected total daily loads represent 110 population equivalents for TCOD (11.0 kg/d × 1000 g/kg ÷ 100 g/(person×d) = 110 population equivalents), 164 population equivalents for TP, 92 population equivalents for TN, and 54 population equivalents for TDS (when referred to 120 g/(person×d)). Considering the lower salt uptake of Namibians (-27%), the adjusted TDS load would be 87.6 g/(person×d) (= 0.73 × 120 g/(person×d)). The TDS load in Outapi would then represent 74 population equivalents (= (6.5 kg/d × 1000 g/kg) ÷ 87.6 g/(person×d)), which is closer to the number of population equivalents of the TN load.

4.3.2 Cluster units

4.3.2.1 Number of potential users

The area where the cluster units were implemented is parceled into rectangular plots. The cluster units were located at the intersections; thus, in most cases four plots were allocated to each cluster unit (Figure 59). The potential number of households was 120 (4 plots or households per cluster unit, 30 cluster units). Not all plots were occupied. The actual number of households was, therefore, only 106 after implementation (Kramm and Deffner 2014). The mean household size was 3.8 persons (Kramm and Deffner 2017) instead of 7 (see Table 7, page 54). The number of potential users was 403 persons (= 3.8 persons/household × 106 households). Thus, about 13.5 persons were allocated to each cluster unit.

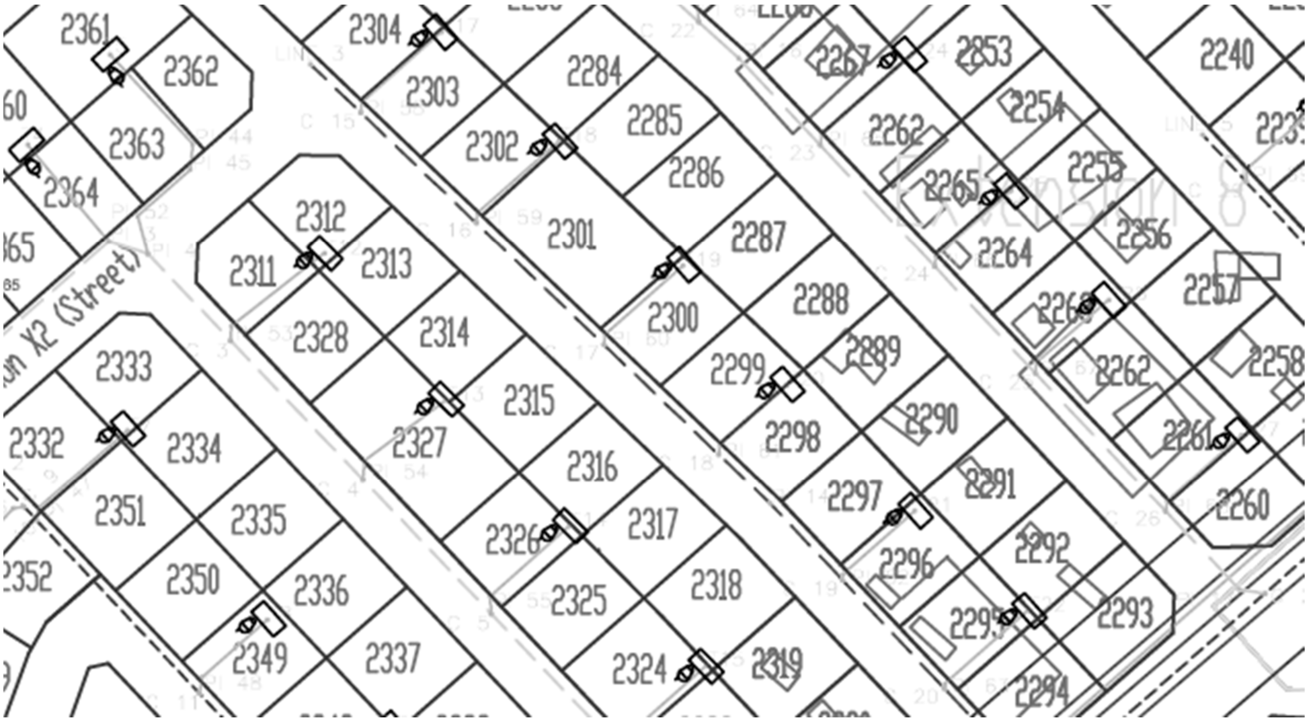

Figure 59 Map section of the area where the cluster units were constructed with plot numbers (provided by Lund Consulting Engineers, Windhoek)

4.3.2.2 Total and specific water use

The cluster units went into operation starting from November 2013 (see Figure 27, page 49). From December 2013 to June 2015, the total water use of the cluster units was 13.2 m³/d (Figure 60). On average, 9.1 m³/d were used by toilets and hand wash basins and 4.1 m³/d by showers and laundry sinks.

The water use for showers and laundry sinks showed only minor variations throughout the survey period. The water use for toilets and hand wash basins varied to a higher degree. Leaking equipment caused a relatively high water use, beginning 2014. After repairs in May 2014, the water use of toilets and hand wash basins decreased. Greywater accounted for only 31% of the wastewater during the whole monitoring period and for 42% of the water from June 2014 to June 2015.

The residents often did not report leaking equipment. This less frequent reporting lead to wasteful over-use of the freely provided tap water for toilet use and hand washing. In contrast, volumetric billing, as carried out for the showers and the laundry sinks, leads to a less fluctuating water use. It seems to trigger water-saving behavior.

Referred to the mean number of persons using the facilities, the average water use was 22.5 L/(person×d) for toilets and hand washing and 10.2 L/(person×d) for laundry washing and showering (4.1 m³/d × 1000 L/m³ ÷ 30 cluster units × 3.5 households/cluster × 3.8 persons/household = 10.2 L/(person×d)).

The average water use per person was therefore much lower than the minimum recommended water use for domestic needs, which is 60 L/(user×d) (Gleick 1996; UN-HABITAT 2003b).

However, the actual number of users was unknown. If all residents used the cluster units, they either satisfied the rest of their water needs from other sources (there are two public standpipes in the vicinity) or indeed, had a very low daily water use. Another possibility would be that only a certain percentage of the residents used the cluster units, but with a higher specific water use, and the rest of the potential users did not use the cluster units at all.

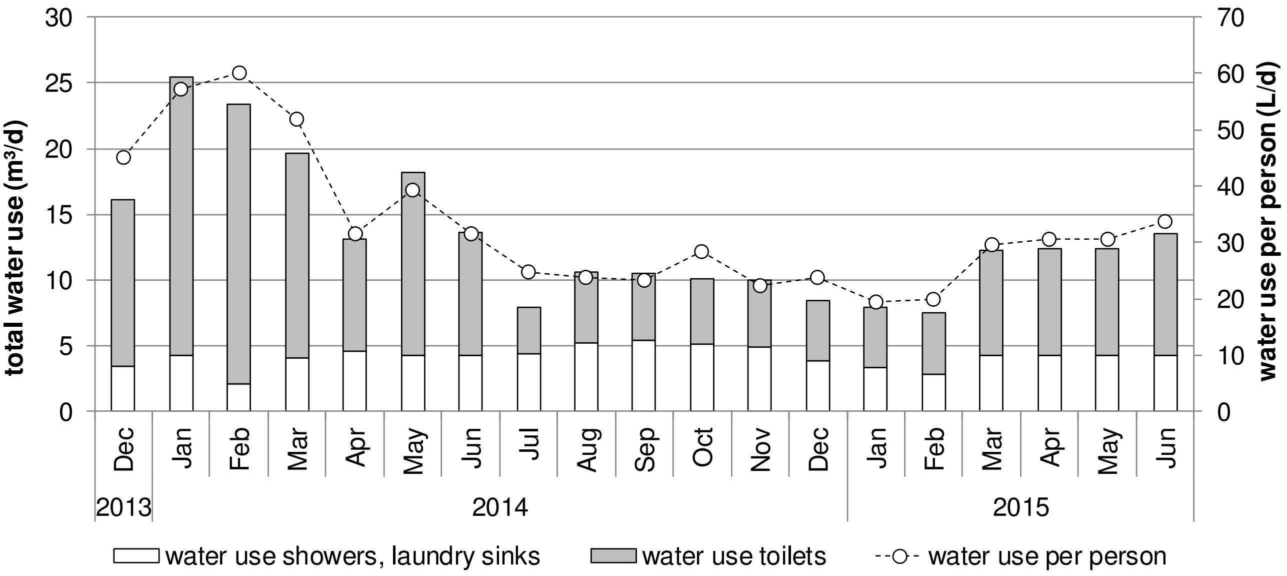

Figure 60 Total water use of the cluster units and average water use per person

It was reported (but not quantified) that some of the water provided free of charge (for toilet flushing and hand washing) was diverted for other purposes. Assuming 3 toilet visits per person per day and a volume of 5.5 L/flush and 1 L/hand washing, the required water quantity for three toilet visits per day would be 20 L/(person×d) (3 × 5.5 L/flush + 3 × 1 L/hand washing ≈ 20 L).

The monitored water use of 25.2 L/(person×d) is above this value. As will be shown later, only 23% of excreta were collected, on average (Table 17). Thus, there is a discrepancy between the water use and excreta collection. To fit to the lower excreta collection, the water demand for toilets and hand washing should be roughly around 5 L/(person×d) (= 20 L/(person×d) × 0.23). Water extraction for other than the intended purposes would then be around 20 L/(person×d) (= 25.2 L/(person×d) to 5 L/(person×d)). This amounts to 77 NAD/(person×a) or 5.4 EUR/(person×a) (tap water price in Outapi: 10.55 NAD/m³, 1 EUR ≈ 14.22 NAD, average currency exchange rate 2014, www.oanda.com) as accompanying costs for provision of free access to toilets and hand wash basins.

4.3.2.3 Water quality and loads

The loads generated by the cluster units were surveyed from November 2013 to March 2014. During the survey period, the average water use was 20.4 m³/d. Average TCOD, TDS, TN and TP concentrations measured in the influent of the wastewater treatment plant were 527, 263,

Table 17 Water quality, water quantities, utilization and loads from the communal washhouse and cluster units; survey period: November 2013 to March 2014, sd = standard deviation, n = number of values, ci = confidence interval, TCOD = total COD, TN = total nitrogen, TP = total phosphorus, TDS = total dissolved solids, EC = electrical conductivity, TSS = total suspended solids, TS = total solids

	planned	**monitored: November 2013 - March 2014**					**monitored ÷ planned**
	mean	**mean**	**sd**	**mean: 95 % ci**	**median**	**n**	
water quality	mg/L	mg/L	mg/L	mg/L	mg/L	-	%
TCOD	1,667	527	190	458 - 596	476	28	32
TN	133	38.3	10.9	34.3 - 42.3	36.2	28	29
TP	16.7	7.3	2.1	6.6 - 8.1	6.6	28	44
TDS	2,000	263	66.2	250 - 277	257	95	13
EC	-	µS/cm	µS/cm	µS/cm	µS/cm	-	-
	-	425	107	403 - 446	415	95	-
water use	m³/d	m³/d	m³/d	m³/d	m³/d	-	%
total	65.4	40.9	-	-	-	-	63
communal washhouse	15.0	20.5	5.4	19.2 - 21.8	18.3	66	137
cluster units	50.4	20.4	-	-	-	-	41
utilization	uses/d	uses/d	uses/d	uses/d	uses/d	-	%
communal washhouse	750	184	76.0	169 - 199	212	95	25
cluster units	persons	persons	persons	persons	persons	-	-
	840	404	-	-	-	-	48
total loads							
a) influent wastewater treatment plant							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	109	20.5	9.0	17.4 - 23.6	18.3	29	19
TN	8.7	1.5	0.5	1.3 - 1.6	1.3	29	17
TP	1.1	0.25	0.08	0.22 - 0.28	0.24	29	23
TDS	131	9.7	2.8	9.2 - 10.3	9.5	96	7.4
b) communal washhouse							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	25.0	8.6	3.9	7.8 -	9.1	101	34
TN	2.0	0.6	0.3	0.5 - 0.6	0.6	101	29
TP	0.3	0.13	0.06	0.12 - 0.14	0.14	101	53
TDS	30.0	6.8	3.1	6.2 - 7.4	7.2	101	23
b) cluster units							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	84.0	11.6	9.1	8.2 - 15	9.8	27	14
TN	6.7	0.9	0.6	0.7 - 1.2	0.9	28	14
TP	0.8	0.12	0.09	0.08 - 0.15	0.12	27	14
TDS	101	2.8	4.2	1.9 - 3.6	2.2	95	3
specific loads							
a) communal washhouse: see Table 16, page 100							
b) cluster units							
	g/(person×d)	g/(person×d)	g/(person×d)	g/(person×d)	g/(person×d)	-	%
TCOD	100	32.5	55.5	11.6 - 53.4	39.0	26	33
TN	8.0	2.6	3.4	1.3 - 3.9	2.7	27	32
TP	1.0	0.26	0.53	0.07 - 0.46	0.32	27	26
TDS	120	2.7	21.6	-1.7 - 7.1	4.1	93	2
TSS (estimate)	39.3	12.8	-	-	-	-	33
TS (estimate)	159	15.5	-	-	-	-	10

38.3 and 7.3 mg/L, respectively (Table 17). During that time frame, water use at the communal washhouse amounted to 20.5 (±5.4) m³/d. Thus, the total water use was 40.9 m³/d.

Total loads of both sanitation facilities were 20.5 kg/d for TCOD, 1.5 kg/d for TN, 0.25 kg/d for TP and 9.7 kg/d for TDS. Loads from the communal washhouse were estimated using the previously determined loads per use (Table 16, page 100). They were deducted from the total loads. The mean TCOD, TN, TP and TDS loads contributed by the cluster units were 11.6, 0.9, 0.12 and 2.8 kg/d, respectively (Table 17). This corresponds to only 14% of the planning data.

The specific loads were 32.5 g TCOD/(person×d), 2.6 g TN/(person×d), 0.26 g TP/(person×d) and 2.7 g TDS/(person×d) (excluding the TDS content of the tap water). Compared to planning data, this was between 26% and 33% of the expected specific TN, TP and TCOD loads and 2% of the expected specific TDS load. The specific TSS and TS loads were estimated as described in Section 4.3.1.4 (page 100). They were 12.8 g TSS/(person×d) and 15.5 g TS/(person×d).

The lower total loads were caused by the much lower number of persons per household (3.8 persons per household, instead of 7, as anticipated during planning) but also due to the lower percentage of collected loads per person. As in the case of the communal washhouse, the very low specific TDS load indicates that the cluster units were used to a minor degree for urination. Even when the lower uptake of common salt in Namibia (27% lower than in Germany and the USA, Powles *et al.* (2013), see page 101) is taken into account, the adjusted TDS collection rate would be 3% and, therefore, also insignificant.

4.3.3 Individual households

47 households were connected to the vacuum sewer system by August 2015 (planned: 66 households, Table 7, page 54). The average total water use per day increased from April 2014 until October 2014 (Figure 61) and remained constant thereafter. Since October 2014, the average water use was 9.0 m³/d. During the whole survey period, the average water use per household was 227 L/d. The average household size was 3.7 persons (Kramm and Deffner 2017) compared to 4 persons as anticipated during planning (Table 7, page 54); thus, the average water use was 61 L/(person×d). This fits to the water use of 60 L/(person×d) that was assumed during planning.

The data basis for estimation of TCOD, TN, TP and TDS loads from the individual households is shown in Table 18. The total loads from the individual households are the total loads in the influent of the wastewater treatment plant minus the estimated loads from the communal washhouse (estimated via the number of uses and the specific loads per use, Table 16, page 100) minus the estimated loads from the cluster units (estimated by the previously determined mean load per cluster unit, Table 17).

Table 18 Water quality, water quantities, utilization and loads from the communal washhouse, cluster units and individual households; survey period: April 2014 to August 2015, sd = standard deviation, n = number of values, ci = confidence interval, TCOD = total COD, TN = total nitrogen, TP = total phosphorus, TDS = total dissolved solids, EC = electrical conductivity, TSS = total suspended solids, TS = total solids

	planned	**monitored: April 2014 - August 2015**					**monitored ÷ planned**
	mean	**mean**	**sd**	**mean: 95 % ci**	**median**	**n**	
water quality	mg/L	mg/L	mg/L	mg/L	mg/L	-	%
TCOD	1,667	928	340	842 - 1015	807	58	56
TN	133	75.1	20.9	69.8 - 80.4	72.8	59	56
TP	16.7	12.2	2.6	11.5 - 12.8	12.4	60	73
TDS	2,000	435	94.4	422 - 447	435	210	22
EC	-	µS/cm	µS/cm	µS/cm	µS/cm	-	-
	-	701	152	680 - 722	701	210	-
water use	m³/d	m³/d	m³/d	m³/d	m³/d	-	%
total	81.2	33.7	-	-	-	-	42
communal washhouse	15.0	15.5	5.0	14.9 - 16.2	14.7	226	104
cluster units	50.4	10.6	-	-	-	-	21
individual households	15.8	7.6					48
utilization	uses/d	uses/d	uses/d	uses/d	uses/d	-	%
communal washhouse	750	146	69.8	138 - 154	121	318	19
	persons	persons (final)	persons	persons	persons		%
cluster units	840	404	-	-	-	-	48
individual households	264	173	-	-	-	-	66
total loads							
a) influent wastewater treatment plant							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	135	32.7	12.6	29.4 - 35.9	30.6	58	24
TN	11	2.6	0.8	2.4 - 2.8	2.5	59	24
TP	1.4	0.43	0.11	0.4 - 0.46	0.43	60	32
TDS	162	14.8	3.8	14.3 - 15.3	14.6	210	9.1
b) communal washhouse							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	25.0	8.6	3.9	7.8 - 9.3	9.1	101	34
TN	2.0	0.6	0.3	0.5 - 0.6	0.6	101	29
TP	0.3	0.13	0.06	0.12 - 0.14	0.14	101	53
TDS	30.0	6.8	3.1	6.2 - 7.4	7.2	101	23
c) cluster units							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	84.0	11.6	9.1	8.2 - 15	9.8	27	14
TN	6.7	0.9	0.6	0.7 - 1.2	0.9	28	14
TP	0.8	0.12	0.09	0.08 - 0.15	0.12	27	14
TDS	101	2.8	4.2	1.9 - 3.6	2.2	95	2.7
d) individual households							
	kg/d	kg/d	kg/d	kg/d	kg/d	-	%
TCOD	26.4	9.0	11.5	6 - 11.9	7.9	57	34
TN	2.1	0.8	0.6	0.7 - 1	0.8	56	39
TP	0.3	0.2	0.1	0.13 - 0.18	0.2	57	59
TDS	31.7	4.1	3.7	3.5 - 4.7	4.1	203	13

Table 18 (continued)

	planned	**monitored: April 2014 - August 2015**					**monitored ÷ planned**
	mean	**mean**	**sd**	**mean: 95 % ci**	**median**	**n**	
specific loads							
a) communal washhouse: see Table 16, page 100							
b) cluster units: Table 17, page 104							
c) individual households							
	g/(person×d)	g/(person×d)	g/(person×d)	g/(person×d)	g/(person×d)	-	%
TCOD	100	74.8	89.0	50.1 - 99.5	68.5	49	75
TN	8.0	6.1	6.3	4.3 - 7.9	6.4	47	76
TP	1.0	1.4	0.9	1.16 - 1.63	1.3	50	140
TDS	120	38.5	27.4	34.5 - 42.5	38.1	181	32
TSS (estimate)	39.3	29.4	-	-	-	-	75
TS (estimate)	159	67.9	-	-	-	-	43

The mean loads per individual household were calculated as 276 g TCOD/d, 22.5 g TN/d, 5.2 g TP/d, and 142 g TDS/d. The amount of collected excreta per person was 74.8 g COD/d, 6.1 g N/d, 1.4 g P/d and 38.5 g TDS/d. The specific TSS and TS loads were estimated as described on in Section 4.3.1.4 (page 100). They were 29.4 g TSS/(person×d) and 67.9 g TS/(person×d).

Compared to literature data, the excreted amounts represented collection rates of 75% for TCOD, 76% for TN, 140% for TP and 32% for TDS. The percentage of the collected loads will always be lower than 100% because the residents spend some of their time outside their homes and thus outside the catchment area of the implemented infrastructure. Hence, the system size needs to be considered during planning. The high percentage of collected TP is due to phosphates contained in detergents. TDS were collected to a much lower extent than TCOD and TN. This finding suggests that toilets were mainly used for defecating and less often for urinating.

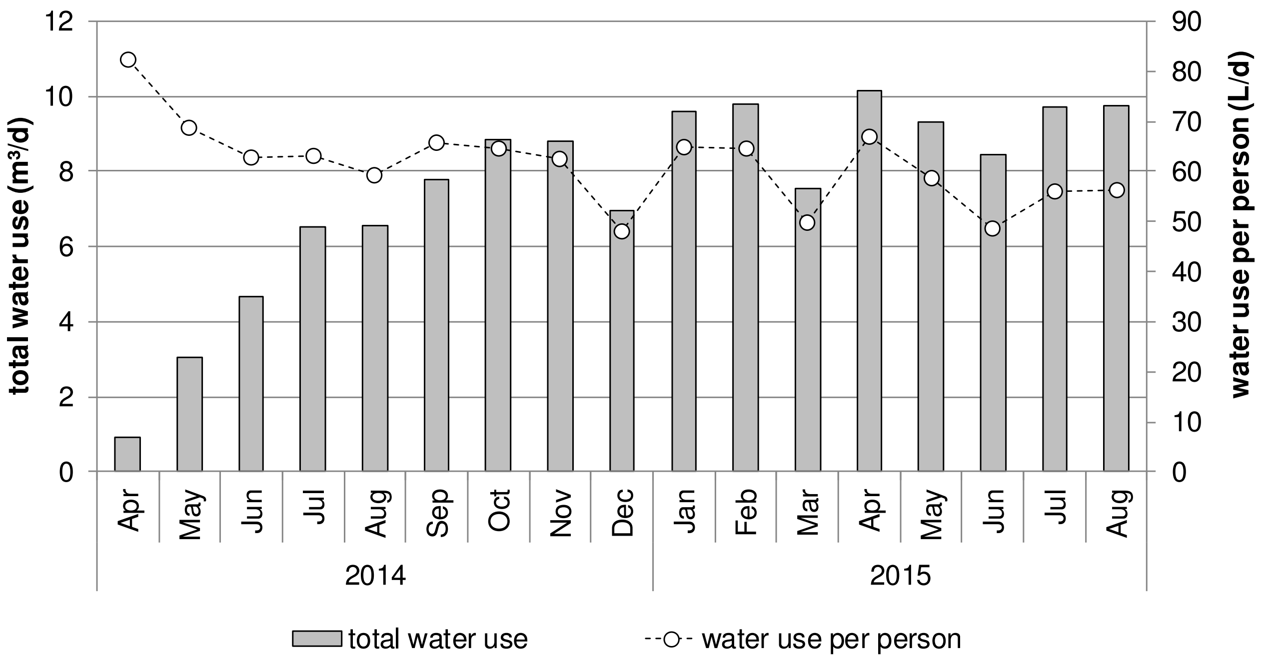

Figure 61 Total and specific average water use of the individual households

If a specific TDS load of 86.4 g/(person×d) is assumed (due to lower common salt content of the local diet, Powles *et al.* (2013), see page 101), instead of 120 g/(person×d), the TDS collection rate would be 45%. Thus, in this case, the gap between TN and TDS collection rates cannot be adequately explained by the lower salt intake. Perhaps, additional N loads are added to the wastewater (e.g., by using the toilets as a form of garbage disposal). Maybe the gap between the TN and TDS collection rate is biased, due to the assumptions made during estimation of loads from the communal washhouse and the cluster units.

4.3.4 Comparison: communal washhouse, cluster units and households with individual connection

4.3.4.1 Uses and utilization

Table 19 presents an overview of the number of users and uses per day, as anticipated during planning, and the monitored number of users and uses per day. The communal washhouse was designed for 250 users and 750 uses per day. However, a facility for 100 to 200 uses per day would have been sufficient in this low-density urban area.

The number of households using the cluster units turned out to be lower than assumed during planning (106 instead of 120 households). However, the lower average household size (3.8 instead of 7 persons) resulted in a considerable decrease of potential users (404 instead of 840).

Until the end of the project period, 71% of the initially planned number of individual households was connected to the vacuum sewer system. The remaining households were not connected, mainly due to financial constraints. The size of the connected households was lower than estimated during planning (3.7 instead of 4 persons per household).

Table 19 Overview on the planned and monitored number of users and utilization

	planned				**monitored**		
	communal washhouse	cluster units	individual households	safety margin	communal washhouse	cluster units	individual households
persons	250	840	264	146	-	404	173
uses/d	750	2,520	792	438	166	-	-
persons/household	-	7.0	4.0	-	3.3	3.8	3.7
households	-	120	66	-	-	106	47

4.3.4.2 Water use

The same water use was assumed for all users during planning (60 L/(person×d)). The monitored specific water use differed among the sanitation facilities (Table 20). The lowest specific water use was monitored at the cluster units (32.8 L/(person×d)). The water use in the individual households was 61.3 L/(person×d). The highest water use occurred at the communal washhouse. The average water use was 101 L/use. Regarding the loads, one utilization of the communal washhouse corresponded to, on average, 0.4 user equivalents. The water use per user equivalent would then be 253 L (= 101 L/use ÷ 0.4 user equivalents/use).

Table 20 Overview of the total and the specific water use assumed for planning and after implementation

	planned					monitored			
	total	communal washhouse	cluster units	individual households	safety margin	total	communal washhouse	cluster units	individual households
m³/d	90.0	15.0	50.4	15.8	8.8	30.3	16.8	13.2	7.6
L/(person×d)	60.0	60.0	60.0	60.0	-	-	-	32.8	61.3
L/use	20.0	20.0	20.0	20.0	-	-	101	-	-

4.3.4.3 Concentrations

During planning, the same concentrations were assumed for each type of sanitation facility. The monitored concentrations in the influent of the wastewater treatment plant reached between 19% and 62% of the planning data (Table 21). The determined specific loads and the specific water use were used to estimate the concentrations in the untreated wastewater of each sanitation facility. They differed between the sanitation facilities. According to these estimates, the wastewater from the communal washhouse had the lowest TCOD, TSS, TN and TP concentrations. The water from the cluster units had the lowest TDS and TS concentrations. The wastewater from the individual households was the one with the highest concentration for each constituent. In general, TP concentrations were closer to the planning data than the other parameters. It even exceeded the estimated concentration for the individual households.

Table 21 Wastewater characteristics of the untreated wastewater from all settlements (column: 'total') and for each kind of sanitation facility (based on the specific loads and the specific water use), TCOD = total COD, TN = total nitrogen, TP = total phosphorus, TDS = total dissolved solids, EC = electrical conductivity, TSS = total suspended solids, TS = total solids

	planned				monitored	estimated with monitoring data		
	total	communal washhouse	cluster units	individual households	total	communal washhouse	cluster units	individual households
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
TCOD	1,667	1,667	1,667	1,667	738	457	877	1,185
TN	133	133	133	133	57.5	30.4	71.8	109.3
TP	16.7	16.7	16.7	16.7	10.3	7.0	8.9	20.5
TDS	2,000	2,000	2,000	2,000	375	359	209	546
EC	3,226	3,226	3,226	3,226	617	579	337	881
TSS	1,000	1,000	1,000	1,000	-	267	389	479
TS	3,000	3,000	3,000	3,000	1,040	585	471	1,108

4.3.4.4 Loads

The total loads were much lower, after implementation, than assumed during planning (Table 22). They reached only 22% (TP), 17% (TCOD), 16% (TN) and 7% (TDS) of the planning data. The lower total loads, compared to planning data, can be explained by the lower number of persons using the sanitation system during the survey period and incomplete excreta collection in the sanitation facilities.

The extent of the collected proportions varies among the wastewater parameters examined. TP is the constituent with the highest collected proportion. At the communal washhouse and the individual households, its collected amount was higher than for the other parameters (47% and

59%, entire monitoring period). This suggests that laundry washing was a very important activity at the communal washhouse and the individual households.

TP, TN and TCOD loads were collected to the same extent at the cluster units (14% of planning data). This indicates that laundry washing and defecation were practiced more or less to the same degree. Altogether, the cluster units were used to a lesser degree than the communal washhouse and the individual sanitation facilities.

For all monitored sanitation facilities, TDS had the lowest collection rate. This was even the case for the individual households. TDS collection at the cluster units was so low that it was virtually insignificant.

Table 22 Planned and monitored total and specific loads; TCOD = total COD, TN = total nitrogen, TP = total phosphorus, TDS = total dissolved solids, EC = electrical conductivity, TSS = total suspended solids, TS = total solids

	planned					**monitored**			
	total	communal washhouse	cluster units	individual households	safety margin	total	communal washhouse	cluster units	individual households
loads									
a) total									
	kg/d	kg/d	kg/d	kg/d	kg/d	kg/d	kg/d	kg/d	kg/d
TCOD	150	25.0	84.0	26.4	14.6	25.1	7.7	11.6	9.0
TN	12.0	2.0	6.7	2.1	1.2	1.9	0.5	0.9	0.8
TP	1.5	0.3	0.8	0.3	0.1	0.33	0.12	0.12	0.15
TDS	180	30.0	101	31.7	17.5	12.0	6.0	2.8	4.1
b) specific									
	g/(person×d)	g/use	g/(person×d)	g/(person×d)	g/(person×d)	-	g/use	g/(person×d)	
TCOD	100	33.3	100	100	100	-	45.0	32.5	74.8
TN	8.0	2.7	8.0	8.0	8.0	-	3.0	2.6	6.1
TP	1.0	0.3	1.0	1.0	1.0	-	0.69	0.26	1.40
TDS	120	40.0	120	120	120	-	32.0	2.7	38.5
TSS	60.0	20.0	60.0	60.0	60.0	-	27.0	12.8	29.4
TS	180	60.0	180	180	180	-	59.0	15.5	67.9

4.3.5 Conclusions

This chapter provided long term monitoring data on water use, utilization rates and loads of shared and individual sanitation facilities. No such data have been analyzed elsewhere. The provided figures can be used for better estimation of utilization and wastewater characteristics when planning comparable projects. Whereas the assumptions regarding the individual households approximately met planning values, this was not the case for the shared sanitation facilities.

Altogether, planning considerably overestimated the future number of users and future utilization, especially of the shared sanitation facilities. The reasons for this are the dynamic population development in areas with non-permanent housing structures (Kramm and Deffner 2017). This shortcoming affected many aspects of the project. It led to oversizing of the sanitation facilities, sewers and wastewater treatment steps, lower quantities of available irrigation water and nutrients for fertilization, lower agricultural revenues, lower amounts of sewage sludge and agricultural residues for energetic utilization, higher specific capital and operation and maintenance costs, all of which caused difficulties in terms of the development of a socially acceptable tariff system.

The water use of shared sanitation facilities was strongly influenced by tariffs and the kind of billing (pay-per-use or volumetric). In this case, water use was lower and less variable for volumetric billing modes (individual households, showers and laundry sinks in the cluster units) and higher for non-volumetric billing modes (communal washhouse, toilets and hand wash basins in the cluster units). It is recommended to control the specific water use for services offered on a pay-per-use basis. This could be achieved via regular inspections and supervision of the users. Because utilization rates depend on the population density, population density also influences the total water use. Hence, the population density is an important parameter to be considered during the planning of shared sanitation.

If water and sanitation services are to be provided for the majority of the residents in the settlement, tariffs need to be low. As a result, total utilization rates and wastewater quantities will be relatively high. The introduction of volumetric billing or other control measures should be considered to control specific water uses. Capital costs for, e.g., prepaid water meters can be relatively high, compared to the water quantities billed for. This needs to be considered before implementation.

It was found that substances mainly contained in feces and laundry detergents (TCOD, TP) tend to have higher collection rates than substances mainly found in urine (TN, TDS). Urination in the public was still widely practiced, even when toilets were available free of charge in the vicinity (cluster units) or on the household level (individual households). This was observed not only in this study but also in a study on shared sanitation facilities in South Africa (see Section 4.3.1.1, page 94, Crous *et al.* (2013)). Hence, access to toilets needs to be made very easy, in order to compete with the convenience of urination in public. In practice, this is very difficult to achieve. The design of the facilities was discussed with the residents during several workshops and adapted accordingly (Deffner and Mazambani 2010). Hence, it can be assumed that the technical layout is satisfactory. Awareness raising among community members is required to augment utilization rates in general and defecation visits in particular. Additional briefings of the users are recommended to assist in increasing toilet use. Further studies should focus on other possible barriers to the use of the sanitation facilities (e.g., habits, behavior patterns).

The concentrations of the wastewater constituents differed among the sanitation facilities. A major influencing factor was the kind of billing mode applied (volumetric or flat rate). Because it influenced the specific water quantities, it also affected the concentrations. Flat rate billing systems lead to relatively high quantities of weak to medium concentrated wastewater. Volumetric billing leads to relatively low quantities of more concentrated wastewater. The differing strengths and volumes of the wastewater flows should be considered when choosing subsequent transport and treatment and reuse steps.

Based on these expected concentrations, several recommendations can be made for wastewater treatment and water reuse. First of all, none of the wastewater flows reached the suggested minimum TCOD concentration required for satisfactory methane recovery from anaerobic wastewater treatment. Even the wastewater from the individual households, which had the highest concentration, was below the recommended minimum range of 1,500 to 2,000 mg/L TCOD (Tchobanoglous *et al.* 2004). The assumed concentrations were only slightly above the range suggested by Tchobanoglous *et al.* (2004). In retrospect, this safety margin was not large enough.

Due to the relatively high proportion of greywater and the determined percentages of collected excreta, water from shared sanitation facilities will usually be too diluted for sensible application of anaerobic wastewater treatment. Instead, aerobic treatment steps should be chosen.

When only individual households are connected, TCOD concentrations can be expected to be much higher. Depending on the specific water use and hygiene practices, concentrations could be high enough for recovery of methane via anaerobic wastewater treatment in some locations. This needs to be assessed on a case-by-case basis.

The relatively high expected TP and TN concentrations and the expected high EC of the untreated water were a major concern during planning, due to the risks of overfertilization, eutrophication and salinization on the agricultural fields. Nutrients should not be removed from the water, in order to be available as fertilizers. But high concentrations of nutrients and salts in the irrigation water can cause yield loss and soil salinization.

After implementation, monitored EC, TP and TN concentrations were much lower than expected. This rendered the water more suitable for agricultural irrigation, because the anticipated risks were much lower and a greater variety of crops could be cultivated because the choice was not limited to salt-resistant species. Crops with a higher salt sensitivity could also be cultivated. A detailed discussion of water quality criteria and salt and nutrient management is provided in Chapter 4.5 (page 122*ff.*) and Chapter 4.6 (page 137*ff.*).

Prior work on shared sanitation in informal settlements acknowledges the contribution they can make for sanitation provision, but mainly focused on general feasibility, institutional and sociological topics, and appropriateness. Data on wastewater characteristics or utilization rates from such facilities has not been published. The results obtained from this study thus contribute to fill this gap and, furthermore, provide information for improved facilitation of shared sanitation in informal settlements. This is particularly valuable for better planning of the general layout of these facilities and planning of subsequent wastewater transport, treatment and reuse.

4.4 Costs of shared sanitation facilities

Achieving long-term sustainability, especially financial sustainability, is difficult in informal settlements. Even quite successful concepts, such as the Ikotoilet approach in Kenya or India's Sulabh toilets, struggle to generate sufficient revenues in such areas (Heierli 2004; Ziegler *et al.* 2013). Of 40 locations with Ikotoilets, only two are located in informal settlements ("a story of difficulty") (Ziegler *et al.* 2013).

To operate the implemented shared sanitation facilities, a tariff and management system that fitted the requirements and capacities of the OTC had to be developed. Funding for capital costs is required only once. Costs for operation and maintenance, reinvestment, interest and depreciation occur regularly over the whole life span of the infrastructure. Generation of regular and sufficient revenues is required for long-term financial sustainability. Because municipalities almost always act under financial constraints, this chapter deals with the possibilities for covering operation and maintenance costs of the communal washhouse and the cluster units via tariffs, and under which conditions full coverage would be achievable. An overall economic evaluation, with cost calculations, benefit considerations, and financing prospects of the whole sanitation system including vacuum sewer system, wastewater treatment plant and irrigation site was prepared by IEEM (2015) and Zimmermann *et al.* (2017a).

4.4.1 Operation and maintenance costs

4.4.1.1 Communal washhouse

Costs for operation and maintenance of the communal washhouse included costs for tap water (10.55 NAD/m³ = 0.73 EUR/m³, 1 EUR ≈ 14.38 NAD, average currency exchange rate 2014, www.oanda.com), cleaners and caretakers (350 NAD/d = 24.34 EUR/d), spare parts (15 NAD/d = 1.04 EUR/d), toilet paper and cleaning utensils (48 NAD/d = 3.34 EUR/d), and electricity for lighting (15 NAD/d = 1.04 EUR/d). Hence, fixed costs amount to 428 NAD/d. Flexible costs include tap water costs and depend on the number of uses per day and the water use per use. The question is, under which conditions the generated revenues would be sufficient to cover these operation and maintenance costs.

Figure 62 illustrates the break-even utilization and water use for recovery of all o&m costs. For instance, for a water use of 15 m³/d, the minimum utilization for recovery of all o&m costs is about 1200 uses/d for a tariff of 0.5 NAD/use and about 300 uses/d for a tariff of 2 NAD/use. In the Outapi case, the water use ranged from 10 to 24 m³/d. The monitored utilization was too low to allow cost recovery.

In theory, much higher utilization rates could be met. Within a perimeter of 200 m and 500 m around the washhouse, there should already be a sufficient number of residents for a potential utilization of 815 uses/d and 5,096 uses/d (Table 14, page 97). However, considering the economic background of the residents, the estimated possible maximum number is 189 uses/d for a tariff of 2 NAD/use and 756 uses/d for a tariff of 0.5 NAD/use (for a catchment area with a radius of 500 m, Table 14, page 97).

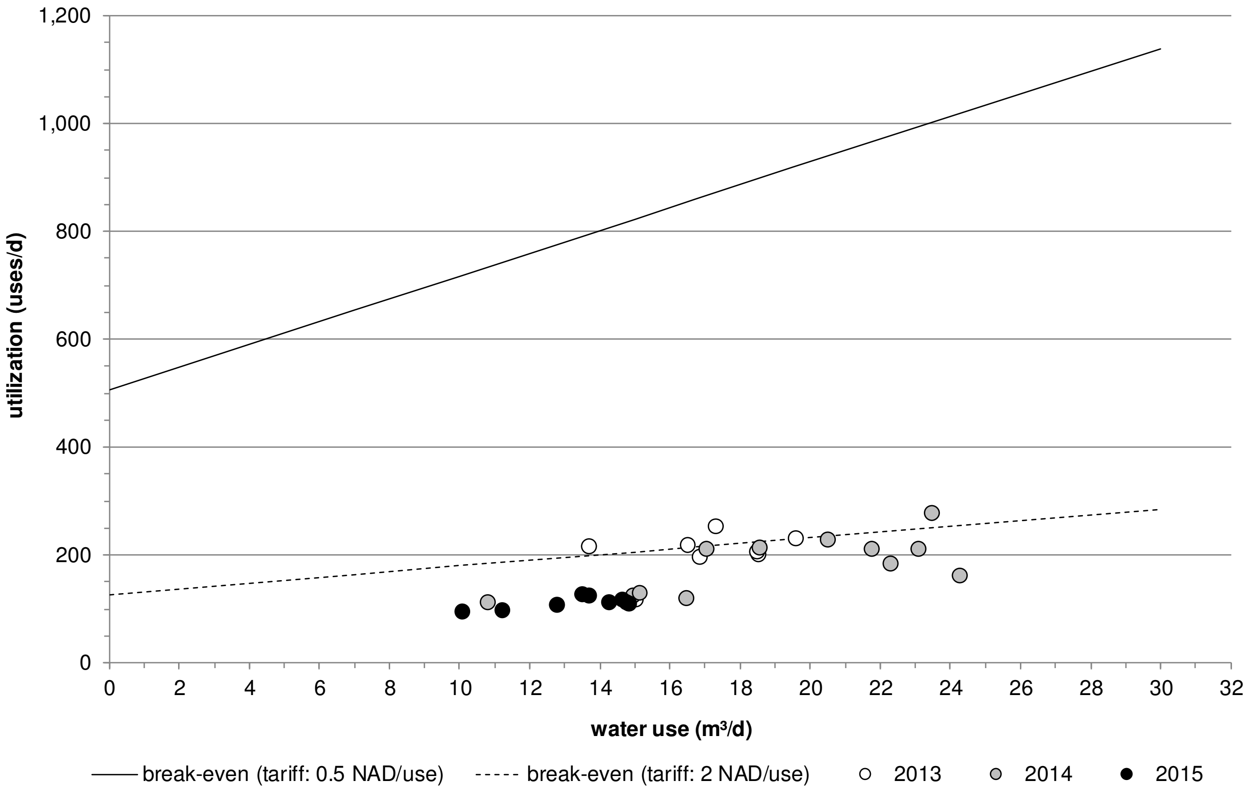

Figure 62 Communal washhouse: break-even water use and utilization required for recovery of full o&m costs at the for a tariff of 2 NAD/use (solid line), and for a tariff of 0.5 NAD/use (dashed line)

Obviously, the desired utilization rates for o&m cost recovery cannot be reached at the suggested tariffs and for the relatively low population density in Outapi. Hence, in the Outapi case, o&m cost recovery of the communal washhouse is very difficult.

The o&m costs need to be reduced if they are to be covered via tariffs. Personnel costs account for more than 80% of the fixed costs and thus bear the greatest potential for cost reduction. When halving staff expenses, break-even points for a water use of 15 m³/d would be 823 uses/d (0.5 NAD/use) and 206 uses/d (2 NAD/use). However, for the lower tariff, the specific water use needs to be kept at a low level (17.6 L/use).

A 50% reduction in personnel costs could be achieved by voluntary work of the community members or adjusting the opening hours. Because more than 70% of toilet and shower use occurs in the morning (6 am to 10 am) or in the late afternoon and evening (4:00 pm to 7:00 pm) (Figure 58), the washhouse could be closed around noon to save expenses for cleaning and caretaking. Whether this is feasible needs to be discussed with the community and the local authority. In any case, promotion of the facilities among residents is necessary to obtain higher utilization.

When utilization is to low, subsidies are required for cost recovery. In this case, subsidies stem from the inclusion of economically stronger town areas into the sanitation system and the sale of reclaimed water to local farmers. The reclaimed water is sold at a price of 8 NAD/m³ to the operator of the irrigation site (Zimmermann *et al.* 2017b). If this revenue flows back to the communal washhouse, the tariff could be kept at between 1 and 2 NAD/use for a minimum utilization of 140 and 150 uses/d (depending on the specific water use). This example shows the importance of considering the entire sanitation system and using synergetic effects between the single components in order to maximize the benefits of the whole system.

Other sources of subsidies are possible: rents from community-meeting rooms (Norman 2011) or shops and outdoor advertising (Njeru 2014; Norman 2011; Ziegler *et al.* 2013). Such features need to be included into the general layout of the sanitation facility during planning to make sure that additional revenues can be generated.

The surroundings of the sanitation facility in Outapi were characterized by a high proportion of undeveloped areas (roughly 50%, Figure 20, page 41). A more favorable location of the sanitation facility in an area with a higher percentage of developed plots and a higher population density would certainly lead to a higher utilization of the communal washhouse and higher revenue generation.

As outlined previously, for 2 NAD/use, the utilization would need to be five times higher to achieve o&m cost recovery. Assuming that utilization rates and population density increase equally, the required minimum population density would be ≈ 65 persons/ha. The mean population density in African cities is 86 persons/ha (UN-HABITAT 2013). Thus, for higher user fees, an average African city has the required population density at its disposal to achieve o&m cost recovery. For lower user fees, shared sanitation facilities are expected to recover o&m costs, without subsidies, only in very densely populated areas.

4.4.1.2 Cluster units

The cluster units were operated and maintained by the allocated households. For the OTC, running costs were limited to tap water fees (10.55 NAD/m³ or 0.73 EUR/m³, average currency exchange rate 2014 = 14.38 NAD/EUR, www.oanda.com) and 68.5 NAD/d (= 4.76 EUR/d) for maintenance and spare parts for all cluster units. This corresponds to 62.04 NAD/(person×a). Access to the toilets and hand wash basins was provided free of charge. Water for showering and laundry washing was sold at a higher fee of 30 NAD/m³. The objective was to cover all costs with the revenues from water use for laundry washing and showering.

During the project period, expenses for tap water were 126 NAD/(person×a) (32.8 L/(person×d) × 10.55 NAD/m³ × 365 d/a = 9.14 EUR/(person×a), see Table 20, page 109 for water quantities). Total expenses for tap water and o&m were 188 NAD per person and year (= 62.04 NAD/(person×a) + 126 NAD/(person×a) = 14.05 EUR/(person×a)). Revenues were 112 NAD/(person×a) (= 10.2 L/(person×d) × 30 NAD/m³ × 365 d/a = 8.76 EUR/(person×a)). Thus, with the current water use pattern, o&m costs could not be recovered.

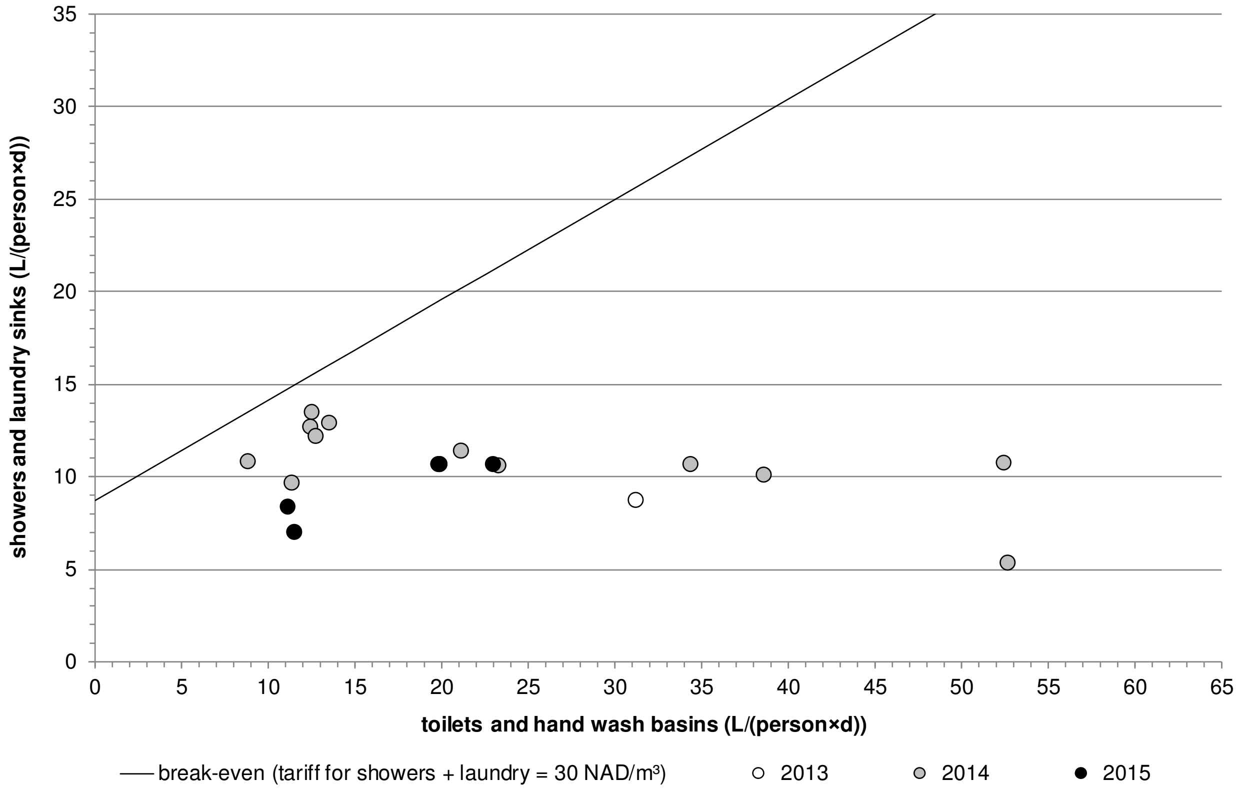

Figure 63 Cluster units: break-even water volumes for laundry washing/showering and toilet flushing/hand washing for recovery of full o&m costs

The question is, for which utilization pattern a break-even for o&m costs could be achieved. Figure 63 illustrates the break-even water volumes for laundry washing/showering and toilets/hand washing. The monitored monthly average water uses for showers and laundry sinks and for toilets and hand wash basins did not allow o&m cost recovery. For the observed average water use for toilet use and hand washing of 22.5 L/(person×d), showers and laundry washing water use needed to be at least 21 L/(person×d) to ensure o&m cost recovery. Conversely, for the observed water use for showering and laundry washing of 10.2 L/(person×d), the toilet and hand wash water use must not exceed 3 L/(person×d). The applied tariff should be adjusted to fit the observed use pattern. Considering the monitored average water use pattern, the tariff for showers and laundry sinks would need to be set at 50 NAD/m³ for recovery of o&m costs.

4.4.2 Capital costs

Construction costs were 1,264,728 NAD for the communal washhouse (= 121,608 EUR incl. value added tax, 10.40 NAD/EUR, average currency exchange rate during construction phase in 2012, www.oanda.com) and 64,236 NAD for one cluster unit (= 6,177 EUR, incl. value added tax, 10.40 NAD/EUR). Installation of the prepaid water meters caused further expenses amounting to 7,498 NAD per cluster unit (= 721 EUR, incl. value added tax, see Section 4.1.2, page 56, average currency exchange rate: 10.40 NAD/EUR). Construction was carried out by a Namibian civil construction company on behalf of the OTC, following a formal tender invitation throughout Namibia. The construction costs presented here exclude costs for planning and project management. Depending on the structure of the project, such costs will be an extra 20% to 50% of the capital costs (Still *et al.* 2009).

Table 23 Construction costs for shared sanitation facilities in Namibia, South Africa, Madagascar and Mozambique, average currency exchange rates (www.oanda.com): NAD/EUR = 10.40 (average in 2012), ZAR/EUR = 10.49 (average 2009-2012, construction period given in Crous (2014)), USD/EUR = 0.74 (average in 2010, year before publication))

	this study (washhouse)	**this study** (cluster unit)	**Crous (2014)**	**Norman (2011)**	**Norman (2011)**	
country	Namibia	Namibia	South Africa	Madagascar	Mozambique	
construction costs	1,264,728 NAD 121,608 EUR	64,236 NAD 6,177 EUR	764,513 ZAR 72,880 EUR	27,000 USD 19,980 EUR	6,427 USD 4,756 EUR	4,463 USD 3,303 EUR
number of toilets	14	1	7	6	4	2
showers or bathrooms	25	1	4	2	4	2
number of laundry washing facilities	11	1	4	no data	1	1
uses per day	166	no data	no data	220	173	60
costs per toilet seat	8,686	6,177 EUR	10,411 EUR	3,330 EUR	1,189 EUR	1,651 EUR
costs per use and day	733 EUR	no data	no data	91 EUR	27 EUR	55 EUR

Table 24 Construction costs for shared sanitation facilities in Kenya

	Schouten and Mathenge (2010)				
country	Kenya	Kenya	Kenya	Kenya	Kenya
construction costs	15,000 EUR	18,000 EUR	26,000 EUR	9,000 EUR	13,000 EUR
number of toilets	5	8	9	6	4
showers or bathrooms	5	6	5	4	4
number of laundry washing facilities	no data	no data	no data	no data	no data
uses per day	600	450	575	110	50
costs per toilet seat	3,000 EUR	2,250 EUR	2,889 EUR	1,500 EUR	3,250 EUR
costs per use and day	25 EUR	40 EUR	45 EUR	82 EUR	260 EUR

Costs were compared with values reported in the literature. Table 23 and Table 24 summarize costs for communal sanitation facilities in South Africa, Kenya, Madagascar and Mozambique. Total construction costs range between 3,303 EUR and 72,880 EUR per facility.

For comparison, costs were referred to each provided toilet seat and to daily utilization. In the literature, costs were 10,411 EUR per toilet seat in South Africa (Crous 2014), between 1,189 and 3,330 EUR per toilet seat in Madagascar and Mozambique (Norman 2011) and 1,500 to

3,250 EUR per toilet seat for various types of toilets in Kenya (including VIP, pour flush and flush toilets) (Schouten and Mathenge 2010).

The costs per toilet seat were 8,686 EUR (communal washhouse) and 5,050 EUR (cluster units). This is surprising, because one would expect that the communal washhouse generates lower specific costs than the construction of the single cluster units. In this specific case, the higher costs for the communal washhouse arose from higher expenses for concrete work, structural steelwork and cladding.

The costs per use per day range from 25 to 260 EUR for Kenya, Madagascar and Mozambique. This value is 648 EUR for Outapi and thus much higher, due to the higher capital costs and relatively low utilization.

Construction costs in Namibia are relatively high but lower than those reported for South Africa. Table 23 and Table 24 show the great variation in construction costs among the presented examples. The costs are certainly influenced by the design of the facilities, but they also reflect different price levels in the regions. In Namibia and South Africa, price levels are 40% above the African average and in Kenya and Madagascar, price levels are generally 20% to 30% lower than the African average (African Development Bank Group 2014). In general, price levels in Southern Africa tend to be higher than for the rest of the continent (Angola: +60%, Mozambique: +25%, Botswana: +20%, Zambia: +10%, Zimbabwe: +10%, African Development Bank Group (2014)).

The literature also provides figures for costs of facilities providing solely toilets. Construction of urine-diverting dry toilets in Namibia costs about 800 EUR (Kleeman and Berdau 2011) and about 700 EUR in South Africa (Lüthi *et al.* 2011b; Starkl *et al.* 2011; Still *et al.* 2009). Capital costs for flush toilets are around 750 EUR (Starkl *et al.* 2011; Still *et al.* 2009). Costs for ventilated improved pit latrines in South Africa are lower: around 400 EUR (Starkl *et al.* 2011; Still *et al.* 2009). These costs are much lower than the construction costs of shared sanitation facilities. However, full provision of water and sanitation facilities has to include hand wash basins, showers and facilities for laundry washing. Provision of these services increases costs.

4.4.3 Cost comparison: communal washhouse and cluster units

Table 25 compares capital costs and o&m costs of the communal washhouse and one cluster unit. Capital costs of the cluster unit include the costs for one prepaid water meter. The utilization rates and TCOD loads presented in Chapter 4.3.1 and Chapter 4.3.2 were used for easier comparison of both facilities. Monitored values and planning data were considered.

The construction costs of the communal washhouse were 18 times higher than for one smaller cluster unit. Referred to the construction costs per toilet seat, costs for the communal washhouse were 1.3 times higher.

Table 25 Comparison of capital and o&m costs for the communal washhouse and one cluster unit, as monitored and as planned; the costs for the prepaid water meter are included in the costs of the cluster unit (= 64,236 NAD/cluster unit + 7,498 NAD/prepaid water meter). O&m costs: average currency exchange rate during operation in 2014: 14.38 NAD/EUR (www.oanda.com), capital costs: average currency exchange rate during construction in 2012: 10.40 NAD/EUR (www.oanda.com), assumptions: 3 uses/(person×d)

	communal washhouse		cluster unit	
	planned	monitored	planned	monitored
utilization				
persons	250	-	28.0	13.5
uses/d	750	166	84.0	-
persons/household	-	3.3	7.0	3.8
households/cluster unit	-	-	4.0	3.5
water use				
m^3/d	15.0	16.8	1.7	0.4
L/(person×d)	60.0	-	60.0	32.8
L/use	20.0	101	20.0	-
TCOD loads				
g/(person×d)	100	100	100	100
kg/d	25.0	7.7	2.8	0.4
capital costs				
total costs	1,264,728 NAD 121,608 EUR		71,734 NAD 6,898 EUR	
costs/toilet seat	8,686 EUR		6,898 EUR	
costs/(use/day)	162 EUR	733 EUR	82 EUR	-
costs/person	486 EUR	-	246 EUR	512 EUR
costs/kg TCOD	4,864 EUR	15,865 EUR	2,463 EUR	17,838 EUR
o&m: flexible costs (tap water)				
per year	57,761 NAD 4,017 EUR	64,614 NAD 4,493 EUR	6,469 NAD 450 EUR	1,699 NAD 118 EUR
costs/((use/d)/a)	5 EUR	27 EUR	5 EUR	-
costs/(person×a)	16 EUR	-	16 EUR	9 EUR
costs/kg TCOD	2,310 EUR	8,429 EUR	2,310 EUR	4,395 EUR
o&m: fixed costs for the OTC				
per year	156,220 NAD 10,864 EUR		833 NAD 58 EUR	
costs/((use/d)/a)	14.48 EUR	65 EUR	1 EUR	-
costs/(person×a)	43 EUR	-	2 EUR	4 EUR
costs/kg TCOD	6,249 EUR	20,380 EUR	298 EUR	2,155 EUR
o&m: total costs				
per year	213,981 NAD 14,880 EUR	220,834 NAD 15,357 EUR	7,303 NAD 508 EUR	2,533 NAD 176 EUR
costs/((use/d)/a)	20 EUR	93 EUR	6 EUR	-
costs/(person×a)	60 EUR	-	18 EUR	13 EUR
costs/kg TCOD	595 EUR	2,003 EUR	181 EUR	455 EUR

One cluster unit was originally designed to collect 2.8 kg TCOD/d. The communal washhouse should have collected roughly ten times more: 25.0 kg TCOD/d. The monitored loads were lower. One cluster unit collected only 0.4 kg TCOD/d and the communal washhouse only 7.7 kg TCOD/d.

According to the planning data, the capital costs of the communal washhouse would be 4,864 EUR/kg TCOD and 2,463 EUR/kg TCOD for the cluster unit. Referring the capital costs to the monitoring data, the costs for the communal washhouse were 15,865 EUR/kg TCOD and 17,838 EUR/kg TCOD for the cluster unit. Thus, as the communal washhouse has a higher efficiency regarding collection of excreta, it performs better than the cluster unit in terms of monitoring data. However, both facilities remain far behind the planning data.

During planning, the specific water use and therefor the costs for tap water were assumed to be identical for the communal washhouse and for the cluster units. The monitored specific water use of the cluster units was only half the specific water use monitored for the communal washhouse. Thus, expenses for tap water were lower.

One cluster unit was intended to serve 28 persons. The communal washhouse was intended to serve 250 persons, i.e., 9 times more. The o&m costs of nine cluster units occurring for the OTC were 522 EUR/a (= 9 × 58 EUR/a, only spare parts, no staff costs); thus, only 5% of the expenses required for the communal washhouse (522 EUR/a ÷ 10,864 EUR/a = 0.05). This is due to the staff costs required for operating the communal washhouse (8,883 EUR/a of 10,864 EUR/a). The remaining fixed costs were 1,981 EUR/a for the communal washhouse (spare parts, toilet paper, cleaning equipment, electricity).

One cluster unit had a lower unit price, lower capital costs referred to the number of toilet seats, and a lower specific water use than the communal washhouse. After implementation, the cluster units were less efficient in excreta collection than the communal washhouse and thus capital costs were higher when referred to collected TCOD equivalents.

From the OTC`s perspective, the most striking advantage of the cluster units was that staff expenses could be avoided because management is carried out by the users. This lowers overall o&m costs considerably. However, compared to the communal washhouse, more capacities need to be available at the OTC for carrying out regular repairs and maintenance of toilets and hand wash basins, water meter monitoring, for facilitating initial workshops and for continuous accompanying measures to assist the participating households with organizational issues. These costs were not accounted for here but incurred by the OTC. If the disadvantage of low excreta collection could be overcome, the cluster units would clearly outperform the communal washhouse regarding the aspects considered here.

4.4.4 Conclusions

Tariffs and o&m costs were very important issues during the project period because financial sustainability is a key issue for successful implementation. Therefore, this chapter dealt with capital and o&m costs of the implemented shared sanitation facilities and compared them with literature data.

For both sanitation facilities, o&m cost recovery could not be achieved for the anticipated levels of fixed and variable costs. Additional funds were required to cover the running costs.

O&m cost recovery for a larger sanitation facility like the communal washhouse is difficult when implementing under local conditions comparable to the ones in Outapi (low population density and per capita income). Sufficient revenues can only be achieved for relatively high utilization rates and relatively low specific water uses.

In order to provide access to water and sanitation for a large proportion of the population, tariffs need to be kept at a low level. If subsidies are required, they can be generated at the level of the sanitation facility (e.g., via shops and outdoor advertising, as suggested in the literature), by other components of the sanitation system (e.g., by selling the reclaimed water to the operator of the irrigation site) or by inclusion of economically stronger neighbourhoods.

If the objective is to cover all o&m costs via tariffs, this could be achieved in areas with a higher population density. As a rough indicator, a minimum population density of 70 persons/ha is recommended. In most African cities and, particularly in informal settlements, population densities are usually higher. Hence, it should be possible to cover o&m costs for long-term financial sustainability.

Another option for reducing o&m costs was explored at the cluster units. Here, caretaking and cleaning activities were transferred to the users of the facilities. Such smaller sanitation facilities that are manageable by the allocated households have considerably lower o&m costs than larger facilities that need to be managed by paid staff. However, sufficient capacities need to be available within the local authority for ongoing community involvement and post-implementation measures.

Altogether, the cluster units had lower investment and lower o&m costs than the communal washhouse. Hence, implementation of such smaller facilities is the recommended approach for sanitation provision in informal settlements. However, in terms of excreta collection, the communal washhouse yielded better results than the cluster units. If this shortcoming of the cluster unit could be overcome, they would clearly outperform the communal washhouse.

In any case, targeted sanitation marketing is recommended to promote the offered services among the local population, in order to achieve higher utilization rates and, thus, higher revenues. Future research should focus on the reasons that lead to low utilization of the facilities and how such barriers could be reduced.

4.5 Quality of the reclaimed water

Development or adoption of comprehensive guidelines facilitates the realization of water reuse projects. In this chapter, appropriate water quality objectives for the presented water reuse project are developed. The project was implemented in a setting where national regulations did not yet exist. Available international guidelines were applied to the local context. This chapter addresses the parameters that should be modified or added, considering the water quality requirements for agricultural irrigation; the suitability of the irrigation water for this specific case; the measures that have to be taken to comply with the desired water quality requirements; and possible water reuse applications for the obtained water quality.

4.5.1 Definition of water quality parameters

Guidelines for irrigation water quality or water reuse do not exist in Namibia. During planning, the question about which water quality objectives should apply to the implemented water reuse project arose. The use of guidelines from neighboring countries would be an option because it can be assumed that they match the (similar) local conditions. In the region, only South Africa has guidelines for irrigation water quality. However, they date back to 1978 and require drinking water quality for the irrigation of "vegetables and crops consumed raw by men" (DNHPD 1978). These guidelines are assessed as "largely inappropriate for low- to middle-income South African settlements" because they pursue a zero-risk approach without consideration of the available financial capacities and conceptual adaption to the local conditions (Ilemobade *et al.* 2009).

Usually, detailed background information is not given in national guidelines on how the suggested parameters and recommended limits were chosen. Paranychianakis *et al.* (2015) conclude that "water reuse criteria have been set (semi-)empirically, instead than based on the interpretation of the available scientific knowledge". Since detailed information is not available, an assessment of whether guidelines for other regions fit to the local conditions in Namibia is not possible. Consequently, at the moment, a rationale for using national guidelines from another country for implementation in Namibia is not available. Thus, in this case, the quality of irrigation water was assessed using the internationally accepted FAO (1985) (Ayers and Westcot 1985) and WHO (2006) guidelines.

As it turned out, additional parameters and modification of existing ones were needed to carry out the water quality monitoring required for this water reuse project. The individual aspects are described in more detail here. An overview of recommended limits from the FAO and WHO guidelines and additional values suggested in this study is given in Table 26. This section outlines the definition of additional water quality limits needed for water quality monitoring.

4.5.1.1 Total suspended solids

Suspended solids are among the major constituents of domestic wastewater. They need to be managed, because particles might cause clogging of irrigation equipment (e.g., drip lines),

Table 26 Effluent water quality of the water reuse plant (July 2013 to July 2015), water quality objectives of the FAO and WHO guidelines (Ayers and Westcot 1985; WHO 2006) and suggested additional limits, FAO (1985) distinguishes three "degrees of restriction on use" (1 = none, 2 = slight to moderate, 3 = severe)

		monitoring data									**water quality objectives**			
		untreated wastewater			**effluent**			**storage pond**			**objective**			
parameter	**unit**	**mean**	**sd**	**n**	**mean**	**sd**	**n**	**mean**	**sd**	**n**	**1**	**2**	**3**	**source**
physical characteristics														
EC	µS/cm	612	180	372	527	132	344	596	100	75	< 700	700 - 3,000	> 3,000	FAO (1985)
TDS	mg/L	-	-	-	375	46.7	4	455	133	5	< 450	450 - 2,000	> 2,000	FAO (1985)
turbidity	FNU	507	218	322	7.5	5.6	326	16.7	9.7	62	< 21[a] < 10[b]	23 - 43[a]	> 43[a]	this study
TS	mg/L	1,040	428	24	381	74.8	18	476	96.0	8	-	-	-	-
TSS	mg/L	-	-	-	9[c]	-	-	30[c]	-	-	< 50[a] < 25[b]	50 - 100[a]	> 100[a]	FAO (1985) this study
chemical characteristics														
pH	-	7.8	0.3	347	6.8	0.5	344	8.0	1.3	74	"normal range": 6.5 - 8.4			FAO (1985)
TCOD	mg/L	738	364	132	57.7	26.1	125	64.9	25.4	48	according to BOD/TCOD ratio			this study
TN	mg/L	57.5	25.8	131	33.5	17.2	127	32.6	13.1	48	< 5	5 - 30	> 30	FAO (1985)
TP	mg/L	10.3	3.3	121	8.3	2.4	121	9.9	4.0	48	< 3.5	3.5 - 13	> 13	this study
K^+	mg/L	17.3	2.9	14	18.8	3.4	19	24.2	1.9	5	< 6.5	6.5 - 28	> 28	this study
Na^+	mg/L	58.7	21.7	14	53.2	15.5	19	64.4	4.5	5	< 3[d]	3 - 9[d]	> 9[d]	FAO (1985)
Ca^{2+}	mg/L	10.4	3.9	12	17.8	4.5	17	7.8	3.2	5	-	-	-	-
Mg^{2+}	mg/L	3.6	0.8	12	4.4	1.5	17	5.6	2.5	6	-	-	-	-
SAR	-	4.0	-	-	2.9	-	-	4.3	-	-	> 1,200[e]	1,200 - 300[e]	< 300[e]	FAO (1985)
B^-	mg/L	0.02	0.00	9	0.02	0.01	12	0.02	0.01	6	< 0.7	0.7 - 3.0	> 3.0	FAO (1985)
Cl^-	mg/L	30.3	6.5	3	37.0	4.4	3	44.0	4.0	3	< 4[d]	4 - 10[d]	> 10[d]	FAO (1985)
biological characteristics														
BOD_5	mg/L	196	152	10	5.5	2.0	13	16.0	8.5	6		15		this study
HE	1/L	-	-	-	308	359	5	0.0	0.0	3		case specific		WHO (2006)
E. coli	MPN/ 100 mL	2.3E+07	2.2E+07	65	2.2E+03	8.5E+03	57	4.0E+01	1.1E+02	45		case specific		WHO (2006)
		1.7E+07[f]	-	-	1.3E+01[f]	-	-	9.6E+00[f]	-	-		-		-
total coliforms	MPN/ 100 mL	6.3E+07	6.2E+07	66	6.9E+03	2.4E+04	59	4.5E+03	1.0E+04	45		-		-
		5.3E+07[f]	-	-	2.0E+02[f]	-	-	2.0E+02[f]	-	-		-		-

n = number of measurements, sd = standard deviation, EC = electrical conductivity, TDS = total dissolved solids, TS = total solids, TSS = total suspended solids, TCOD = total chemical oxygen demand, TN = total nitrogen, TP = total phosphorus, SAR = sodium adsorption ratio, BOD_5 = 5-day biochemical oxygen demand, HE = helminth eggs, TC = total coliforms, MPN = most probable number, [a] prior to drip irrigation, [b] prior to UV disinfection, [c] calculated value (TSS = TS - TDS), [d] surface irrigation, [e] EC limits are given for SAR = 3 to 6, for a higher or lower SAR, EC limits are different, [f] median influence the efficiency of disinfection and lead to aesthetic impairment of the water (Ayers and Westcot 1985). The amount of particles in water can be expressed by the turbidity or by the total suspended solids (TSS) content (Tchobanoglous *et al.* 2004). Ayers and Westcot (1985) give limits of < 50 mg/L (= no restriction on use) and > 100 mg/L (= severe restrictions on use) for TSS, when irrigating with drip lines. Turbidity values are not given in this guideline.

In the Outapi case, the determination of TSS was not possible, due to the rapid clogging of glass fiber filters (weighable filter cakes could not be obtained). TSS determination was only possible via the determination of TS and TDS in the same sample. Due to the low TDS content of the water, relatively large sample quantities had to be filtered and evaporated. Since this was very time-consuming, turbidity measurements were used as a surrogate.

Turbidity measurements are much easier to perform, and the results are immediately available. Even though it has to be kept in mind that the relationship between TSS and turbidity is plant-specific, it is approximately TSS (mg/L) = turbidity (NTU) × 2.35 for settled secondary effluents (Tchobanoglous *et al.* 2004). The corresponding turbidity limits are < 21 (= no restriction on use) and > 43 NTU (= severe restrictions on use) to protect drip lines.

Disinfection of reclaimed water may be required for irrigation. Bulk parameters such as turbidity and TSS are often used to assess water quality prior to disinfection. Mamane (2008) concludes from the reviewed literature that turbidity levels up to roughly 10 NTU can be neglected for the inactivation of seeded viruses, bacteria and parasites via UV disinfection. For this case, a turbidity limit < 10 NTU is set as the required water quality objective prior to UV disinfection.

For TSS, some studies show a relationship between TSS content and microorganisms after UV disinfection (Carnimeo *et al.* 1994; Darby *et al.* 1993; Severin 1980; Whitby and Palmateer 1993; White *et al.* 1986), and some show only minor or even no effects (Cantwell and Hofmann 2011; Petrasek *et al.* 1980; Qualls 1983). Suggested TSS concentrations prior to UV disinfection are < 30 mg/L (Carnimeo *et al.* 1994; Severin 1980) or < 20 mg/L (Darby *et al.* 1993; White *et al.* 1986). Although TSS measurements could not be used for water quality monitoring in this case (the determination of TSS was not possible due to rapid clogging of the glass fiber filters), a water quality objective of 25 mg/L is suggested whenever regular determination of TSS is possible.

On the whole, common solids-related parameters such as TSS and turbidity do not reliably predict UV disinfection performance (Madge and Jensen 2006). Instead, the size of the particles is crucial. Particles < 10 µm do not influence UV disinfection (Emerick *et al.* 1999; Parker and Darby 1995). If, and to what extent, UV radiation can penetrate larger particles depends on the respective characteristics of the particles, e.g., their porosity. The critical size is therefore plant-specific (Emerick *et al.* 1999; Parker and Darby 1995). Particles > 10 µm should be removed from the water when provision is made for UV disinfection. Ideally, the plant-specific maximum admissible particle size is determined and monitoring of particle size (via serial filtration, electronic particle size counting or microscopic observation (Tchobanoglous *et al.*

2004)) is performed regularly; otherwise, the corresponding turbidity or TSS content should be used for water quality monitoring.

4.5.1.2 Chemical and biochemical oxygen demand

Degradable organic matter may cause anaerobic conditions during storage and trigger the clogging of irrigation equipment, either directly or indirectly, by stimulating the growth of microorganisms (Ayers and Westcot 1985). Therefore, organic matter contained in irrigation water should be stabilized to a large extent, to inhibit further biodegradation. On the other hand, the input of organic matter has a positive effect on soil properties (Ayers and Westcot 1985). However, to protect the implemented infrastructure, control of degradable organic matter content is required.

In the FAO guidelines, a recommendation for the BOD of irrigation water is not given. Several other guidelines include limits for 5-day BOD. The United States Environmental Protection Agency, for example, recommends a maximum BOD_5 of 10 mg/L for food crops and 30 mg/L for non-food and processed food crops (USEPA 2012). AQUAREC (2006a) recommend a BOD_5 of 10 to 20 mg/L for irrigation purposes. In European guidelines, recommended BOD limits range from 10 to 20 mg/L for irrigation of vegetables eaten uncooked (Paranychianakis *et al.* 2015). A BOD_5 of 15 mg/L could be set as water quality objective for irrigation water quality.

The TCOD is a widely used alternative parameter for BOD when assessing the efficiency of wastewater treatment steps, because results are obtained faster and values are more reproducible (Tchobanoglous *et al.* 2004). This parameter is not included in the FAO guidelines. If the BOD_5/TCOD ratio is stable, TCOD water quality objectives can be derived and used to assess the degree of stabilization of the water.

4.5.1.3 Nitrogen, phosphorus and potassium

In some crops, excessive nitrogen might cause over-stimulation of growth, delayed maturity, or poor crop quality (Ayers and Westcot 1985). Excess P and K may accumulate in the soil or leach out (Pescod 1992). Thus, monitoring N, P and K loads is necessary for optimal nutrient management when reclaiming water for irrigation.

The FAO nitrogen limit of 5 to 30 mg/L corresponds to the range of N requirements for typical crops (Table 27, page 126, Doorenbos (1979)). For adequate fertilization of, for example, tomatoes, TN should not exceed 25 mg/L. This limit is based on an N requirement of 125 kg N/ha per growing period and an irrigation demand of 5,000 m³/ha per growing period (125 kg N/ha ÷ 5,000 m³/ha = 25 mg/L) under two preconditions: irrigation only with reclaimed water and no leaching via excess irrigation or rainfall (Table 27).

When reclaiming water for agricultural irrigation, TN concentrations will usually exceed the FAO limits, if an N removal step is not implemented. Even for relatively low per capita N loads (e.g., 8 g/(person×d), Table 2, page 16, Sperling (2007c)) and high water use (e.g., 200 L/(person×d)), the total N concentration in the irrigation water will be higher than the N requirements of most crops (8 g/(person×d) ÷ 200 L/(person×d) = 40 mg/L; N removal via sedimentation or incorporation into biomass is neglected). To avoid negative impacts from excessive N loads, N-intensive crops should be chosen for irrigation with reclaimed water. High N concentrations could also be handled by introducing a denitrification step. If all nutrients are to be used for irrigation, the reclaimed water might need blending with other water sources.

When conventional water sources (surface water, groundwater (FAO 2015)) are used for irrigation, P and K concentrations in irrigation water are not expected to exceed crop requirements (UNEP 2008, 2007). However, when applying treated (waste)water, higher P and K loads are expected. For a typical P load ranging from 1 to 3 g/(person×d) (Table 2, page 16) and a water use between 50 and 200 L/(person×d), P concentrations are within a range of 5 to 60 mg/L and exceed most of the limits listed in Table 27 (P incorporation in biomass is neglected, no P removal during wastewater treatment). Potassium concentrations in treated wastewater might range from 15 to 120 mg/L and will also exceed requirements for many crops (3 to 6 g K/(person×d) and water use between 50 and 200 L/(person×d) (DWA 2008c)).

In this study, maize, peppers and tomatoes were mostly cultivated. The adapted water quality objectives are 18 to 25 mg/L for TN, 5 to 18 mg/L for TP and 10 to 40 mg/L for K. For further classification, the limits for TP and K can be set at < 3.5 and < 6.5 mg/L (no restriction on use), 3.5 to 13 and 6.5 to 28 mg/L (slight to moderate restriction on use) and > 13 and > 28 mg/L, respectively (severe restriction on use, Table 26).

Table 27 Irrigation and nutrient requirement for various crops (Doorenbos 1979) and water quality objectives for total N, P and K in irrigation water (when irrigated only with reclaimed water, no leaching e.g., via excess irrigation or rainfall)

crop	**water and nutrient requirement per growing period**				**target concentration irrigation water**		
	water m³/ha	TN kg/ha	TP kg/ha	K kg/ha	TN mg/L	TP mg/L	K mg/L
groundnut	6,000	15.0	27.5	32.5	2.5	4.6	5.4
bean	4,000	30.0	50.0	85.0	7.5	12.5	21.3
sunflower	8,000	75.0	32.5	92.5	9.4	4.1	11.6
safflower	9,000	85.0	22.5	32.5	9.4	2.5	3.6
banana	17,000	300	52.5	260	17.6	3.1	15.3
pepper	7,500	135	37.5	75.0	18.0	5.0	10.0
watermelon	5,000	90.0	42.5	57.5	18.0	8.5	11.5
wheat	5,500	125	40.0	37.5	22.7	7.3	6.8
maize	6,500	150	65.0	80.0	23.1	10.0	12.3
sugarbeet	6,500	150	60.0	130	23.1	9.2	20.0
tomato	5,000	125	87.5	201	25.0	17.5	40.1
cabbage	4,400	125	57.5	115	28.4	13.1	26.1
olive	7,000	225	62.5	185	32.1	8.9	26.4

4.5.2 Water quality monitoring

4.5.2.1 Electrical conductivity and total dissolved solids

The salinity of irrigation water must be monitored in order to prevent soil salinization and reduced crop yields (Ayers and Westcot 1985). Because the determination of TDS was time-consuming in this project, the EC was used as a surrogate parameter to characterize the irrigation water quality (see Section 3.3, page 46).

In this study, the EC of the water increased from 52 µS/cm in tap water to 527 µS/cm in the effluent (due to domestic water use) up to 596 µS/cm in the storage pond (due to evaporation, Table 26). Similarly, TDS concentrations increased from 40.2 mg/L in tap water to 375 mg/L in the effluent and to 455 mg/L in the storage pond. The mean EC value in the storage pond and in the effluent was lower than the FAO limit of 700 µS/cm. The mean TDS concentration in the storage pond slightly exceeded the FAO limit of 450 mg/L (Table 26, page 123). Although there was no immediate limitation in crop choice, dissolved salts still need to be monitored and leached, to prevent accumulation in the soil. In this case, salt management was carried out via regular drainage and leaching of the fields.

The example shows that even though the amount of TDS in tap water was very low, domestic water use increased concentrations and loads (depending on the specific water use) to levels only slightly under the FAO limits for EC and TDS. Therefore, in cases with higher TDS levels in tap water, EC and TDS monitoring is even more important. Salts in irrigation water and soil have to be controlled, to allow sustainable irrigation.

4.5.2.2 Turbidity and total suspended solids

The mean value for turbidity was 7.5 FNU in the effluent and 16.7 FNU in the storage pond. Both values met the suggested water quality objective for drip irrigation (21 NTU). However, turbidity was exceeded in 3% of the effluent samples and in 24% of the storage pond samples.

Turbidity varied: from July 2013 until May 2014, mean turbidity was 8.2 FNU in the effluent of the lamella clarifier and 4.4 FNU in the effluent of the microscreen (Figure 64). Subsequently, the mean turbidity increased to 19.4 FNU in the effluent of the lamella clarifier and to 12.1 FNU after passing through the microscreen (June 2014 to January 2015). Retrofitting of the microscreen in February 2015 led to a lower mean turbidity value of 6.8 FNU in the effluent (March 2015 to July 2015), despite higher turbidity levels in the effluent of the RBC and the lamella clarifier (17.6 FNU on average, up to 27.3 FNU in June 2015).

Regarding water quality requirements prior to UV disinfection, there was some room for improvement. 72% of all samples met the suggested limit of 10 NTU. Following retrofitting of the microscreen in February 2015, the turbidity prior to UV disinfection improved slightly: 88% of all samples were below 10 FNU.

There are several scale units and methods with which turbidity can be measured. Here, turbidity measurements are given in FNU (formazine nephelometric unit (ISO 7027 1999)), whereas the EPA method 180.1 gives turbidity values in NTU (nephelometric turbidity unit). Both methods measure scattered light at a 90º angle, but at different wavelengths (Eaton and Franson 2005; ISO 7027 1999). Sadar (1999) found out that both methods deliver almost the same results for samples with low turbidity, i.e., in a simplified way FNU ≈ NTU. For an approximate assessment, one can therefore use values stated either in FNU or NTU.

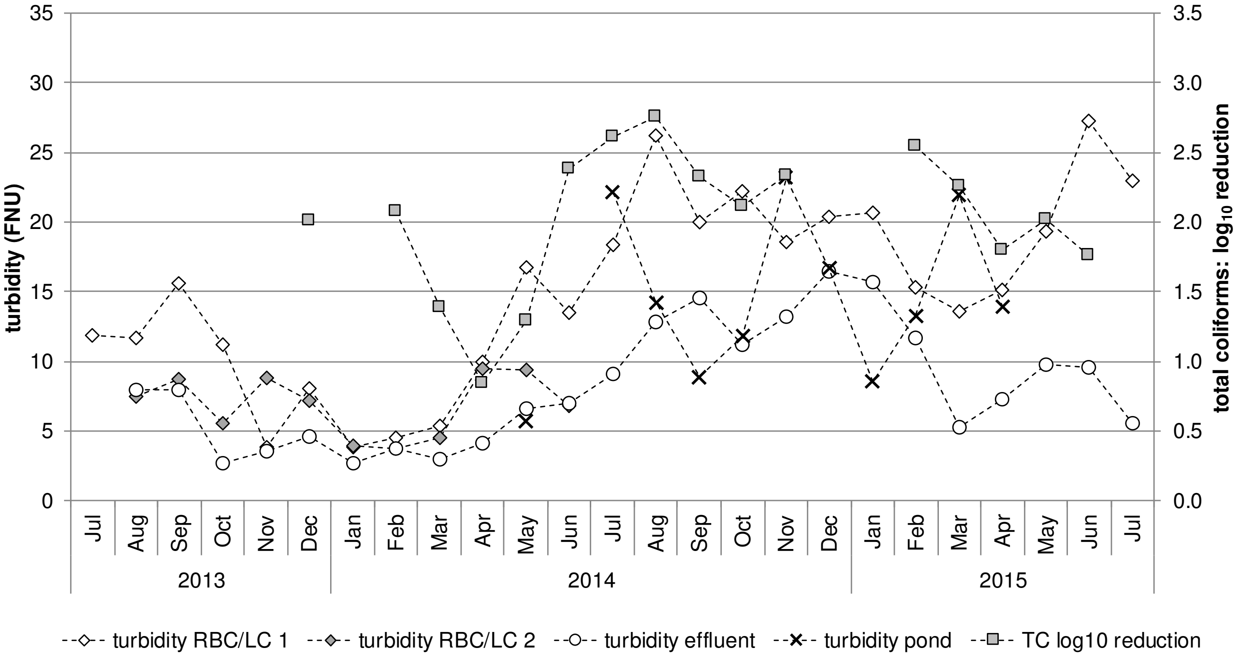

Figure 64 Turbidity in the effluent of the RBCs and lamella clarifiers (RBC/LC 1 and 2), after passing the microscreen (effluent) and in the storage pond, and log10 reduction of total coliforms (TC)

UV doses were very high (> 100 mJ/cm²) because flows were below the design value and UV disinfection was designed for higher (peak) flows. Nevertheless, the mean $\log_{10}$ reduction was only 2.2 for total coliforms and 0.9 for *E. coli*. Here, mean total coliform concentrations were 6,900 MPN/100 mL in the effluent (median = 200 MPN/100 mL, Table 26). A dose of 100 mJ/cm² should be sufficient to obtain mean total coliform concentrations below 2.2 MPN/100 mL (NWRI 2012).

The results show that optimal $\log_{10}$ reduction rates of *E. coli* and total coliforms have not been achieved. The reason for the relatively low $\log_{10}$ reduction of total coliforms and *E. coli*, despite the high UV dose, could have been incomplete removal of larger particles in the lamella clarifiers and the microscreen. In the first place, the microscreen was installed for retention of helminth eggs. In the second place, it was installed to remove particles for more efficient UV disinfection.

The content of solids larger than 20 µm was 12 mg/L in the influent of the microscreen and 4.1 mg/L in the effluent of the microscreen (n = 3). Thus, the reduction was roughly 66% (and should be higher for TSS). This corresponded to the average turbidity reduction (57%) and was within the range of 10 to 80% TSS removal (55% on average) reported by Tchobanoglous *et al.* (2004). However, by using a microscreen with a mesh size of 15 µm, it should be possible to remove all particles > 20 µm.

The baskets of the chosen microscreen were initially firmly installed and sealed with foam rubber seals against the frame. Filtration took place from the inside outwards. Nozzles were included for cleaning the microscreen baskets. They sprayed material off when an adjustable pressure loss was exceeded. The operation of the screen should be automated. A disassembly of the screens is not intended for routine cleaning and should be avoided, because the seals do not allow multiple use.

Clogging of the microscreen occurred during operation. The spray nozzles were operated at 5 bar, but they could not remove the biofilm sufficiently. The required additional maintenance (repeated assembly and disassembly of the screen elements and manual, high-pressure cleaning) probably led to leaky rubber foam strips of the screen baskets as well as incomplete removal of solids and helminth eggs (see Section 4.5.2.9, page 134). In spite of retrofitting the microscreen and additional training of the operating staff, the required maintenance of the lamella clarifier and microscreen exceeded the available capacities.

Overall, the removal efficiency of a freshly cleaned, undamaged and carefully built microscreen with new seals is acceptable. However, the actually installed cleaning devices were not able to ensure a long service life without removal and cleaning of the interior.

From a conceptual point of view, the installation of a UV disinfection system in the effluent of the storage pond should be considered. Emerick *et al.* (1999) investigated the number of bacteria-associated particles in various wastewater samples. They determined that between 4% and 31% of particles in samples from aerobic treatment steps (activated sludge process, trickling filter) contained embedded coliform bacteria. In aerated or facultative lagoons, this percentage was below 1%. The number of residual coliform bacteria surviving high UV doses was low, despite high TSS concentrations. Another study found that polishing pond effluents can achieve a high $\log_{10}$ reduction for *E. coli* (2.8 to 3.4) and total coliforms (2.6 to 3.1) despite a high TSS content (87 to 102 mg/L) and low absorbance (0.67 to 0.79) caused by algae. This was due to the high percentage (94%) of particles < 10 µm in the effluent (Alves *et al.* 2012). Thus, if it is not possible or desired to provide the required water quality prior to UV disinfection, the disinfection system could be installed in the effluent of the storage pond. In this way, a high $\log_{10}$ reduction for *E. coli* and total coliforms can be achieved, despite high TSS concentrations and absorbance.

In existing guidelines, water quality requirements prior to UV disinfection are high. For instance, when irrigating food crops, the USEPA (2012) suggests a 24-hour average turbidity of ≤ 2 FNU, that should not exceed 5 NTU at any time, and an average TSS of < 5 mg/L. The achievable removal rates for microorganisms are lower for inferior water quality (i.e., higher particle content). In case UV disinfection is nevertheless applied (e.g., in low-quality water or water with a relatively high particle content), water quality objectives for unrestricted irrigation, as suggested in many standards, cannot be met. However, the WHO (2006) guidelines allow a lower degree of wastewater treatment, combined with other measures, to achieve the required $\log_{10}$ pathogen reduction for a specific health-based target (HBT). Thus, even though the achieved $\log_{10}$ removal rates for *E. coli* and total coliforms were rather low in this case, the achieved reduction contributed to meeting the required HBT (see Section 4.5.2.8, page 133).

4.5.2.3 PH and alkalinity

Water with a low pH can be corrosive, whilst water with a high pH might be scale-forming (Tchobanoglous *et al.* 2004). The FAO guidelines generally recommend a "normal" range of pH 6.5 to 8.4 (Table 26). A range of pH 7.0 to 8.0 is recommended for drip irrigation systems. For sprinkler irrigation, the pH should not fall below 6.5 (Ayers and Westcot 1985).

In the present case, the mean pH in the untreated wastewater was 7.8 (Table 26, page 123). Following anaerobic pretreatment, the mean pH was 6.9 (±0.4) (Figure 65), which is within the optimal range for methane-producing microorganisms (pH 6.6 to 7.4 (Chernicharo 2007)). The alkalinity of the untreated wastewater was 10.3 (±2.1) mmol/L and 9.8 (±3.8) mmol/L after anaerobic pretreatment.

Upon completion of the anaerobic pretreatment, the wastewater was treated aerobically. Rotating biological contactors were designed for TCOD removal. After implementation, flows were much lower than planned (30.3 m^3/d instead of 90.0 m^3/d). Because nutrients should remain in the water for fertilization, denitrification was not implemented. This caused a further decrease of the pH during (unintended) nitrification (mean pH = 6.8, Table 26) although only one out of two RBCs was operated, and, most notably, a decrease of alkalinity was observed (effluent: 1.5 (±1.5) mmol/L). As a consequence, variation of the pH after aerobic treatment was relatively high (±0.5).

In the present case, water was applied via surface drip irrigation. Thus, the pH should be between 7.0 and 8.0 (Ayers and Westcot 1985). In most cases, the effluent did not meet the required pH for drip irrigation (65% of the measured values were below pH 7.0, pH 8.0 was not exceeded). Overall, the combination of anaerobic pretreatment and nitrification during aerobic treatment led to low effluent pH values with a high variation. This should be considered when reclaiming waters with low alkalinity.

In the storage pond, the pH increased because algae consumed CO_2 and HCO_3^- during photosynthesis. The average pH was 8.0, but varied: 30% of the measured values fell below pH 7.0 and 55% exceeded pH 8.0. Since the pond's commissioning in April 2014 (Figure 65), the pH increased continuously. Alkalinity remained very low (1.6 (±0.4) mmol/L). Thus, the water was not expected to cause scaling, despite the high pH.

Liming would be an easily implementable solution for pH control in the effluent of the wastewater treatment plant in order to prevent corrosion. Implementation of a denitrification step would also lead to a higher pH and lower standard deviation. Because the pond water was less aggressive than the effluent of the wastewater treatment plant, it should be more suitable for irrigation.

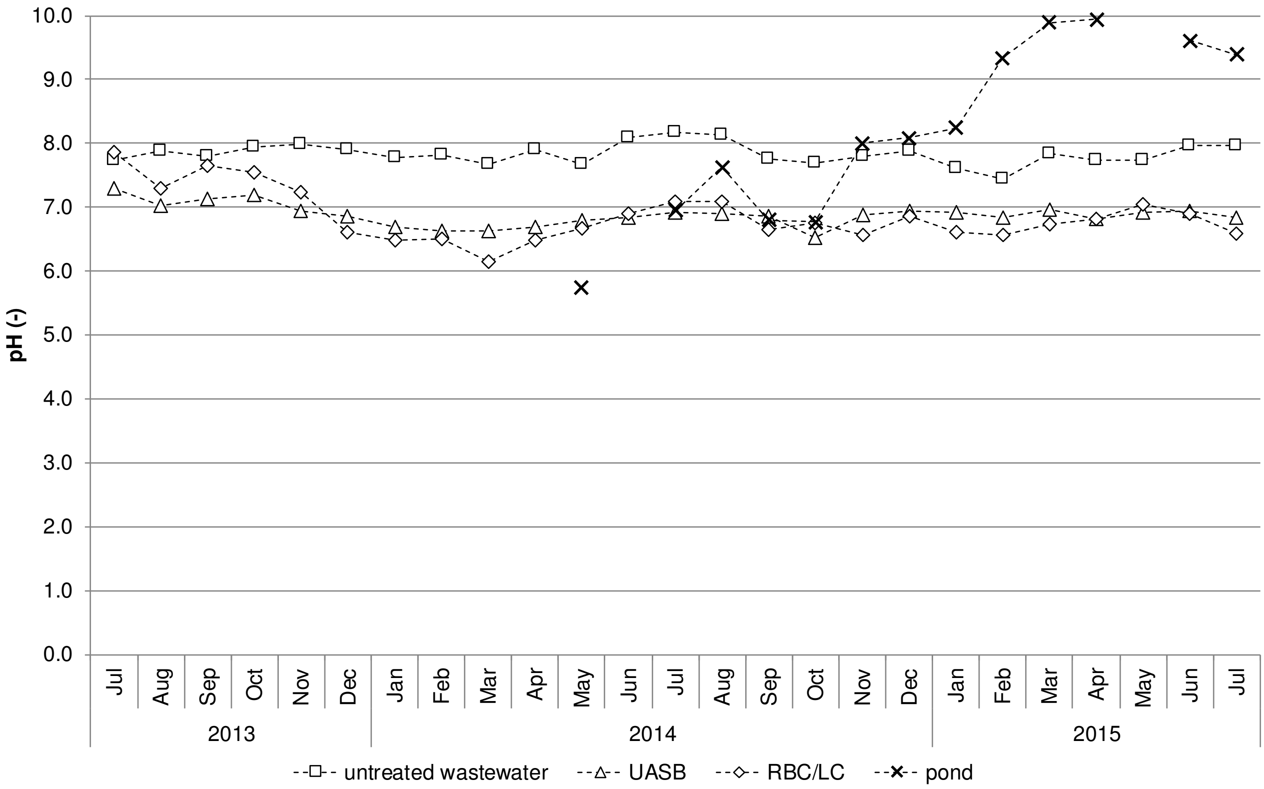

Figure 65 PH in the untreated wastewater, after sedimentation and anaerobic pretreatment (UASB), after aerobic treatment and separation of solids via lamella clarifiers (RBC/LC) and in the storage pond (bars represent the standard deviation of the mean)

4.5.2.4 Chemical and biochemical oxygen demand

The BOD_5 was reduced from 196 mg/L in the untreated wastewater to 5.5 mg/L in the effluent (Table 26). Thus, the effluent water was stabilized to a large degree. In the storage pond, the BOD increased to 16.0 mg/L because organic matter was added by algal growth and animals.

TCOD was 57.7 mg/L in the effluent and 64.9 mg/L in the storage pond. Consequently, the BOD_5/TCOD ratio was 0.1 in the effluent (5.5 mg/L ÷ 57.7 mg/L = 0.1) and 0.25 in the storage pond (16.0 mg/L ÷ 64.9 mg/L = 0.25). Thus, assuming a stable ratio and a BOD_5 limit of 15 mg/L, the adapted TCOD limit is 150 mg/L for the effluent (15 mg/L ÷ 0.1 = 150 mg/L) and 60 mg/L for the storage pond (15 mg/L ÷ 0.25 = 60 mg/L).

None of the effluent's TCOD concentrations exceeded 150 mg/L. However, in the storage pond, 53% of the samples showed values > 60 mg/L. To prevent the clogging of drip lines, disc filters were installed in the irrigation system. Regarding the TCOD to BOD_5 ratio, the effluent of the wastewater treatment plant was more suitable for drip irrigation than the water extracted from the storage pond.

4.5.2.5 Nitrogen, phosphorus and potassium

The mean TN content of the effluent water was 33.5 mg/L. Almost the same mean concentration was measured in the storage pond (32.6 mg/L). Thus, the concentrations slightly exceeded the recommended FAO limit of 30 mg/L and also exceeded the requirements of most crops (Table 27).

The mean total P was 8.3 mg/L in the effluent and 9.9 mg/L in the storage pond. For most crops listed in Table 27, the P loads applied via the irrigation water exceeded the requirements.

The same applied to potassium with mean concentrations of 18.8 mg/L in the effluent and 24.2 mg/L in the storage pond. Whilst this was not high enough to supply sufficient amounts to tomatoes, most crops require less K.

In this study, mainly maize, peppers and tomatoes were cultivated. Compared to the adapted water quality objectives for these crops, the TP and K concentrations met the requirements of cultivated crops. TN concentrations were lower than expected, but still slightly exceeded crop requirements. Irrigation management should consider alternately irrigating with tap water and reclaimed water, to prevent adverse effects of nitrogen in plants.

4.5.2.6 Sodium, calcium and magnesium

Excessive Na^{+} can cause dispersion of fine soil particles and clogging (Ayers and Westcot 1985). This might occur when irrigating with low conductivity water that leaches Mg^{2+} and Ca^{2+} out of the soil, or when Na^{+} concentrations are very high, compared to Mg^{2+} and Ca^{2+} concentrations. This can be assessed with the sodium adsorption ratio (SAR) and the EC (SAR = $c_{Na+} \div \sqrt{(c_{Ca^{2+}} + c_{Mg^{2+}}) \div 2}$, concentrations in meq/L) (Ayers and Westcot 1985). For water with high carbonate and bicarbonate contents, the SAR should be adjusted (Ayers and Westcot 1985).

Here, the SAR was 4.0 in the effluent and 4.3 in the storage pond. For SAR values ranging between 3 and 6, negative effects are not expected for an EC > 1,200 µS/cm. For an EC < 300 µS/cm, severe infiltration problems will occur (Ayers and Westcot 1985). The EC of the irrigation water was between 527 µS/cm (effluent) and 596 µS/cm (storage pond). Thus, moderate to severe infiltration problems could be expected. Soil properties need to be monitored.

If infiltration rates are low, remedial actions are only required if the crop water demand or leaching requirements cannot be met (Ayers and Westcot 1985). Chemical and physical remedial measures, such as adding of gypsum to the soil, blending of the reclaimed water with other water sources, or tillage, can be applied (Ayers and Westcot 1985).

4.5.2.7 Boron and trace elements

High boron concentrations are toxic for plants (Ayers and Westcot 1985). Because household detergents might contain boron, B concentrations could be an issue when irrigating with reclaimed water (Pescod 1992). Manganese can cause clogging and be toxic for plants, whereas heavy metals can accumulate in soil and plants (Tchobanoglous *et al.* 2004). These parameters were monitored in the irrigation water in Outapi; however, they never reached FAO limits. The results for Boron are displayed in Table 26 (page 123). The concentrations of Cd, Co, Cr, Cu, Mn, Ni and Pb were always below the detection limit. The maximum concentration of Fe was 0.12 mg/L. This is below the recommended maximum concentration of 5.0 mg/L (Ayers and Westcot 1985).

4.5.2.8 E. coli

E. coli is the indicator organism suggested by WHO (2006) for pathogens. Depending on the irrigation method and the kind of crop, WHO (2006) recommends an overall $\log_{10}$ reduction between 2 and 7 units for *E. coli* in order to achieve a health-based target of $\leq 10^{-6}$ disability-adjusted life years per person per year (see WHO (2006)). For the unrestricted irrigation of crops with above-ground harvested parts, the recommended reduction is 6 $\log_{10}$ units (WHO 2006).

The mean $\log_{10}$ reduction for *E. coli* was 3.1 units during wastewater treatment (prior to UV disinfection) and 0.9 units after UV disinfection. Die-off in the storage pond was about 1.7 $\log_{10}$ units. Local drip irrigation of low-growing crops further reduced pathogens by an assumed 2.0 $\log_{10}$ units (WHO 2006). This led to an overall $\log_{10}$ reduction of 7.7 units when all barriers (anaerobic + aerobic wastewater treatment, UV disinfection, storage pond and drip irrigation) were functioning properly. Other barriers might have existed and provided additional reduction of pathogens (e.g., washing produce at home, die-off during storage), but they could not be controlled under the local conditions and were therefore not considered.

An average reduction of 6 $\log_{10}$ units could still be achieved when only three barriers were operating (e.g. $\log_{10}$ reduction for wastewater treatment + die-off in storage pond + drip irrigation = 3.1 + 1.7 + 2.0 = 6.8); thus, UV disinfection would not have been necessary. In practice, however, these barriers were often bypassed. Farmers might have irrigated vegetable crops with hoses or extracted irrigation water from the effluent of the wastewater treatment plant or the storage pond for soil preparation. UV disinfection occasionally experienced operational problems. Water might have been pumped directly from the effluent to high-level tanks without retention in the storage pond.

During normal operation, the required water quality was exceeded. The question arose as to whether the 7.7 $\log_{10}$ reduction was reasonable, because every barrier consumed resources in one way or another. In theory, the circumvention of barriers could be avoided by improved infrastructure management. However, since operational malfunctions (human and technical) cannot be avoided, and in order to achieve the desired HBT for sufficient public health protection, all barriers were necessary to achieve an *E. coli* reduction of 6 $\log_{10}$ units at any time.

4.5.2.9 Helminth eggs and larvae

WHO (2006) recommends a maximum of one helminth egg (HE) per liter of irrigation water and 0.1 HE/L when children under 15 years are exposed. For localized irrigation of high-growing crops, a water quality objective is not required. The guidelines refer to the human intestinal nematodes *Ascaris lumbricoides* (human roundworm), *Trichuris trichiura* (human whipworm), *Ancylostoma duodenale* and *Necator americanus* (human hookworms) (Mara and Kramer 2008). The final hosts of *Hymenolepsis nana* are humans and mice (WHO 2004). *Hymenolepsis nana* is not included in the relevant helminths for the WHO (2006) water quality objectives, even though it can also infect humans (WHO 2004).

308 HE/L (including hookworm larvae) were counted in the effluent of the wastewater treatment plant. Hookworm eggs and hookworm larvae (presumably *Necator americanus*) constituted 99% of counts. Roundworm species (0.3 HE/L), *Taenia* sp. (0.7 HE/L, presumably *Taenia saginata*) and *Hymenolepsis nana* (0.9 HE/L) were less frequent. *Trichuris trichiura* eggs were never found. Using a microscreen mesh size of 20 µm more than 99% of helminth eggs should be retained (DWA 2016). The microscreen reduced helminth eggs only by an average of 33% – probably due to the reasons discussed in the previous section – and thus failed to provide the required water quality.

The standard deviation was high (±359 HE/L), whereas mean concentrations, collected in effluent samples from July 2014 to October 2014, ranged from 127 to 773 HE/L; the mean concentration in samples collected between March 2015 and April 2015 ranged from 0.4 to 5.8 HE/L. It is unknown whether this was due, for example, to changed sedimentation patterns in the plant or to lower concentrations in the untreated wastewater. Because analyses for helminth eggs are very time-consuming, they were only conducted in the influent and effluent of the microscreen.

The fact that helminth eggs could not be retained sufficiently (the recommended limit of ≤ 1 HE/L (WHO 2006) was exceeded) meant that direct use of the effluent of the wastewater treatment plant was only possible for localized irrigation of high-growing crops. Because *Taenia saginata* requires cows or pigs as the intermediate host, irrigation of, for example, fodder crops or pasture, is only an alternative if there is a gap of at least 14 days between irrigation and use as fodder (WHO 2004). However, this procedure is seen critically, because *Taenia* eggs can survive up to six months on grass and soil (WHO 2004).

Helminth eggs were completely retained in the storage pond. The pond is therefore the most important location for the retention of pathogens. Irrigation water should always be extracted from the pond.

4.5.2.10 Storage and water quality

As outlined in the previous sections, storage influenced the water quality. TCOD, BOD_5, EC, TDS, TSS, turbidity, and SAR increased during storage. The increase of TSS, TCOD, BOD_5 and turbidity could be remedied by installing common automatic backwash disc filters between the high tank and the drip irrigation system. PH also increased during water storage, but in contrast to the previously mentioned parameters, this was beneficial for the water quality in this project, because the stored water was less aggressive than the effluent of the wastewater treatment plant.

In addition, the storage pond was important for protection of public health. It equalized the variation in *E. coli* and total coliform concentrations, which led to a more uniform water quality. Die-off during storage further reduced *E. coli* and total coliform concentrations. Most notably, the pond was indispensable for retention of helminth eggs.

Altogether, the benefits by helminth egg retention, lower *E. coli* and total coliform concentrations and the higher pH outweighed the disadvantages caused by increasing TCOD, BOD_5, EC, TDS, TSS, turbidity and SAR. Hence, it is recommended to extract the irrigation water exclusively from the storage pond.

4.5.3 Conclusions

In this study, the FAO (1985) and WHO (2006) guidelines were used to monitor irrigation water quality. Which parameters should be modified or added, considering the water quality requirements for agricultural irrigation and considering the local conditions, has been discussed. In this fashion, water quality limits were developed that are tailored to the site-specific needs. Emphasis was placed on water quality requirements prior to UV disinfection, drip irrigation systems, and the nutrient requirements of cultivated crops. In order to meet the requirements of water reuse projects, additional water quality objectives for turbidity, BOD_5, TCOD, TP, and K were suggested. Depending on the water reuse concept and disinfection step, the objectives for TN and TSS may require modification.

The WHO (2006) guidelines provide a comprehensive approach for public health protection in water reuse projects. In the present case, to achieve the required *E. coli* $\log_{10}$ reduction at any time, an additional barrier was needed. Thus, during normal operation, the required water quality was exceeded. However, the extra barrier was necessary, because operational malfunctions could not be avoided. This finding conflicts with the objective of providing the required water quality efficiently. Nevertheless, public health protection is a priority and needs to be guaranteed. Redundancy assures the reliability of *E. coli* reduction.

Possible water reuse purposes are primarily determined by whether successful removal of helminth eggs is achievable or not. Helminth eggs could not be removed to the required degree during wastewater treatment, but were completely retained in the storage pond. Thus, the helminth egg concentrations are decisive and, for irrigation of crops eaten raw, the water should only be extracted from the storage pond. If irrigation water contains helminth eggs and storage is not possible, or the pond is frequently bypassed, the water should be used only for drip irrigation of high-growing crops. Irrigation of fodder crops and pasture is an option for effluent water without a prevalence of *Taenia* spp.

Anaerobic pretreatment of domestic sewage reduces alkalinity and usually leads to an effluent pH between 6.6 and 7.4 (Chernicharo 2007). Alkalinity is further reduced during nitrification. If the alkalinity of untreated wastewater is low, the pH can drop significantly and show marked variation. A low pH may be harmful to irrigation equipment. Because of the expected excess of N and a low pH, a denitrification step should be included when planning treatment plants for the reclamation of nitrogen-rich water with low alkalinity. If N should remain in the water, liming or blending with other water sources could be used for pH adjustment. In general, the effect of anaerobic pretreatment and aerobic treatment on pH and alkalinity needs to be taken into account.

Initially, the storage pond was included in the water reuse project to compensate for the gap between irrigation water supply and demand. However, it turned out to be a necessity to achieve the required water quality. Public health aspects and the lower corrosiveness of the water prescribe that irrigation water is extracted only from the storage pond.

The general approach for defining water quality criteria for a specific project should be to use the limits presented in the FAO (1985) guidelines for prevention of soil salinization (EC, TDS) and prevention of toxic effects on plants (Na, B, Mn, Cl, trace elements), for the protection of irrigation infrastructure (TSS, pH) and to maintain sufficient soil infiltration (SAR). The WHO (2006) guidelines should be used for choosing an adequate approach for public health protection and defining limits for *E. coli* and helminth eggs. The recommendations in this study should be used to include wastewater-related parameters and to develop site-specific water quality limits for protection of irrigation infrastructure (turbidity, TCOD, BOD_5), the required water quality prior to UV disinfection (turbidity, TSS, particle size) and prevention of eutrophication and negative effects on plants (TN, TP and K). Water storage facilities should be considered as an additional treatment step that contributes to the reliability of the water reclamation process and to achieving the required water quality.

Realization of water reuse projects can be facilitated by providing more detailed information on water quality requirements to relevant stakeholders. The parameters contained in the FAO (1985) guidelines provide a basis for monitoring irrigation water quality and should be further extended to include the wastewater-related parameters presented in this study. More detailed information on the required maximum particle content and suitable monitoring parameters prior to disinfection steps (UV, chlorine, chlorine dioxide, ozone) and on different types of irrigation infrastructure (drip irrigation, subsurface irrigation, sprinkler systems) is needed. Further characteristics of the irrigation site, such as soil conditions and climate, should be taken into account. This will facilitate water quality monitoring in water reuse schemes and assist in providing acceptable irrigation water.

4.6 Salt and nutrient management

This section focuses on the nutrient and salt content of the water flows and how nutrients and salts can be managed in this water reuse scheme. TDS, EC, TN, TP and K are addressed consecutively. Each section starts by describing the figures used for the project design. Then, monitoring results on water quantities, salts and nutrients are presented and compared; differences are discussed and possible options for the control of input to the agricultural area are outlined. The final section offers some general recommendations on planning and implementation of measures for salinity and nutrient management when reusing water for agricultural irrigation.

4.6.1 Electrical conductivity and total dissolved solids

4.6.1.1 Planning data

Figure 66 shows the components of the water reuse scheme, water quantities, EC, TDS concentrations and loads, and the factors affecting increase and decrease. The infrastructure was designed for up to 1,500 users and a water use of 60 L/(person×d). Thus, tap water use was assumed to be 90 m³/d or 32,850 m³/a (Table 28).

During planning, EC of the tap water in Outapi was measured in one grab sample. Its TDS concentration was estimated using a TDS/EC conversion factor of 0.625 mg×cm/(L×µS) (Eaton and Franson 2005). The calculated TDS content for the measured EC of 75.0 µS/cm is 46.9 mg/L (= 0.625 mg×cm/(L×µS) × 75 µS/cm, see Table 28). 32,850 m³ of tap water are used every year (= 60 L/(person×d) × 1,500 persons × 365 d/a). The TDS load in the tap water is then 1.5 t/a (= 32,850 m³/a × 46.9 mg/L ÷ 10^6 L×t/(m³×mg)).

During water use, 65.7 t TDS/a are added to it in the sanitation facilities while it is used for toilet flushing, showering and laundry washing (Figure 66). Accordingly, EC and TDS increase to 2,047 mg/L and 3,275 µS/cm (= 67.24 t/a × 10^9 mg/t ÷ (0.625 mg×cm/(L×µS) × 32,850 m³/a × 1,000 L/m³)).

During wastewater treatment, excess sludge is generated in the UASB reactors and in the RBCs. Mineral salts incorporated into cell biomass are removed from the water. The mineral content of cell biomass is estimated to equal 19% of the dry weight (Tchobanoglous *et al.* 2004). Sludge production (cell biomass) is 15.1 kg TSS/d in the RBCs and 22.5 kg TSS/d in the UASB reactors; thus, in total, 13.7 t TSS/a (see Table 43, page 167 for detailed calculation). Then, 2.6 t TDS/a are removed from the water via incorporation into biomass (= 0.19 × 13.7 t TSS/a). However, this constitutes only 3.9% of the TDS load contained in the untreated water. Apart from TDS contained in excess sludge, a change in EC and TDS was not assumed within the wastewater treatment plant because salts are not removed and flocculants or other chemicals are not added.

In the storage pond, algal and bacterial biomass is produced (Gloyna 1971). Algae concentrations in pond systems are usually between 50 and 70 mg/L (Walmsley and Shilton 2005). Thus, the expected concentration of algal biomass is estimated at 60 mg/L dry weight. Assuming the same mineral content for algal biomass and cell biomass in excess sludge, the TDS content of algae is 0.3 t/a (= 60 g/m³ × 28,908 m³/a × 0.19 ÷ 10^6 g/t).

Similar to excess sludge production in the UASB reactors, bacterial cell mass production is estimated using the TCOD load applied to the storage pond: 6.3 kg/d (= 70 mg/L × 90,000 L/d, Table 11, page 89). 345 kg TSS of bacterial biomass develop in the pond each year (= 6.3 kg TCOD/d × 0.15 kg TCOD applied to the system × 365 d/a). The TDS content is 65.5 kg/a (= 365 kg TSS × 0.19). This is less than 1% of the TDS load discharged to the storage pond. Altogether, 0.4 t TDS/a are removed in the pond via incorporation into bacterial and algal biomass (Figure 66). The TDS load is reduced to 64.2 t/a. Due to evaporation, the EC increases to 3,555 µS/cm.

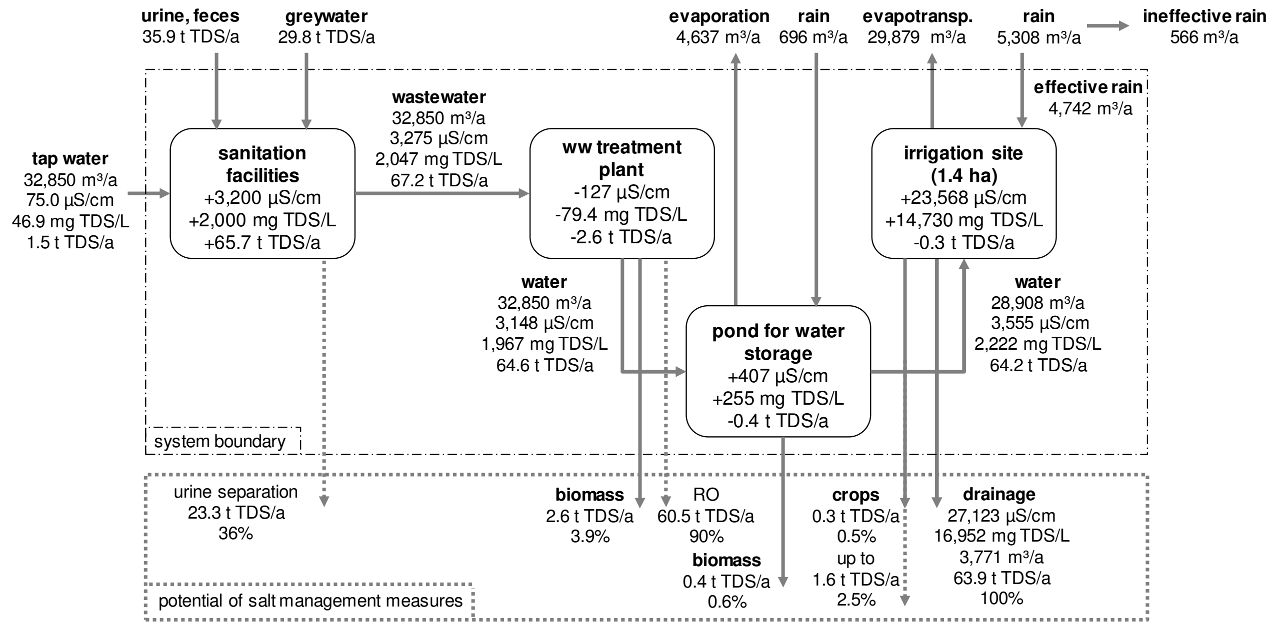

Figure 66 Planned water quantities, TDS and EC for the Outapi water reuse scheme; all values based on project design and literature data (Table 28 and Table 32), ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TDS management, dash-dotted lines = system boundary

Data for evaporation and rainfall were taken from Mendelsohn *et al.* (2000). Effective rainfall – that part of the rainfall that can be utilized by the crop – was estimated according to Savva and Frenken (2002). Loss is caused by evaporation, infiltration below the root zone, and surface runoff (Savva and Frenken 2002).

A cropping pattern achieving high revenues on local markets and producing biomass for co-digestion with sewage sludge was chosen for implementation (Woltersdorf *et al.* 2015). This cropping pattern includes maize, peppers, pumpkins, spinach, tomatoes, sweet melons, and watermelons (Figure 67). The water demand of these crops was modelled by Woltersdorf *et al.* (2015) for the available agricultural area in Outapi using the FAO software CROPWAT 8.0 (Smith 1992).

Leaching may be required to prevent accumulation of salts in the soil (Ayers and Westcot 1985). The required leaching fraction can be calculated using the salinity of the irrigation water and the average soil salinity tolerated by the crops (Ayers and Westcot 1985). To estimate the amount of drainage water, a leaching fraction of 0.15 of the irrigation demand, as recommended by Ayers and Westcot (1985) was used in this case. When choosing crops for cultivation, their salt sensitivity needs to be considered.

Table 28 Data used for calculation of TDS and EC during planning

	value	unit	source
water quantities			
users	1,500	persons	project design value
water use	60	L/(person×d)	project design value
pond surface area	1,855	m²	project design value
rain	375	mm/a	Mendelsohn *et al.* (2000)
evaporation	2,500	mm/a	Mendelsohn *et al.* (2000)
irrigation demand	17,760	m³/(ha×a)	Woltersdorf *et al.* (2015)
leaching fraction	15	%	Ayers and Westcot (1985)
EC and TDS			
EC tap water	75	µS/cm	grab sample (n = 1)
TDS/EC	0.625	mg×cm/(L×µS)	Eaton and Franson (2005)
TDS input (via urine, feces and greywater)	120	g/(person×d)	Sperling (2007c)
TDS urine	44	%	Jönsson *et al.* (2005), DWA (2008c)
TDS feces	10	%	Jönsson *et al.* (2005), DWA (2008c)
TDS greywater	45	%	Jönsson *et al.* (2005), DWA (2008c)

Considering the leaching requirement, the available amount of irrigation water (rain (pond) - evaporation (pond) + effective rain (irrigation site) - drainage water) is 28,908 m³/a. The irrigation demand is 17,760 m³ per ha and year (Woltersdorf *et al.* 2015). Thus, the available quantity of irrigation water is sufficient for irrigating 1.4 ha.

Crop residues are co-digested with sewage sludge and used as fertilizer and bulking material on the agricultural site. Thus, the TDS contained in crop residues remains within the system boundary. TDS is only removed by selling harvested parts of the crop. However, the TDS load removed in this way is negligible. It was estimated at 0.3 t TDS/a (on 1.4 ha) or 0.5% of the TDS load (Table 29). Thus, about 63.9 t of TDS would accumulate on the irrigated area every year if measures for salinity management are not implemented (Figure 66).

field	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	1st	maize			20th		1st	maize			17th	
2		1st	maize			20th		1st	maize			18th
3	1st	peppers		30th	10th	pumpkin		17th	1st	spinach		9th
4	1st	tomatoes			15th	10th	weet melons			7th 10th	watermelons	28th

Figure 67 Cropping scheme for the irrigation site in Outapi (Woltersdorf *et al.* (2015), modified)

In addition, TDS concentrations or EC could cause yield loss in salt-sensitive crops. Water with an EC above 2,250 µS/cm has a very high salinity hazard (Richards 1954). Above an EC of 3,000 µS/cm, severe restrictions regarding its use for irrigation apply (Ayers and Westcot 1985). If applied directly, it can cause yield losses of roughly 50% for maize and peppers and

25% for tomatoes, spinach and pumpkins (Ayers and Westcot 1985). This should be considered when cultivating salt-sensitive crops. Dilution of the water is required to achieve optimal yields.

Table 29 TDS removal via crops chosen for Outapi, average yield for Namibia: PWC (2005), harvests per year and field size: Woltersdorf *et al.* (2015) and Figure 67, ash content: USDA (2014)

	field number	**yield t/(ha×harvest)**	**field size ha**	**harvests 1/a**	**ash content % (weight)**	**TDS removed t/a**
maize	1 and 2	8	0.5	2	0.62	0.05
peppers	3	12	0.25	1	0.43	0.01
pumpkins	3	35	0.25	1	0.80	0.07
spinach	3	17	0.25	1	1.72	0.07
tomatoes	4	70	0.25	1	0.50	0.09
watermelons	4	35	0.25	1	0.25	0.02
sweet melons	4	35	0.25	1	0.25	0.02
total		**59.0**	**1.5**			**0.3**

4.6.1.2 Monitoring data

During the survey period, the average water quantity in the effluent of the wastewater treatment plant was 30.3 m³/d or 11,068 m³/a. This was only one third of the expected volume and was due to a lower total number of users and lower utilization of the infrastructure (see Chapter 4.3, page 93*ff.*).

The precipitation and evaporation were monitored from October 2012 to July 2015. The mean precipitation was 333 mm/a and the mean evaporation was 1,966 mm/a (Figure 68 and Figure 69). This was lower than in the literature, which reports values of 375 mm/a (precipitation) and 2,500 mm/a (evaporation) (Mendelsohn *et al.* 2000). The monitored rainfall showed a much higher variation than the monitored evaporation, which is in agreement with Mendelsohn *et al.* (2000).

EC was 51.9 µS/cm in tap water, 612 µS/cm in the influent of the wastewater treatment plant, 527 µS/cm in the effluent of the wastewater treatment plant, and 596 µS/cm in the storage pond (Table 26, page 123 and Table 28, page 139). TDS was 40.2 mg/L in the tap water, 375 mg/L in the effluent of the wastewater treatment plant, and 455 mg/L in the storage pond (Figure 70, Table 30). TDS was not measured in the untreated water due to rapid clogging of the filters (see Section 3.3, page 46). Hence, the same TDS concentration as in the treated water was assumed.

The monitored EC and TDS values were 31% and 14% lower in tap water and roughly 80% lower in the untreated wastewater, in the effluent of the wastewater treatment plant, and in the irrigation water than anticipated during planning. This gap is due to lower overall utilization of the sanitation facilities and relatively high water use in the shared sanitation facilities in combination with incomplete excreta collection (see Chapter 4.3, page 93*ff.*).

The TDS load of the water increases during its use in the sanitation facilities (+3.6 t/a). This causes an EC and TDS increase there (+560 µS/cm and +335 mg/L). The decrease in EC during wastewater treatment (-85 µS/cm) can be explained by the consumption of HCO_3^- during nitrification (see Chapter 4.2.3, page 71*f.*). As shown previously, the amount of TDS incorporated into biomass is negligible. Thus, TDS did not change significantly during wastewater treatment.

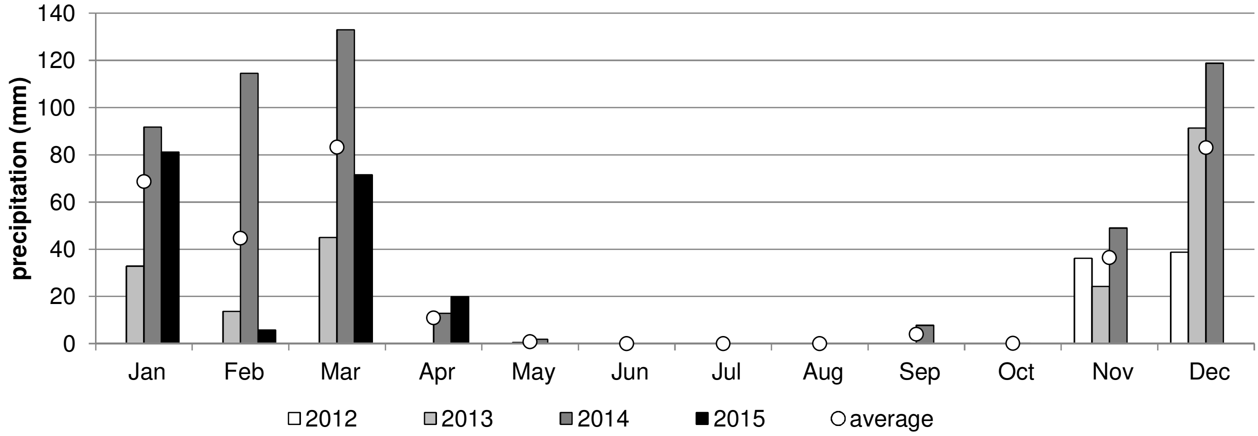

Figure 68 Monitored precipitation at the irrigation site in Outapi (October 2012 to July 2015)

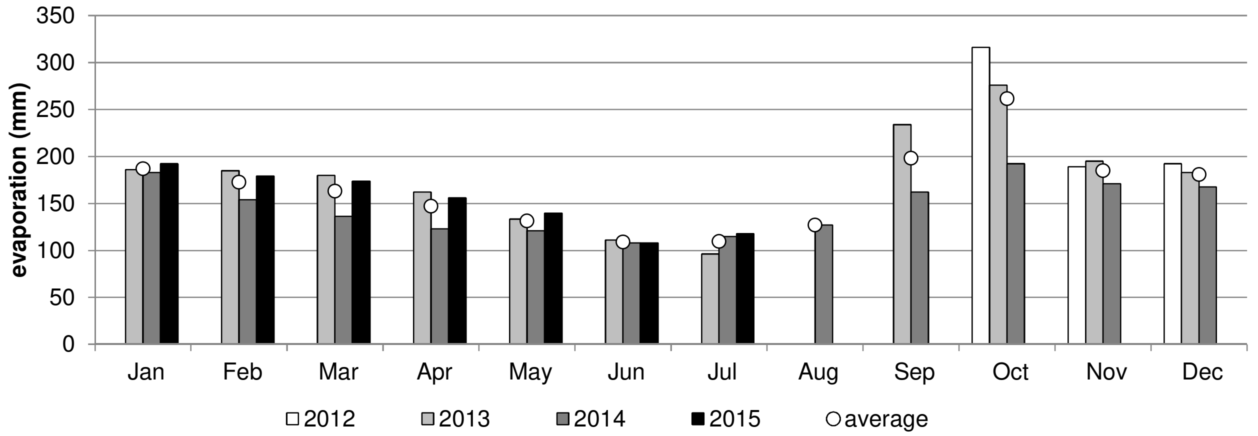

Figure 69 Monitored evaporation at the irrigation site in Outapi (October 2012 to July 2015)

EC and TDS increase in the storage pond due to evaporation (+69.0 µS/cm and +80.2 mg/L). The TDS load is reduced by 0.5 t/a and, thus, to a higher degree than presumed during planning. Therefore, 12% of the TDS load is removed in the pond (compared to 0.6% in the planning case).

The calculated size of the irrigable area is 0.4 ha (= (11,068 m³/a - 3,647 m³/a + 617 m³/a - 1,048 m³/a) ÷ 17,760 m³/(ha×a), = (treated water - evaporation storage pond + rain storage pond - leaching requirement) ÷ irrigation requirement). After implementation, 3 ha were cultivated with additional tap water (Zimmermann *et al.* 2017b). Hence, the reclaimed water is used promptly and without long storage. Bias in the samples due to dilution with rainwater is very likely (see Section 4.2.7, page 80). Thus, the EC increases only to a minor degree, from 527 µS/cm to 596 µS/cm.

If the irrigable area was only 0.4 ha, the EC would, by calculation, increase up to 726 µS/cm (= 527 µS/cm × 11,068m³/a ÷ 17,760 m³/(ha×a) × 0.4 ha). There would be no or only slight

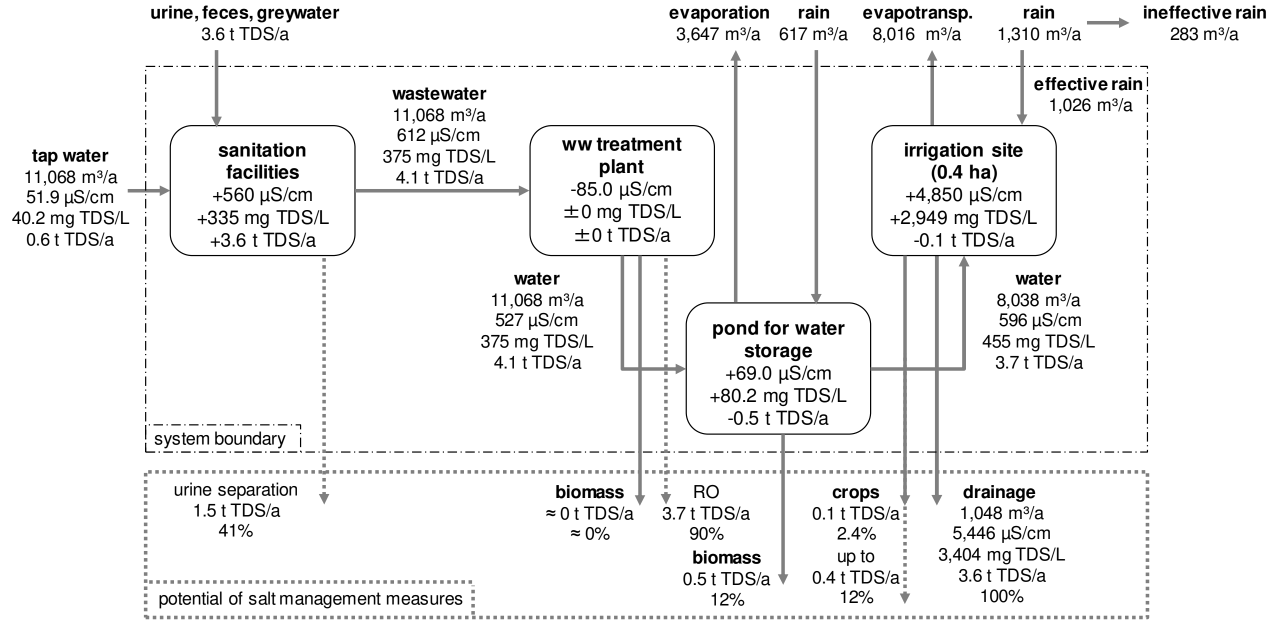

Figure 70 Water quantities, TDS and EC after implementation of the water reuse scheme in Outapi, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TDS management, dash-dotted lines = system boundary

limitations in crop choice due to the salt content of the water (Ayers and Westcot 1985). If drainage is not applied, the TDS load in the irrigation water would be 3.7 t/a for an irrigable area of 0.4 ha or 9.3 t/(ha×a).

The dilution with rainwater and the prompt use of the water without longer storage and lower evaporation could also explain the relatively high calculated percentage of TDS removal. Assuming a lower evaporation from the pond`s surface of 2,736 m³/a (instead of 3,647 m³/a, thus -25%, reason: shorter storage time and less evaporation), the irrigation water would contain 4.1 t TDS/a, which corresponds to a reduction in the pond by 0.1 t TDS/a and represents a reduction of only 1.8%. This would be within the scale of the planning data.

Table 30 Data used for calculation of water quantities and TDS loads after implementation of the water reuse scheme, wwtp = wastewater treatment plant

	value	unit	source
water quantities			
water use	11,068	m³/a	monitoring, see, Figure 30, page 69
pond area	1,855	m²	monitoring, see Section 4.1.6, page 66
rain	333	mm/a	monitoring, see Figure 68, page 141
evaporation	1,966	mm/a	monitoring, see Figure 69, page 141
irrigation demand	17,760	m³/(ha×a)	Woltersdorf *et al.* (2015)
leaching fraction	15	%	Ayers and Westcot (1985)
EC and TDS			
EC tap water	51.9	µS/cm	monitoring, see Figure 33, page 72
EC effluent wwtp	527	µS/cm	monitoring, see Table 26, page 123
EC pond	596	µS/cm	monitoring, Table 26, page 123
TDS tap water	40.2	mg/L	monitoring, see this section
TDS effluent wwtp	375	mg/L	monitoring, see this section
TDS pond	455	mg/L	monitoring, see this section

In the Outapi case, salt management was carried out via regular drainage and leaching of the fields. A drainage system was implemented that discharges drainage water to a basin, where the water evaporates. As an additional measure, blending of the irrigation water with tap water is possible, if required.

4.6.1.3 Options for salinity management

It is obvious that TDS loads need to be removed for sustainable agricultural irrigation. Strategies for soil salinity control were outlined in Section 2.5 (page 24*f.*). One option to reduce salt input to the fields is the separate collection of urine. Urine contains 44% of the TDS load in wastewater (Table 31). It contains 90% of the common salt (Powles *et al.* 2013; Sherwood 2006), 80% of the nitrogen (DWA 2008c; Johansson 2000), and 50% of the phosphorus excreted by humans (DWA 2008c). On average, 80% of excreted urine is collectable (Johansson 2000). Consequently, source separation of urine could be applied to reduce the amount of salts in the water.

Table 31 TS, TSS and TDS in urine, feces and greywater, TS = total solids, TSS = total suspended solids, TDS = total dissolved solids

	TS (DWA 2008c)		**TSS (Jönsson *et al.* 2005)**		**TDS (calculated)**	
	g/(person×d)	**%**	**g/(person×d)**	**%**	**g/(person×d)**	**%**
urine	57	34	0.8	1.9	56	44
feces	38	23	25	64	13	10
greywater	71	43	14	34	58	45
total	166	100	39	100	127	100

In theory, 23.3 t or 36% (= 0.8 × 53 g/(person×d) × 1,500 persons) of the TDS load (67.2 t/a) could be collected (Figure 66). In Outapi, urine-diverting toilets would be feasible in the shared sanitation facilities, but it is difficult to prescribe installation to individual households. Considering this for the design data, only 64% of urine could be collected per year (0.8 × (250 + 840 users) ÷ 1,354 users = 0.64 (for number of users see Table 19, page 108). Thus, only 18.7 t TDS could be removed per year.

To reduce salt input to the field, the collected urine would have to be disposed of, or used outside the system boundaries. Nutrients would also be lost. Since recipients for reasonable use of the urine outside the system boundaries could not be identified during planning, urine separation was not pursued for the Outapi sanitation system. Technical considerations also played a role (e.g., need for double piping, further processing, storage, and transport of urine, potential NH_3-emissions).

As discussed in Section 4.3 (page 93*ff.*), the shared sanitation facilities were mainly used for defecation and laundry washing. Considering TDS, the percentage of the collected specific loads varied from 2% at the cluster units, 80% at the communal washhouse and 32% at the individual sanitation facilities. Because implementation of urine-separating toilets would only be possible at the cluster units and the communal washhouse but not at the individual households, this salt management option would be ineffective.

Among the considered options for salinity removal during wastewater treatment (Section 2.5, page 25), reverse osmosis membrane filtration is most suitable for salt removal during wastewater treatment. In this case, salt input could be reduced by 60.5 t/a (literature data) or 3.7 t/a (monitoring data). Nevertheless, the accumulation of salts on the fields would be considerable in the long term. Brine disposal needs to be considered. In this case, the brine could be disposed of in the evaporation pond, where drainage water from the agricultural fields is collected. The water quantity available for irrigation and the nutrient content of the water would be reduced by the proportion of water and nutrients contained in the brine. Furthermore, membranes appeared to be too expensive in terms of operation, too energy-intensive and would have increased the complexity of wastewater treatment. Therefore, they were not selected for Outapi.

Crop residues were intended to be co-digested and biosolids were to remain on the agricultural field as fertilizer and bulking material. Harvested crops were sold directly on site or at local markets. Salts contained in the crops would partially re-enter the sanitation system and partially leave it. Data on the quantification of salt uptake by agronomic crops are scarce in the literature. Richards (1954) and Ayers and Westcot (1985) exclude this topic. FAO (2003) recommends yearly or periodical cultivation of salt harvesting crops, such as sudax, barley, bermuda grass, and sorghum, to reduce salinity build-up in soil, but does not give detailed information.

The ash content after ignition at 600°C represents the total mineral content of a food sample (Nielsen 2014). Similar to the determination of fixed solids in wastewater and sludge, volatile matter, such as organics and some mineral salts, is lost (Eaton and Franson 2005; Nielsen 2014).

The majority of vegetables and fruits has a mineral content below 1% (Table 32). The amount of TDS removed per hectare and year depends on the mineral content of each crop, the yield per hectare, and the number of harvests per year. Crops with a high yield, a high mineral content, and a short individual growth period have the highest potential to remove TDS from the agricultural area. The data suggest that spinach, tomatoes, and pumpkins remove the largest amount of TDS from agricultural fields. The removal potential is much lower for e.g., grapes, oranges, and peppers (Table 32).

For an irrigable area of 1.4 ha, if the whole area is cultivated with crops that remove a relatively large TDS load, e.g., spinach or tomatoes, the maximum TDS removal via harvested crops would be 1.6 t/a or 2.5% (literature data, Figure 66, page 138). For the smaller agricultural area that is irrigable with the lower monitored water quantities, 0.4 t/a or 12% TDS could be removed (Figure 70, page 142). However, the actually implemented cropping pattern consists of crops that remove relatively low TDS loads, on average, only 0.22 t TDS/(ha×a). This is 0.3 t TDS/a or 0.5% for planning data and 0.1 t TDS/a or 2.4% for monitoring data. Thus, TDS removal via harvested crop parts has an obvious effect on the TDS balance only if overall TDS loads are relatively low. This is in accordance with a review given by Heuperman *et al.* (2002) who conclude that salt removal by crops is only significant for irrigation water with relatively low TDS content.

Table 32 Calculation of the TDS removal potential for some vegetables and fruits, average yield for Namibia: PWC (2005), total duration of individual growth period: Doorenbos (1979), ash content: USDA (2014)

crop	**yield**	**duration of individual growth period**	**harvests**	**ash content**	**TDS removed**
	t/(ha×harvest)	d	1/a	% (weight)	t/(ha×a)
spinach	17	95	3.8	1.7	1.12
tomatoes	70	115	3.2	0.5	1.11
pumpkins	35	95	3.8	0.8	1.08
cabbage	55	125	2.9	0.6	1.03
potatoes	35	125	2.9	1.0	1.02
beans	13	75	4.9	0.7	0.42
watermelons	35	95	3.8	0.3	0.34
onions	28	120	3.0	0.4	0.30
wheat	6	115	3.2	1.5	0.29
maize	8	120	3.0	0.6	0.15
peppers	12	135	2.7	0.4	0.14
oranges	16	300	1.2	0.6	0.12
grapes	12	225	1.6	0.5	0.09

Some publications report much higher TDS removal potentials for edible crops that are less commonly cultivated. Borage is reported to contain minimum values of 2.0% and 1.5% (dry weight) of Na^+ and Cl^- under non-saline conditions and up to 9.1% and 5.7% of Na^+ and Cl^- for higher EC (15 dS/m = 15,000 µS/cm) (Badi and Sorooshzadeh 2010). USDA (2014) reports an ash content of 1.4% for borage. The yield of borage is relatively high, with 25 to 100 t/ha, and the individual growth period of 50 to 120 days is quite short (Hernández Bermejo and León 1994). The TDS removal potential could be between 1.5 (= 25 t/(ha×harvest) × 4.3 harvests/a × 1.4% ash content) and 5.4 t/(ha×a) (= 62.5 t/(ha×harvest) × 4.3 harvests/a × 2% ash content). This would be much higher than most of the values in Table 32.

Suaeda fruticosa (shrubby sea-blite) can remove 3.0 t salt per hectare (Chaudhri *et al.* 1964). 55% is contained in the edible leaves, thus the TDS removal potential is 1.7 t/ha (Chaudhri *et al.* 1964). Removal of up to 5.0 t/(ha×a) could be achieved (Chaudhri *et al.* 1964). Other edible crops with a high TDS removal potential are *Portulaca oleracea* (purslane) and *Tetragonia tetraonoides* (New Zealand spinach) or the fodder crop *Beta maritima* (sea beet) (Aksoy *et al.* 2003). Altogether, it seems possible to remove up to 5 t TDS/(ha×a). This would increase the proportion of removable TDS to 10% (literature data) and 48% (monitoring data).

In regions with alternating wet and dry seasons (as in North Namibia), leaching during wet season can contribute to TDS removal. Mechilia (2002) concludes that 500 mm precipitation leads to salt removal in the upper soil layer (0-125 cm), and more than 600 mm are needed for leaching to a depth of 200 cm. In another study, 500 mm is reported to be sufficient for leaching, if precipitation occurs in a short time frame (4 months) (Ben-Hur 2004). In the case of the

Sacramento-San Joaqin Delta in California, even 400 mm of rainfall was sufficient for salt leaching (Ayers and Westcot 1985).

Precipitation in Outapi is 375 mm on average, but varies considerably (Mendelsohn *et al.* 2000). Since 1940, roughly 40% of the recorded years had precipitation above 500 mm (Mendelsohn *et al.* 2000). Thus, leaching during the rainy season might be sufficient to remove salts from the root zone in some years.

The monitored rainy seasons had rainfalls of 167 mm (2012/13), 477 mm (2013/14) and 354 mm (2014/15) (Figure 68, page 141). In 2013/14 and 2014/15 precipitation might have been sufficient to leach salts. However, 250 mm of precipitation are not enough for sufficient leaching (Melgar *et al.* 2009) and it is very likely that rainfall was not adequate in 2012/13.

4.6.2 Total nitrogen

4.6.2.1 Planning data

The TN load was estimated at 8.0 g/(person×d) (Sperling 2007c). During water use, 4.4 t TN are added per year (Figure 71). The TN concentration in the untreated water is 133 mg/L. This concentration is reduced to 83 mg/L and 2.7 t/a in the effluent of the wastewater treatment plant, due to incorporation of nitrogen into biomass during anaerobic and aerobic treatment (N content of cell biomass = 12% of the dry weight (Tchobanoglous *et al.* 2004), sludge production in the UASB reactors = 22.5 kg TSS/d, sludge production in the RBCs = 15.1 kg TSS/d (Table 43, page 167)).

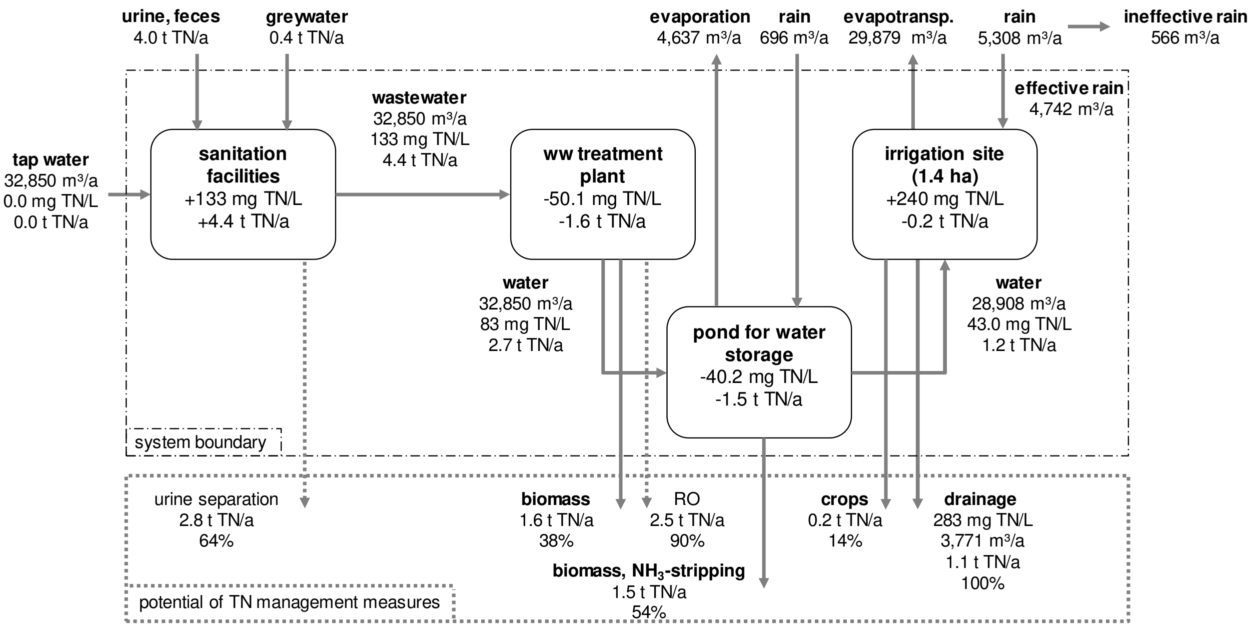

Figure 71 Water quantities and TN loads and concentrations according to planning data, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TN management, dash-dotted lines = system boundary

During water storage, nitrogen can be removed via ammonia stripping, assimilation by algae, sedimentation of particulate nitrogen, and nitrification/denitrification (Sperling 2007b). The most important mechanisms are ammonia stripping and assimilation by algae (Sperling 2007b). The nitrogen assimilated by algae would end up in the sludge on the bottom of the pond or is removed in disc filters that were installed to protect the drip irrigation system from particles. In the long term, algal and bacterial biomass removed from the pond`s effluent needs to be disposed of outside the system boundary, if it is considered as a sink for nitrogen. If the sludge is used as a fertilizer on the agricultural fields, the contained nitrogen would remain within the system boundaries.

The nitrogen incorporated into algal and bacterial biomass was estimated as follows. The bacterial biomass production in the pond is 345 kg TSS/a (= 0.15 kg TSS/kg COD × 6.3 kg TCOD/d × 365 d/a, see previous section). The N content is assumed to be 12% (Tchobanoglous *et al.* 2004). Thus, the N content of this biomass is 41.4 kg/a (= 345 kg/a × 0.12). The algae concentration in the effluent is estimated at 60 mg/L dry weight (Walmsley and Shilton 2005). The algal biomass is estimated at 1.7 t/a (= 60 g/m³ × 28,908 m³/a ÷ 10^6 g/t). Its nitrogen content is 9% (Arceivala 1981). Thus, the N content of algal biomass is 156 kg/a (=1.734 t/a × 0.09). In total, 197 kg N would be incorporated into algal and bacterial biomass each year.

Assuming a pH of 8.0 in the storage pond, a surface area of 1,855 m² (see Chapter 4.1.6, page 66 for pond details), an average water temperature of 20°C, and a hydraulic detention time of 41 days (= 3,712 m³ ÷ 90 m³/d), the TN concentration is reduced to 43.9 mg/L, due to ammonia stripping (= 83 mg/L ÷ (1 + (5.035 × 10^{-3} × (1,855 m² ÷ 90 m³/d) × $e^{(1.54 \times (8.0-6.6))}$)); the empirical formula given in Sperling and Chernicharo (2005)). Thus, 1.3 t TN/a are removed via ammonia stripping.

In view of TN reductions due to incorporation into biomass, ammonia stripping, and water loss due to evaporation, the final TN concentration in the irrigation water is 43.0 mg/L. Thus,

Table 33 Calculation of the TN removal potential for some vegetables and fruits, average yield for Namibia: PWC (2005), total duration of the individual growth period: Doorenbos (1979), protein content and nitrogen-to-protein conversion factor: USDA (2014)

crop	**yield**	**duration of individual growth period**	**harvests**	**protein content**	**conversion factor**	**N content**	**N removed**
	t/(ha×harvest)	d	1/a	% (weight)	-	% (weight)	t/(ha×a)
wheat	6	115	3.2	9.6	3.6	2.7	0.51
potatoes	35	125	2.9	2.1	6.3	0.3	0.34
maize	8	120	3.0	3.3	2.4	1.3	0.33
cabbage	55	125	2.9	1.3	6.3	0.2	0.32
tomato	70	115	3.2	0.9	6.3	0.1	0.31
spinach	17	95	3.8	2.9	6.3	0.5	0.30
pumpkin	35	95	3.8	1.0	6.3	0.2	0.22
beans	13	75	4.9	1.8	6.3	0.3	0.19
onion	28	120	3.0	1.1	6.3	0.2	0.15
watermelon	35	95	3.8	0.6	6.3	0.1	0.13
peppers	12	135	2.7	0.9	6.3	0.1	0.04
orange	16	300	1.2	1.3	6.3	0.2	0.04
grapes	12	225	1.6	0.7	6.3	0.1	0.02

Table 34 Nitrogen removal by the crops chosen for Outapi, average yield for Namibia: PWC (2005), harvests per year and field size: Woltersdorf *et al.* (2015), protein content and conversion factor: USDA (2014)

crop	**field number**	**yield**	**field size**	**protein content**	**conversion factor**	**N content**	**harvests**	**N removed**
		t/(ha×harvest)	ha	% (weight)	-	% (weight)	1/a	kg/a
maize	1 and 2	8	0.5	3.3	2.4	1.3	2	107
peppers	3	12	0.25	0.9	6.3	0.1	1	4.1
pumpkin	3	35	0.25	1.0	6.3	0.2	1	14.0
spinach	3	17	0.25	2.9	6.3	0.5	1	19.4
tomato	4	70	0.25	0.9	6.3	0.1	1	24.6
watermelon	4	35	0.25	0.6	6.3	0.1	1	8.5
sweet melon	4	35	0.25	0.6	6.3	0.1	1	8.5
total			**1.5**					**187**

roughly half of the load and concentration is removed in the pond. 1.2 t TN/a are applied to the irrigation site.

The nitrogen removal potential is relatively high for, e.g., wheat, potatoes, and maize and relatively low for, e.g., grapes, oranges and peppers (Table 33). For the cropping pattern implemented in Outapi, the estimated N removal via crops is 187 kg per 1.5 ha or 124 kg/(ha×a) (Table 34). The irrigable area for planning data is 1.4 ha. Thus, roughly 174 kg TN can be removed via crops each year and 1.5 t TN/a remain unused. This load is leached from the fields via drainage water and is collected in the evaporation pond, where it is dried by the sun.

A larger area could be fertilized with the TN load. Assuming a concentration of 25.0 mg/l as a water quality objective for the requirements of tomatoes (Table 27, page 126), an additional area of 2.5 ha could be fertilized with this load. An additional water quantity of 44,000 m³/a or 117 m³/d would be needed for optimal use of the nitrogen (44,000 m³/a = 1,100 kg TN/a ÷ 0.025 kg/m³; 2.5 ha = 44,000 m³/a ÷ 17,760 m³/a).

Beside incorporation into biomass, algae, ammonia stripping, and uptake in crops, TN can be reduced via reverse osmosis membrane filtration and urine separation. Reverse osmosis membrane filtration has the highest potential for nitrogen removal (-2.5 t/a or -90%). Separation of urine and its disposal or use outside the system boundaries could reduce the nitrogen load by 2.8 t/a or 64%.

4.6.2.2 Monitoring data

The TN loads and concentrations after implementation are shown in Figure 72. In the sanitation facilities, the TN concentration increased by 57.5 mg/L. The TN load was 0.6 t per year. Thus, the concentration represented only 43% and the yearly load only 15% of the planning data.

During wastewater treatment, the TN concentration decreased by 24.0 mg/L, which is almost the same as anticipated during planning. Altogether, 0.3 t TN/a were removed via sedimentation or incorporation into biomass (30% of the planning data). The concentration in the effluent of the wastewater treatment plant was 33.5 mg/L (30% of the planning data).

In the storage pond, the concentration decreased slightly to 32.6 mg/L. The TN load was reduced from 0.4 t/a to 0.3 t/a. Thus, in the storage pond, the concentration decreased by 3% (compared to 47% as assumed during planning) and the load decreased by 29% (compared to 53% as assumed during planning). The lower relative TN loss was due to less ammonia stripping, because most of the nitrogen was nitrified during wastewater treatment and oxidation, which continued in the storage pond.

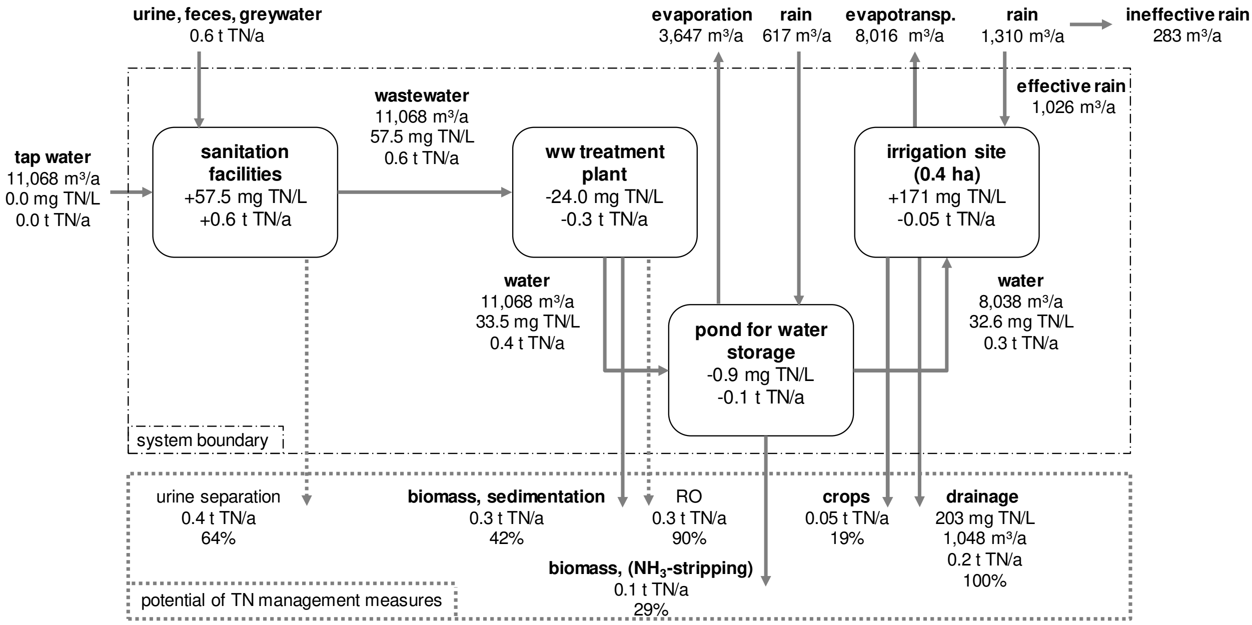

Figure 72 Water quantities and TN loads and concentrations according to monitoring data, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TN management, dash-dotted lines = system boundary

TN removal in harvested crops (cultivated on 0.4 ha) is estimated at 0.05 t/a. This is less than planned, due to the smaller size of the agricultural area. Excess nitrogen is removed from the agricultural field in the drainage water. It amounts to 0.2 t/a (15% of planning data) and would be sufficient to fertilize an additional area of 0.5 ha. The additionally required water quantity would be about 8,524 m³/a or 23.4 m³/d.

Reverse osmosis membrane filtration has the highest potential for nitrogen removal (0.3 t/a or 90%), followed by urine separation (0.4 t/a or 64%), incorporation into biomass and sedimentation during wastewater treatment (0.3 t/a or 42%), removal in the pond (0.1 t/a or 29%), and removal via parts of harvested crops (0.05 t/a or 19%).

4.6.3 Total phosphorus

4.6.3.1 Planning data

During planning, the phosphorus load was estimated at 1 g/(person×d) (Sperling 2007c). The fractions contained in urine, feces, and greywater were estimated at 0.5, 0.25 and 0.25 g/(person×d), respectively, using the distribution given in DWA (2008c). The overall P quantities are much lower than for TDS and N. Excreta contribute 411 kg TP/a and greywater contributes 137 kg TP/a (Figure 73). During water use, TP concentrations increase by 16.7 mg/L.

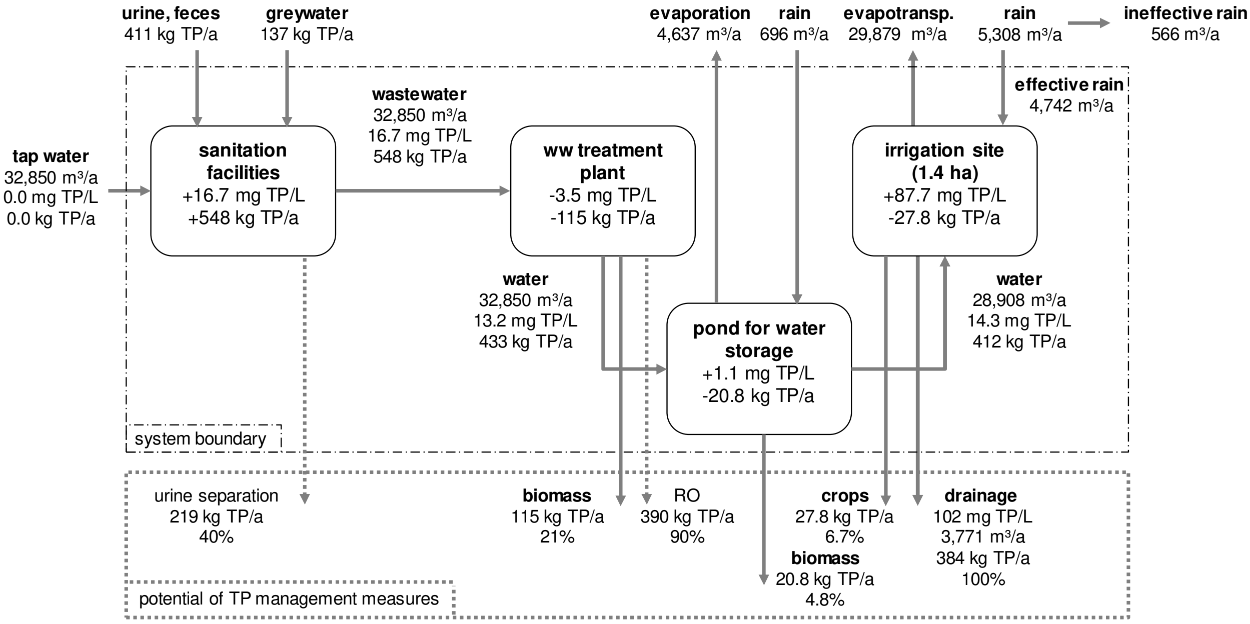

Figure 73 Water quantities, TP loads and concentrations according to planning data, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TP management, dash-dotted lines = system boundary

During wastewater treatment, its concentration is reduced by 3.5 mg/L and the load is reduced by 115 kg/a, due to incorporation into bacterial biomass during anaerobic treatment (1.2 g P per kg COD removed, Bischofsberger (2005); COD removal = 99 kg/d, Table 43, page 167) and during aerobic treatment (0.01 of BOD_5 in influent, ATV-DVWK (2000); BOD in influent = 19.5 kg/d, Table 43, page 167).

During water storage, phosphorus can be removed via incorporation into algal and bacterial

Table 35 Calculation of the TP removal potential for some vegetables and fruits, average yield for Namibia: PWC (2005), total duration of individual growth period: Doorenbos (1979), P content: USDA (2014)

crop	**yield**	**duration of individual growth period**	**harvests**	**P content**	**P removed**
	t/(ha×harvest)	d	1/a	% (weight)	kg/(ha×a)
wheat	6	115	3.2	0.32	61.5
pumpkin	35	95	3.8	0.04	59.2
potatoes	35	125	2.9	0.06	58.3
tomato	70	115	3.2	0.02	53.3
maize	8	120	3.0	0.21	51.1
cabbage	55	125	2.9	0.03	41.8
spinach	17	95	3.8	0.05	32.0
onion	28	120	3.0	0.03	24.7
beans	13	75	4.9	0.04	24.0
watermelon	35	95	3.8	0.01	14.8
pepper	12	135	2.7	0.02	6.5
orange	16	300	1.2	0.02	4.3
grapes	12	225	1.6	0.02	3.9

biomass and precipitation under high pH (Sperling 2007b). However, precipitation of hydroxyapatite or struvite is only relevant for pH values above 9 (Sperling 2007b). As pH is estimated at 8, removal via precipitation is neglected for planning purposes.

Following the example in Sperling (2007b), the P incorporation into algae can be estimated at 0.6 mg P/L (assumptions: dry weight of algae in the effluent is 60 mg/L, P constitutes 1% of algal mass). The TP is reduced by 20.8 kg/a during storage to 412 kg/a. After accounting for water losses due to evaporation, the average concentration in the irrigation water is 14.3 mg/L.

Crops with a high removal potential for TP, due to a high specific yield, a high number of possible harvests per year, and a high P content are wheat, pumpkin, potatoes, tomatoes, and maize (Table 35). For the crops cultivated in Outapi (Figure 67, page 139), a TP removal of 29.5 kg/a or 19.6 kg/(ha×a) can be expected from the harvested parts. Thus, the TP load is reduced to 359 kg/a in the drainage water, with a mean concentration of 95.1 mg/L.

Table 36 TP removal by crops chosen for Outapi, average yield for Namibia: PWC (2005), harvests per year and field size: Woltersdorf *et al.* (2015), P content of crops: USDA (2014)

crop	**field number**	**yield** t/(ha×harvest)	**field size** ha	**P content** % (weight)	**harvests** 1/a	**P removed** kg/a
maize	1 and 2	8	0,5	0,21	2	16,8
peppers	3	12	0,25	0,02	1	0,6
pumpkins	3	35	0,25	0,04	1	3,9
spinach	3	17	0,25	0,05	1	2,1
tomatoes	4	70	0,25	0,02	1	4,2
watermelons	4	35	0,25	0,01	1	1,0
sweet melons	4	35	0,25	0,01	1	1,0
total			**1,5**			**29,5**

With the excess TP load, an additional area of 1.2 ha could be irrigated if an optimal concentration of 17.5 mg/L for the irrigation of tomatoes is achieved in the irrigation water (Table 27, page 126). For irrigation, an additional water quantity of 56 m³/d would be required.

Removal of TP occurs during incorporation into biomass and sedimentation in the wastewater treatment plant (about 115 kg/a or 21%), incorporation into algae during storage (20.8 kg/a or 4.8%), and uptake in harvested crop parts (27.8 kg/a or 6.7%). Additional measures such as urine separation and reverse osmosis membrane filtration could retain 219 and 367 kg/a or 40% and 90%, respectively.

4.6.3.2 Monitoring data

After implementation, monitored TP loads and concentrations in the untreated water were only 114 kg/a and 10.3 mg/L (Figure 74 and Table 26, page 123). This is only one fifth of the planned load and 62% of the presumed concentration. During wastewater treatment, the TP concentration was reduced by 2.0 mg/L and the load was reduced by 22.1 kg/a. These reductions represent 47% and 16% of the reductions calculated for planning data.

During storage, the concentration increased by 1.6 mg/L to 9.9 mg/L in the irrigation water. Hence, about 12.3 kg/P were retained in the pond each year. 79.6 kg P were available per year for fertilization of the agricultural fields. Only 7.7 kg P/a were estimated to be removed via harvested crop parts. By calculation, 71.9 kg P/a or 68.5 mg/L remained in the drainage water. This amount would be sufficient for fertilization of an additional 0.23 ha.

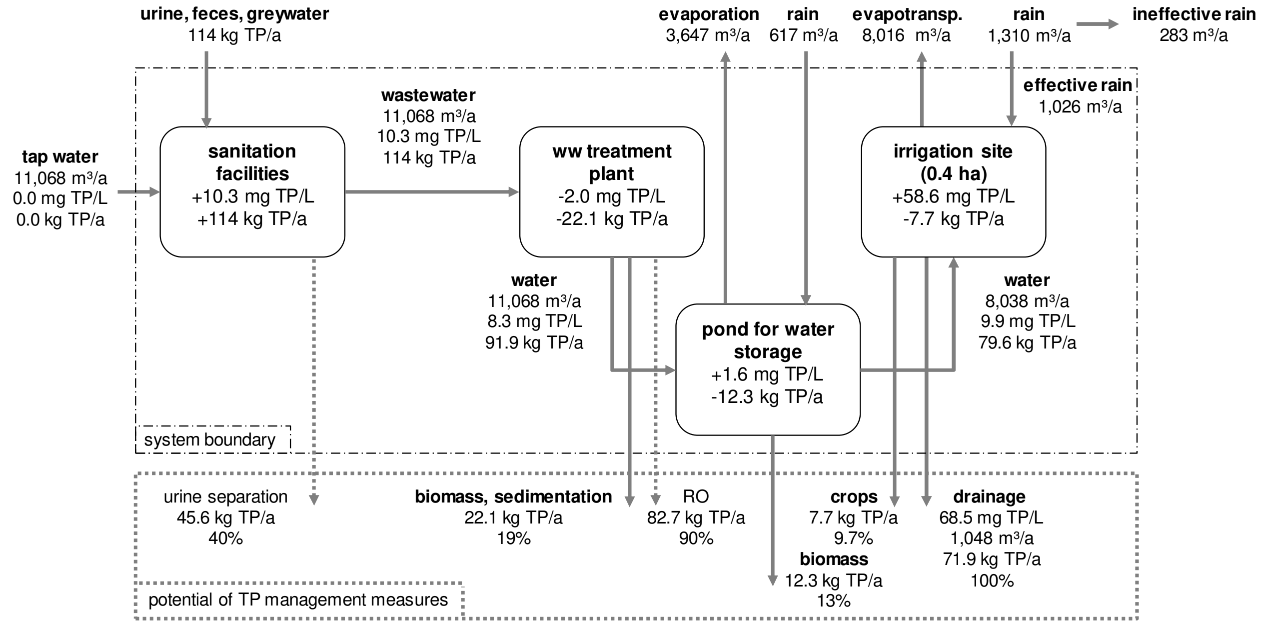

Figure 74 Water quantities and TP loads and concentrations after implementation, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for TP management, dash-dotted lines = system boundary

4.6.4 Potassium

4.6.4.1 Planning data

Typical per capita potassium loads for developing countries are not included in Sperling (2007c). The values in DWA (2008c) include K but refer to developed countries. The TCOD, TN and TP loads in Sperling (2007c) are, on average, 66% of the loads given in DWA (2008c). Thus, the potassium load used in this study was estimated at 2.8 g/(person×d), assuming a specific load of 66% of the K load in DWA (2008c) (= 0.66 × 4.2 g/(person×d)). Using the distribution in urine, feces and greywater in Figure 4 (page 13), K amounts to 1.65 g/(person×d) in urine, 0.46 g/(person×d) in feces, and 0.66 g/(person×d) in greywater.

The K load added by urine and feces is estimated at 1.15 t/a. Greywater contributes 0.36 t/a (Figure 75). The total load and concentration in the untreated water is 1.5 t/a and 46.1 mg/L, respectively.

Cell biomass contains 1% K (Tchobanoglous *et al.* 2004). Sludge production is 22.5 kg TSS/d in the UASB reactors and 15.1 kg TSS/d in the RBCs (Table 43, page 167)). During wastewater treatment, the K concentration is reduced by 4.2 mg/L and the load is reduced by 137 kg/a.

The algal and bacterial biomass production in the pond is 1,735 kg TSS/a and 345 kg TSS/a (see Section 4.6.2.1, page 146). Assuming a K content of 1% in bacteria cells (Tchobanoglous *et al.* 2004) and 0.5% in algal cells (Arceivala 1981), 12.1 kg/a of K are incorporated. After loss of some water due to evaporation, the K concentration in the irrigation water is 47.2 mg/L. The K load in the irrigation water is estimated at 1.4 t/a.

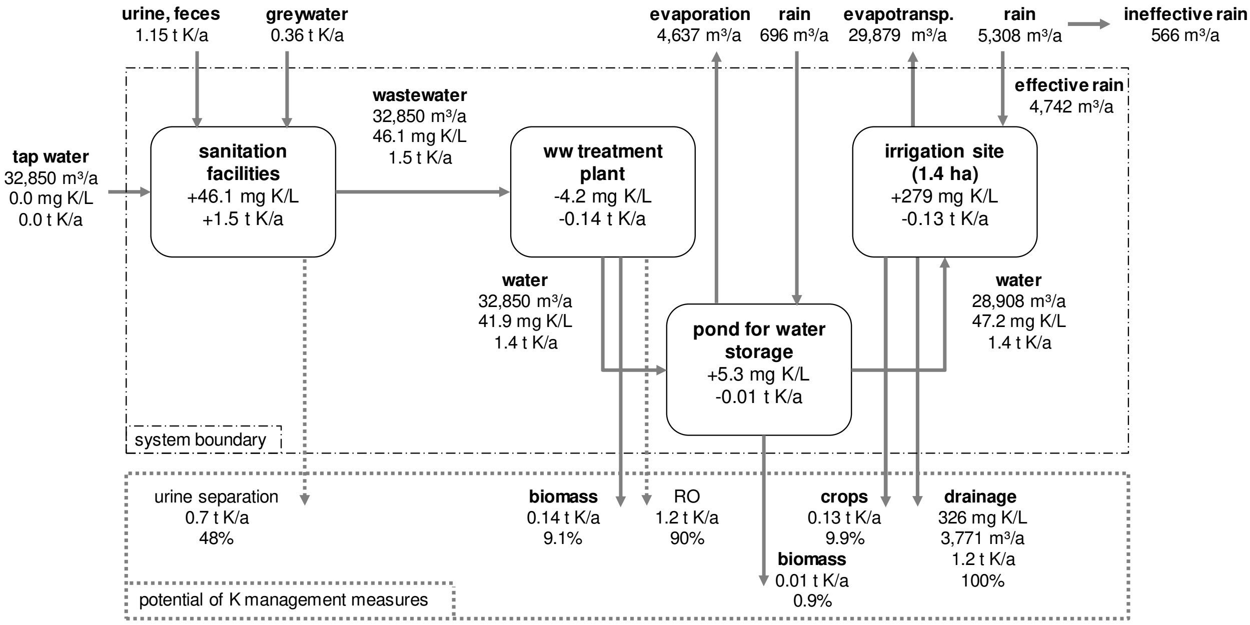

Figure 75 Water quantities, K loads and concentrations according to planning data, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for K management, dash-dotted lines = system boundary

After removal of 0.13 t/a via harvested crop parts, 1.2 t/a end up in the drainage water, which has a concentration of 326 mg/L. This load would be sufficient for fertilization of an additional area of 1.7 ha of tomatoes, considering an irrigation water quality objective of 40.1 mg/L (Table 27, page 126).

Table 37 Calculation of the K removal potential for some vegetables and fruits, average yield for Namibia: PWC (2005), total duration of individual growth period: Doorenbos (1979), K content: USDA (2014)

crop	**yield**	**duration of individual growth period**	**harvests**	**K content**	**K removed**
	t/(ha×harvest)	d	1/a	% (weight)	kg/(ha×a)
tomatoes	70	115	3.2	0.24	527
pumpkins	35	95	3.8	0.34	457
potatoes	35	125	2.9	0.43	434
spinach	17	95	3.8	0.56	364
cabbage	55	125	2.9	0.17	273
watermelons	35	95	3.8	0.11	151
beans	13	75	4.9	0.21	133
onions	28	120	3.0	0.15	124
wheat	6	115	3.2	0.39	75.0
maize	8	120	3.0	0.29	69.8
peppers	12	135	2.7	0.18	56.8
oranges	16	300	1.2	0.20	38.2
grapes	12	225	1.6	0.19	37.2

Table 38 K removal by crops chosen for Outapi, average yield for Namibia: PWC (2005), harvests per year and field size: Woltersdorf *et al.* (2015), K content of crops: USDA (2014)

crop	field number	yield	field size	K content	harvests	K removed
		t/(ha×harvest)	ha	% (weight)	-	kg/a
maize	1 and 2	8	0.5	0.29	2	23.0
peppers	3	12	0.25	0.18	1	5.3
pumpkins	3	35	0.25	0.34	1	29.8
spinach	3	17	0.25	0.56	1	23.7
tomatoes	4	70	0.25	0.24	1	41.5
watermelons	4	35	0.25	0.11	1	9.8
sweet melons	4	35	0.25	0.11	1	9.8
total			**1.5**			**143**

Reverse osmosis membrane filtration has the highest K removal potential (1.2 t/a or 90%), followed by urine separation (0.7 t/a or 48%), and removal via harvested crops (0.13 t/a or 9.9%). The estimated average K removal for the cropping pattern chosen for Outapi is presented in Table 38. Crops with a relatively high K utilization are tomatoes, pumpkins, potatoes, and spinach (Table 37). Removal during wastewater treatment is 0.14 t/a or 9.1%. Removal in the storage pond is 0.01 t/a or 0.9%.

4.6.4.2 Monitoring data

Similar to the other water constituents, the loads and concentration of K were much lower after implementation than assumed during planning (Table 26, page 123 and Figure 76, page 154). The average concentration in the untreated water was 17.3 mg/L (62% lower) and the load was about 0.19 t/a (87% lower).

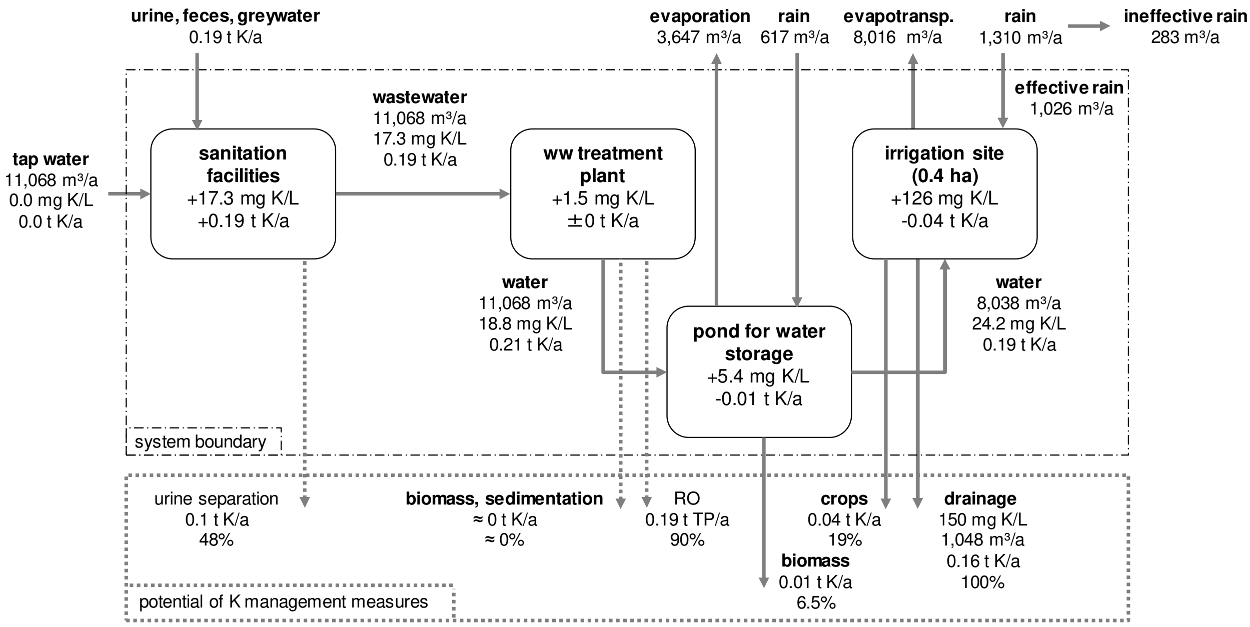

Figure 76 Water quantities and K loads and concentrations after implementation, ww = wastewater, RO = reverse osmosis membrane filtration, solid lines = implemented infrastructure, dotted lines = further possibilities for K management

The monitored K concentration in the effluent was slightly higher than in the influent of the wastewater treatment plant. This is attributed to the lower total number of values for K (14 and 19 samples for K analyses compared to, e.g., 132 and 125 samples for TCOD analyses) and the relatively high standard deviation (see Table 26, page 123). There is a calculated increase of 16.6 kg/a. Thus, the change in K mass during wastewater treatment is set at 0 t/a in Figure 76.

During water storage, the concentration increased from 18.8 mg/L in the effluent of the wastewater treatment plant to 24.2 mg/L in the irrigation water. The K load was slightly reduced from 0.21 t/a to 0.19 t/a. After removal of 37.2 kg via harvested crops, 0.16 t/a would end up in the drainage water that has a calculated concentration of 150 mg/L.

The removal potential of reverse osmosis membrane filtration is estimated at 0.19 t TP/a (90% removal). 0.1 t/a or 48% would be contained in removed urine. The K contained in harvested crops corresponds to a 19% removal rate. About 6.5% of the biomass is retained in the pond.

4.6.5 Review of the fate of TDS, TN, TP, and K loads

Figure 77 gives an overview on the proportion of TDS, TN, TP and K loads that are removed from the water in the wastewater treatment plant, in the storage pond, via harvested crop parts, and that remain in the drainage water. Most of the TDS, TP and K loads end up in the drainage water. This percentage is higher for the planning data (95%, 70% and 81%) than for the monitoring data (86%, 63% and 82%). The proportion of TN in the drainage water is lower (24% and 33%).

For TN, the proportion that is discharged in the drainage water is almost the same for planning and monitoring data. Because the monitored loads are lower, the percentage removed via crops is twice as much as in the planning data. The main difference is TN removal during wastewater treatment and in the pond. During planning, the percentage removed via ammonia stripping in the pond was relatively high for the effluent, which contains mainly ammonium nitrogen. After implementation, ammonia stripping played a minor role in the partly nitrified effluent of the wastewater treatment plant.

TP removal during wastewater treatment is almost the same as for literature data. K removal is lower during wastewater treatment. TP and K removal rates are higher during storage and via harvested crop parts, when planning and monitoring data are compared. Altogether, roughly the same percentages end up in the drainage water for planning data and monitoring data.

The additional areas that could be fertilized by TN, TP and K contained in the drainage water are between 1.2 and 2.4 ha for planning data and between 0.2 and 0.5 ha for monitoring data. Whereas heavy overfertilization was expected during planning, overall loads and concentrations were much lower after implementation. Nevertheless, large proportions of N, P and K remain unused.

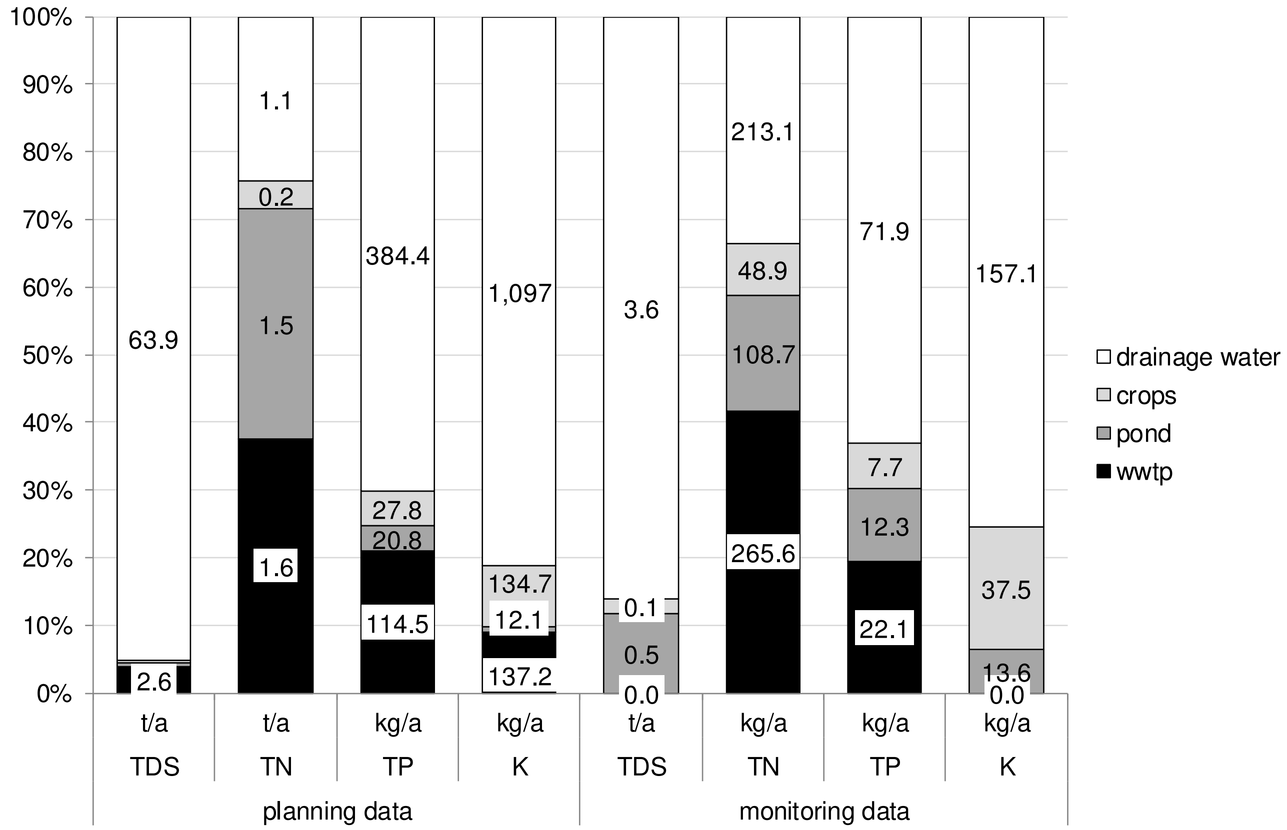

Figure 77 Overview on TDS, TN, TP and K loads and removal during wastewater treatment, in the storage pond, via harvested crops, and remaining loads in the drainage water

To make use of these nutrients, the irrigation water needs dilution or the drainage water could be further used. The EC in the drainage water was estimated at 27,123 µS/cm for planning data (Figure 66, page 138) and 5,446 for monitoring data (Figure 70, page 142). Only crops with a certain salinity resistance could be cultivated with this water. Tomatoes, maize, and peppers would yield only 50% for ECs of 5,000, 3,900 and 3,400 µS/cm, respectively. When growing salt-resistant crops such as barley, cotton, sugar beet, durum wheat or date palms, yield declines of 50% will not be expected until EC values > 10,000 µS/cm (Ayers and Westcot 1985).

4.6.6 Risk of reduced crop yields due to high N, P, and K loads

The N, P, and K requirements of common vegetable crops range from 15 to 300, 23 to 88 and 33 to 260 kg/ha, respectively, per individual growth period (Table 27, page 126). For an average of 3 harvests per year (Table 32, page 145), the N, P and K requirements are maximally 900, 264 and 780 kg/(ha×a), respectively. For planning data, these values are exceeded by the nutrient load contained in the irrigation water. After implementation, the loads were much lower, but the requirements of most crops were still exceeded (see Section 4.5.2.5, page 132).

Figure 8 (page 27) illustrates how an increase in nitrogen fertilizer initially leads to an increase in the dry matter production of *Lolium perenne* (perennial rye grass) in vessel experiments but, with further increasing nitrogen supply, to a decreasing dry matter yield. Similarly, excessive

TN, TP, and K loads and concentrations in the reclaimed water can cause declining yields and damage crops (Figure 7, page 26).

For this reason, Ayers and Westcot (1985) provide information on the effect of an increasing EC on yields. For instance, yield losses of 10% due to water salinity are expected for peppers, cucumbers, and potatoes when the EC in the irrigation water exceeds 1,500 µS/cm, 2,200 µS/cm and 1,700 µS/cm, respectively. These values are exceeded for the water quality assumed during planning. Thus, yield losses would have to be expected when irrigating with water of this quality.

After implementation, the EC was much lower: only around 600 µS/cm after storage. Thus, yield losses were not expected due high EC, considering the values in Ayers and Westcot (1985). However, for sweet corn, peppers, and potatoes, high N, P and K loads could cause yield losses, even though the EC was on an acceptable level.

Studies on the effect of N, P, and K fertilization carried out in Florida were reviewed by the Institute for Food and Agricultural Sciences at the University of Florida. The yields of peppers, watermelons, cucumbers, and potatoes decreased for TN rates above 224 kg/ha (Hochmuth and Hanlon 2014, 2013a, 2013c, 2010). Assuming the irrigation water requirements in Table 27 (page 126), these loads were not exceeded for watermelons and cucumbers after implementation of the project in Outapi. They were exceeded when irrigating peppers and potatoes. Thus, even though the EC thresholds in Ayers and Westcot (1985) would not warn against declining yields, the applied N loads might be too high for peppers and tomatoes.

The same applies to the P requirement of peppers and the K requirement of sweet corn. The relative yield of peppers can decline when P loads exceed 49 kg/(ha×growth period) (Hochmuth and Hanlon 2013a) and the relative yield of sweet corn can decline for K loads exceeding 56 kg/(ha×growth period) (Hochmuth and Hanlon 2013b). Both loads were exceeded when irrigating with reclaimed water.

Hence, the water quality objectives given in Ayers and Westcot (1985) provide criteria for recognizing harmful salinity levels in the irrigation water as long as increasing TN, TP and K loads are sufficiently reflected by the increased total salinity. This means that, even when EC is relatively low, it is recommended to additionally check the expected TN, TP and K loads.

Besides the total load applied to the fields, the concentration and ratio of chemical species are also subject to optimization. For instance, in one Israeli case study, the optimal N concentration for the cultivation of peppers was 132 mg/L and optimal yield was obtained for a NO_3-N:NH_4^+-N ratio of 4 in the irrigation solution (Bar-Tal *et al.* 2001). Other studies on the cultivation of pepper plants found that the optimal concentration was 120 mg N/L (Kirda *et al.* 2003) and 56.2 mg/L N (Yasuor *et al.* 2013).

In addition to concentrations and ratios, the kind of cultivar, cultivation system (greenhouse or open field), irrigation system, base dressing and growing conditions influence what is optimal for plants and what is needed for optimal growth and yields (Sonneveld and Voogt 2009).

Greenhouse crops are cultivated with higher nutrient concentrations in the irrigation water than field crops (Mengel 2001; Sonneveld and Voogt 2009). For instance, P concentrations are usually 0.32 to 1.7 mmol/L in soil solutions from greenhouses and much higher than concentrations of 0.01 to 0.02 mmol/L from field soils (Sonneveld and Voogt 2009).

A systematic overview on recommended loads and concentrations as a function of the local conditions and cultivation practices would be useful for fertilizer management in water reuse schemes. With this information, optimal blending of the reclaimed water with other water sources and harmonization with additionally needed fertilizers would be possible.

4.6.7 Conclusions

Salts and nutrients in irrigation water have to be controlled to allow sustainable irrigation. For water reuse, this is very important because salinity and nutrient levels are higher in reclaimed water than in conventional water sources. Hence, this chapter focused on the estimated and monitored TDS and nutrient loads and concentrations of various water flows in the sanitation system, their effect on soil and plants, and how they could be managed.

According to the planning data, considerable amounts of TDS would accumulate on the agricultural fields in the long term. About 46.5 t TDS will accumulate in the agricultural fields per hectare and year if salts are not removed. Yield losses had to be expected when irrigating salt-sensitive crops. Even in this case, with relatively low TDS concentrations in tap water and incomplete excreta collection, as well as relatively high specific water use in the communal washhouse, about 9.3 tons of TDS would accumulate on the agricultural area per hectare and year.

Thus, if rain is not sufficient for leaching, measures for salt management have to be included in sanitation systems that include water reuse in irrigation. Salt management measures prior to agricultural use, such as urine separation or reverse osmosis membrane filtration, can reduce TDS loads, but salts will still accumulate on the fields in the long term. For this case, a drainage system was installed on the agricultural fields and options for dilution of the reclaimed water with tap water were foreseen. Whenever required, the fields could be exclusively irrigated with tap water.

Salt removal by harvested crop parts is only substantial for the salt balance if the TDS content of the irrigation water is relatively low. Up to 1.1 t TDS/(ha×a) can be removed with conventional crops. However, this amount is much lower for most vegetables. Evidence from the literature suggests that TDS removal of up to 5 t/(ha×a) is possible with crops such as borage. Then, salt removal via crops could be substantial.

Precipitation of at least 500 mm, preferably in a short time period of 4 months, is required for “natural” leaching. In cases with relatively low TDS loads and sufficient precipitation in at least some years, it could be an option to abstain from installation of a drainage system and concerted leaching. For TDS management, salt removal via salt-removing crops and leaching during some wet seasons might be sufficient for salinity control. However, if these specific conditions are not met reliably, a drainage system needs to be installed for leaching the fields.

In arid and semi-arid regions, excessive water use is not desired. If feasible, options not requiring additional water input, such as leaching during the rainy season, salt removal with crops, water-saving irrigation techniques, or irrigation scheduling, should be considered for salinity control.

During planning, and similar to TDS, relatively high TN, TP and K concentrations were also estimated. The TN concentrations were predicted to exceed the maximum concentrations recommended by Ayers and Westcot (1985). A relatively large proportion of the TN was expected to be removed in the storage pond, due to ammonia stripping, because RBCs were mainly designed for oxidation of organics and not for nitrification. Nevertheless, most of the nitrogen would not be utilized by the plants and would end up in the drainage water. The TP and K concentrations were expected to remain more or less at the same level during wastewater treatment and to only increase slightly during storage, due to evaporation. After implementation, the monitored EC, TN, TP and K concentrations were relatively low. Thus, salinization, over-fertilization, and eutrophication risks were not as high as expected.

In general, for both planning and monitoring data, the majority of the TDS, TP and K loads will remain in the drainage water. For TN, the overall percentage remaining in the drainage water was lower. A considerable proportion of TN is removed from the water during wastewater treatment and storage. Ammonia stripping played only a marginal role after implementation because the capacity of the RBCs partly nitrified the water.

The uptake of salts and nutrients via crops is, in general, relatively low. The nutrient loads that are discharged with the drainage water would be sufficient for fertilization of an additional 1 to 3 hectares of agricultural fields for the planning data and between additional 0.2 and 0.5 ha for the monitoring data.

For control of salt content, the water quality criteria in the FAO guidelines can be used (Ayers and Westcot 1985). If the salinity of the water is managed, the nutrient contents can also be kept at an acceptable level. However, for sensitive crops, it is recommended to check the nutrient loads, even when overall salt levels are not of concern.

In the Outapi project, water quality criteria were met because a considerable proportion of the reclaimed water originated from shared sanitation facilities with relatively low excreta collection rates, compared to the water use. In cases where mainly individual households are connected, loads and concentrations would be higher. Then, the salinity and nutrient management measures outlined here need to be applied for sustainable provision of water for agricultural fields.

4.7 Energetic aspects

The technical layout of the Outapi water reuse scheme was developed with consideration of environmental and socio-economic characteristics, the local conditions in the settlements, as well as the city-wide sanitation approach of the OTC. In a next step, a tariff and management system for the components of the water reuse scheme had to be developed. Revenues should, in some way, flow back into the sanitation system to cross-subsidize operation and maintenance costs and ideally also capital costs. This should act as a financing mechanism, in order to assist the OTC in covering the costs.

In Outapi, revenues were originally intended to be generated in the form of money collected from the users via the introduction of tariffs, the sale of cultivated crops and credits for generated electricity. Electricity generation should be maximized via co-digestion of biomass from agriculture with sewage sludge. The produced biogas could be used for the generation of electricity and heat and thus lower expenses for external supply.

Consequently, in sanitation systems with agricultural water reuse, the elaboration of a crop cultivation scheme is closely linked to energetic aspects of the sanitation concept and influences generation of revenues and thus, in a broader sense, the tariff system and the required subsidies. The quality of the reclaimed water and the market demand determine which plants can and should be irrigated with the reclaimed water. Depending on the kind of cultivated crops, different quantities of crop residues are available for biogas production via co-digestion with sewage sludge. The methane yields depend on the quantity and characteristics of the available substrates.

This chapter focuses on energetic aspects that were of major interest during the development of this energy recovery, management, and tariff system and its implementation. It outlines the potential for electricity generation from agricultural biomass for several scenarios, provides an overview on the monitored electricity consumption of the sanitation system, and to what extent agricultural residues could contribute, by calculation, to achieve energy self-sufficiency. Finally, impediments originating from the structure of the electricity tariffs in the region are briefly discussed.

4.7.1 Co-digestion

4.7.1.1 Methane yield of agricultural residues

The methane yield of agricultural residues was obtained using the values in Table 39. The duration of the individual growth period of each crop was used to calculate the maximum number of harvests per year. Time periods needed for preparation of the fields were neglected. Typical values for crop yields in Namibia were taken from PWC (2005). Data on the total solids content of the crops were provided by nutritional data (USDA 2014).

The harvest index (HI) is the ratio of the crop yield (e.g., leaves, seeds, stalks) to the aboveground biomass of the crop (Smil 1999). This ratio was used to calculate the amount of total solids in the agricultural residues after harvest. For the main field crops, sufficient data on harvest indices were available; however, this index varies depending on the cultivars and environmental conditions (Smil 1999). Thus, for use in this study, mean harvest indices were calculated from harvest indices provided in the secondary literature (compiled in Table 40) The harvest index is relatively high for vegetables such as spinach and tomatoes. It is much lower for cereals such as maize and wheat.

Table 39 Data basis for calculation of methane yields from agricultural residues, growth period = individual growth period of the respective crop (Doorenbos 1979), FM = fresh matter of harvested plant parts (typical values for Namibia, PWC 2005), TS = total solids of harvested plant parts (USDA 2014), HI = harvest index (Döhler 2005; Fink *et al.* 1999; Salmoral and Garrido 2015; Smil 1999; Steduto *et al.* 2012), VS = volatile solids

crop	**growth period**	**harvests**	**yield**				**residues**				
			FM	**TS**	**TS**	**HI**	**TS**	**VS**	**CH_4 yield**		
	d	**1/a**	**t/(ha× harvest)**	**%**	**t/(ha×a)**	**-**	**t/(ha×a)**	**%**	**m³/kg VS**	**m³/(ha× harvest)**	**m³/(ha×a)**
wheat	115	3.2	6	87.6	16.7	0.40	25.0	89	0.276	1,944	6,169
maize	120	3.0	8	89.6	21.8	0.50	21.8	88	0.311	1,958	5,956
potatoes	125	2.9	35	20.8	21.2	0.70	9.1	83	0.360	926	2,704
beans	75	4.9	13	9.7	6.1	0.46	7.1	89	0.276	357	1,740
cabbage	125	2.9	55	7.8	12.6	0.71	5.2	85	0.323	488	1,426
tomatoes	115	3.2	70	5.5	12.2	0.72	4.9	85	0.320	419	1,329
pumpkins	95	3.8	35	8.4	11.3	0.70	4.8	85	0.320	345	1,326
watermelons	95	3.8	35	8.6	11.5	0.80	2.9	85	0.320	205	787
spinach	95	3.8	17	8.6	5.6	0.71	2.3	86	0.314	160	616
onions	120	3.0	28	10.9	9.3	0.84	1.8	85	0.320	163	496
peppers	135	2.7	12	6.2	2.0	0.70	0.9	85	0.330	89	240

Table 40 Overview of harvest indices for various crops and mean values used as input data for the calculations in Table 39

crop	**mean**	**Steduto *et al.* (2012)**	**Fink *et al.* (1999)**	**Döhler (2005)**	**Schmidt and Klöble (2009)**	**Salmoral and Garrido (2015)**	**Lal (2005)**
spinach	0.71		0.75			0.67	
tomato	0.72	0.5 to 0.65				0.78	
pumpkin	0.70					0.70	
cabbage	0.71		0.67			0.75	
potatoes	0.78				0.83	0.70	0.80
beans	0.46		0.43	0.48	0.50	0.45	
watermelon	0.80					0.80	
onion	0.84		0.92			0.75	
wheat	0.48	0.45 to 0.55		0.53	0.48		0.40
maize	0.51	0.50		0.43	0.56	0.57	0.50
pepper	0.70					0.70	

The highest amount of methane can be generated from the residues of maize and wheat (5,956 m³/(ha×a) and 6,169 m³/(ha×a), Table 39). Vegetable residues can be used to produce between 240 m³ CH_4/(ha×a) (peppers) and 2,704 m³ CH_4/(ha×a) (potatoes).

The volatile solids content and the specific methane yield of the residues used for the calculations in Table 39 were also extracted from the literature (see Table 41). Values for residues of tomatoes, pumpkins, watermelons and onions were not available. Thus, the average volatile solids (VS) content and methane yields obtained for spinach, cabbage, potatoes, beans and peppers were used instead (VS = 85.5%, CH_4 yield = 0.320 m³/kg VS).

The specific methane yields reported in the literature and calculated mean and median values are shown in Figure 78. The grey horizontal bars represent the medians and the black horizontal bars represent the means of the collected values. For instance, data on VS content and CH_4 yield of cabbage (leaves and stems) is available in Zubr (1986), Gunaseelan (2004), Cho and Park (1995) and KTBL (2007). The average values are 85.4% for the VS content and the methane yield is 0.323 m³/kg VS. Figure 78 includes the data in Table 41 and further values

Table 41 Literature values for volatile solids (VS) content and specific methane yields of crop residues; the mean values of the literature data were used for the calculations in this study, - = no data

crop residue	**values used for calculation**		**literature values**		
	mean VS content %	**mean CH_4 yield m³/kg VS**	**VS %**	**CH_4 yield m³/kg VS**	**source**
spinach waste	85.5	0.314	85.5	0.314	Knol *et al.* (1978)
cabbage leaves and stems	85.4	0.323	78.0	0.382	Zubr (1986)
			-	0.343	Zubr (1986)
			91.2	0.309	Gunaseelan (2004)
			91.8	0.291	Gunaseelan (2004)
			84.0	0.277	Cho and Park (1995)
			82.0	0.336	KTBL (2007)
potatoes leaves and stems	82.6	0.360	79.0	0.495	Deublein and Steinhauser (2011)
			90.0	0.110	Vlyssides *et al.* (2015)
			-	0.606	Reinhold and Noack (1956)
			78.9	0.229	Keymer (2016)
beans leaves and stems	88.9	0.276	-	0.265	Petersson *et al.* (2007)
			85.4	0.387	Pakarinen *et al.* (2011)
			90.0	0.174	Lopez-Davila *et al.* (2012)
			91.2	0.277	Keymer (2016)
peppers leaves and stems	85.0	0.330	85.0	0.330	Rhee *et al.* (2012)
wheat straw	89.3	0.276	-	0.313	Hashimoto (1989)
			-	0.189	Amon *et al.* (2007)
			82.8	0.154	Döhler (2005)
			89.9	0.362	Sharma *et al.* (1988)
			91.3	0.302	Tong *et al.* (1990)
			90.6	0.333	Tong *et al.* (1990)
			92.0	0.189	KTBL (2007)
			-	0.367	Reinhold and Noack (1956)
maize straw	87.9	0.311	72.0	0.351	Deublein and Steinhauser (2011)
			93.2	0.360	Tong *et al.* (1990)
			89.0	0.214	Menardo and Balsari (2012)
			97.5	0.317	Dinuccio *et al.* (2010)

for other crops (whole plants and straw). Leaves from trees and shrubs have the lowest specific methane yield. The highest specific methane yields are achievable via digestion of potato pulp and whole maize plants. For some crops, such as spinach and peppers, only few values for methane yields of residues are available. For other crops, no values could be found at all (e.g., tomato residues, pumpkin residues) or the reported values vary considerably (e.g., potato residues). This has to be kept in mind for the mean values in Table 41 that were chosen for subsequent calculations.

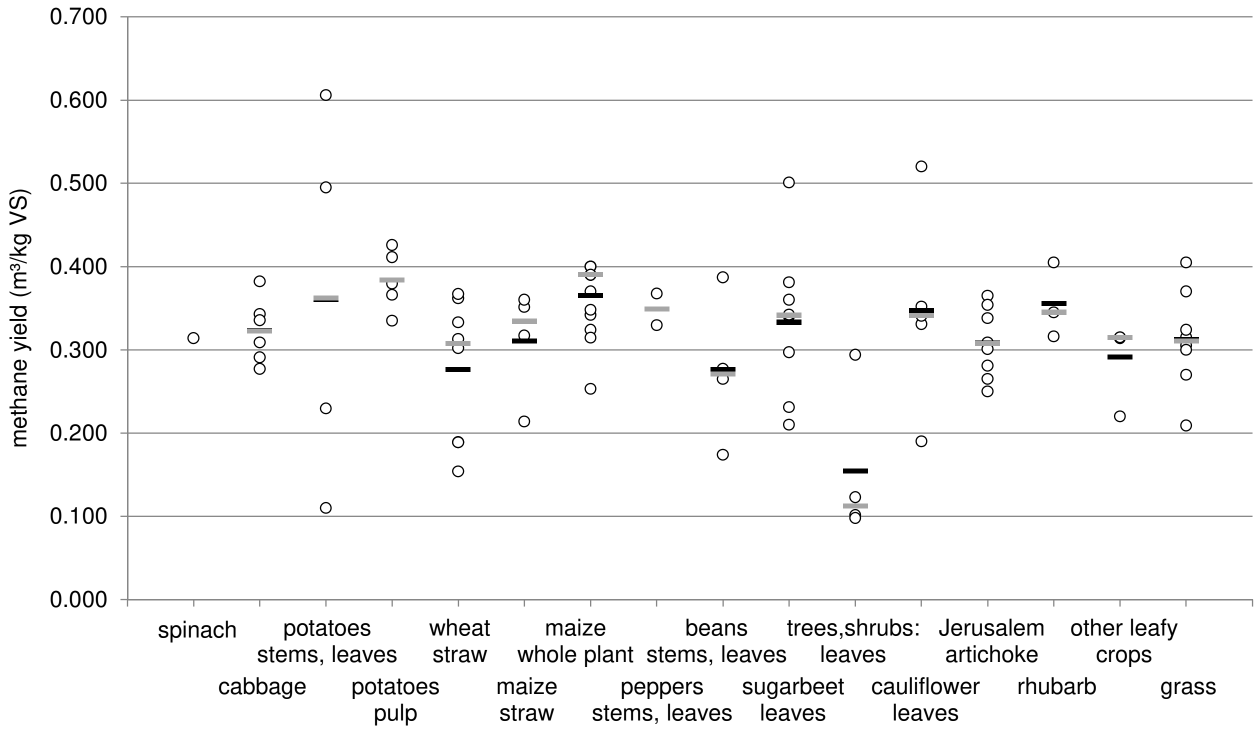

Figure 78 Methane yields of a variety of plant types, as reported in the literature; grey horizontal lines = medians, black horizontal lines = means, a detailed list of all values and references in contained in the appendix (Table 49, Table 50 and Table 51, page 233*ff.*)

4.7.1.2 Potential electricity production

During planning of the sanitation system, several crop patterns were considered. The objective was to find a scenario that maximizes revenues from crops sold at local markets, maximizes methane yields obtainable from co-digestion of crop residues with sewage sludge, and minimizes health concerns for consumers. The habits and preferences of the local population should also be considered.

The first scenario only includes cultivation of vegetables with a high value on the local market. An overview of the crop pattern is given in Table 42 (scenario “market”). The agricultural area consists of 4 fields. Three crops are grown consecutively on each field (Table 42).

The second scenario is identical to the cropping scheme suggested by Woltersdorf *et al.* (2015) (see also Figure 67, page 139). This scenario is a compromise between the objective of obtaining maximal revenues on the local market and maximal biomass production for co-digestion.

Maize is cultivated on half of the area and the remaining area is cultivated with high value vegetable crops (scenario “market/energy”).

The scenarios 3 and 4 assume cultivation of maize on the entire area. In one variant, only the maize straw is fed to the anaerobic digester (scenario “energy 1”). In a second variant, the whole maize plant, including the cobs, is fed to the anaerobic digester (scenario “energy 2”).

For the fifth scenario, health aspects were considered for crop choice. This means the chosen crops are high above ground or need to be peeled or cooked prior to consumption (scenario “health”). It is identical to the scenario “market/energy”, except that spinach and peppers were deleted.

Table 42 Overview of the crop pattern of the scenarios, energy 1: maize cobs are sold, straw is digested, energy 2: the whole maize plant is fed to the digester

scenario	number of harvests	field 1	field 2	field 3	field 4
market	harvest 1	pepper	tomato	pepper	tomato
	harvest 2	pumpkin	sweet melon	pumpkin	sweet melon
	harvest 3	spinach	watermelon	spinach	watermelon
market/energy	harvest 1	maize	maize	pepper	tomato
	harvest 2	maize	maize	pumpkin	sweet melon
	harvest 3	-	-	spinach	watermelon
energy 1	harvest 1	maize	maize	maize	maize
	harvest 2	maize	maize	maize	maize
energy 2	harvest 1	maize	maize	maize	maize
	harvest 2	maize	maize	maize	maize
health	harvest 1	maize	maize	tomato	tomato
	harvest 2	maize	maize	pumpkin	sweet melon
	harvest 3	-	-	watermelon	watermelon

For each scenario and its variants, the calculations were made for a total area of 1 ha, 2 ha and 3 ha. The field size is 0.25 ha 0.5 ha and 0.75 ha, respectively. The potentially producible thermal and electrical energy from the wastewater and sewage sludge is the same in all scenarios (Figure 79, Figure 80 and Figure 81). It is 178 kWh/d electrically and 307 kWh/d thermally. The assumptions for calculating the potential for utilization of the thermal and electric energy from the sewage sludge and organics contained in the untreated water are compiled in Table 43 (page 167).

The thermal energy recovered from solar panels is also identical in all scenarios (120 kWh/d). The required electric energy for operation of the wastewater treatment plant was estimated at 250 kWh/d. The required thermal energy for maintaining thermophilic digestion was estimated at 440 kWh/d. These estimates were provided by the project’s industry partner Bilfinger Water Technologies (dashed horizontal lines in Figure 79, Figure 80 and Figure 81). The monitored electricity consumption is presented in Section 4.7.2 (page 171*ff.*). Because the monitored data were obtained under partial hydraulic loading, the estimated electricity demand for full hydraulic loading was used for the calculations in this section.

For gas motors, the electricity potential is between 33% and 40% electrically and around 50% thermally (DWA 2010). For this case, a conversion efficiency η_{el} of 33% was chosen. 10% of the energy potential are lost due to the generator, radiation and heat exchanger losses (ASUE 1999).

For an agricultural area of 1 ha, scenario “energy 2” achieves electrical energy self-sufficiency (Figure 79). The electrical energy demand can be covered to an extent between 74% and 106% of the requirement. The thermal energy requirement of the wastewater treatment plant is covered by 100% to 131%. In all scenarios, agricultural residues contribute to energy generation only to a minor degree. Depending on the scenario, this percentage is between 3% and 33% of the electrical and between 3% and 26% of the thermal energy requirement. CH_4 produced from sewage sludge and in the UASB reactors contributes 67% to 97% of the electrical energy.

When cultivating crops on a total area of 2 ha, the electricity requirement of the sanitation system can be met by the scenarios “energy 1” and “energy 2” (Figure 80). There is even a surplus of 40% for “energy 2”. The other scenarios could cover between 76% and 89% of the electricity demand. The thermal energy demand can be met all scenarios.

The ultimate size of the agricultural area is 3 ha (Zimmermann *et al.* 2017b). The scenarios that can meet the electricity demand of the sanitation system are “energy 1” and “energy 2” (114% and 175% demand coverage). The other scenarios potentially cover the electricity demand by 79% (“market”), 96% (“market/energy”) and 97% (“health”). The thermal energy requirement is met or exceeded by all scenarios (Figure 81).

The assumed electricity demand of the vacuum sewers and wastewater treatment plant is

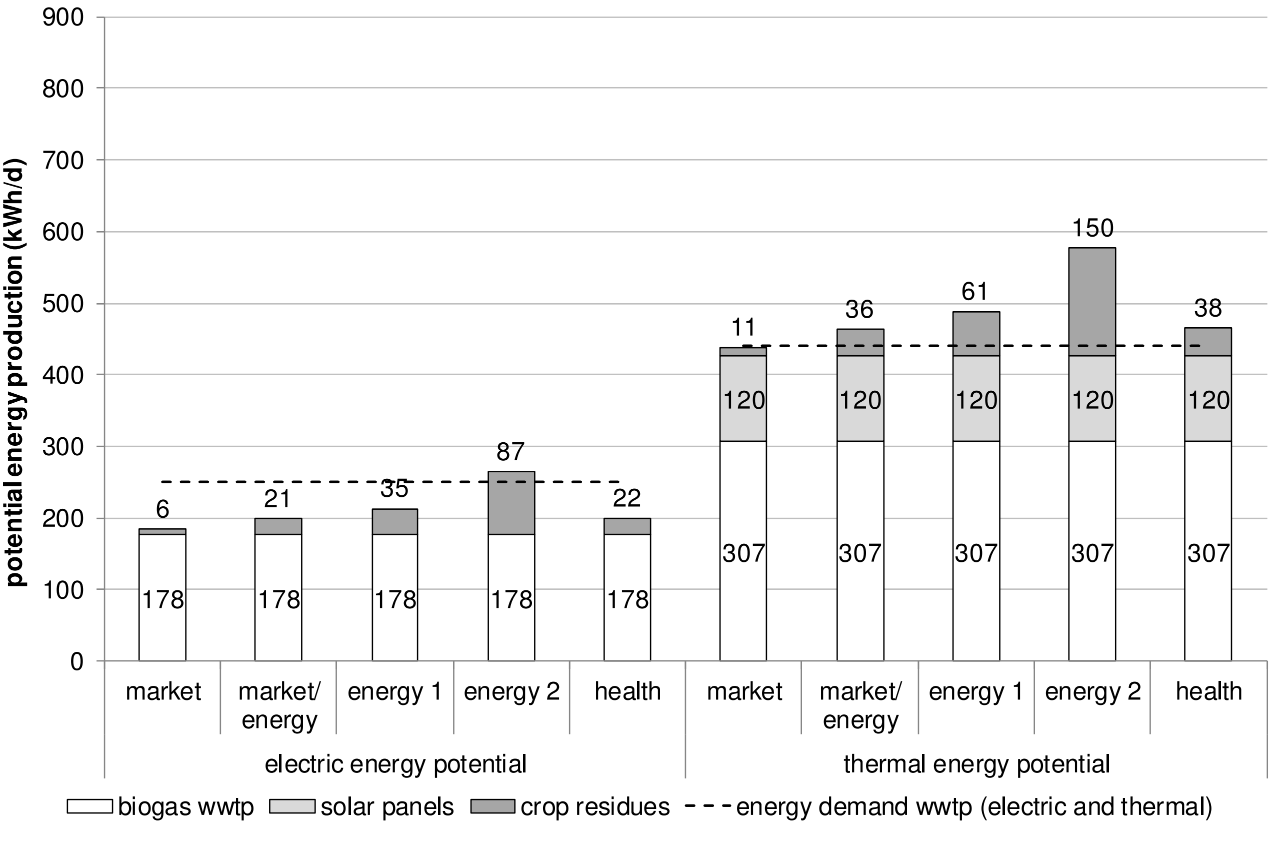

Figure 79 Producible electrical and thermal energy on 1 ha for the examined scenarios, wwtp = wastewater treatment plant

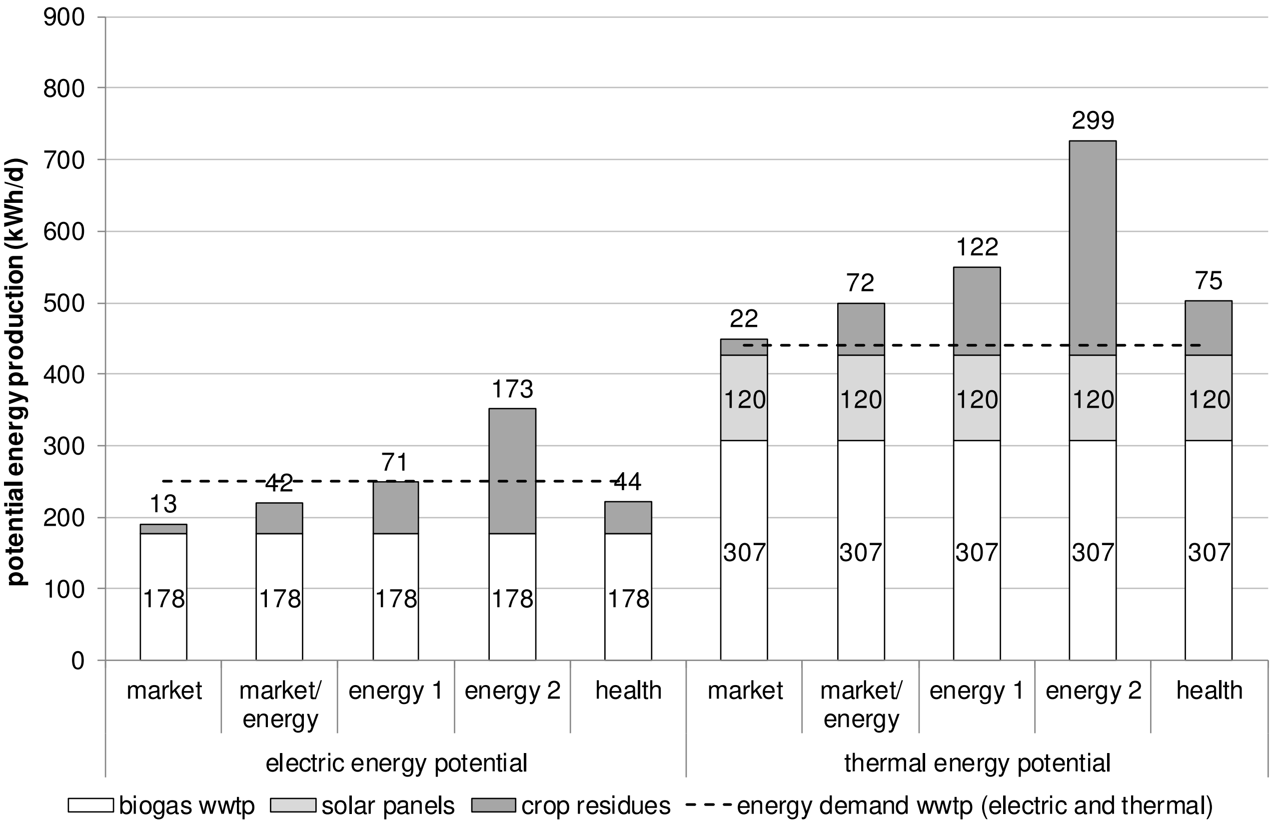

Figure 80 Producible electrical and thermal energy on 2 ha for the examined scenarios, wwtp = wastewater treatment plant

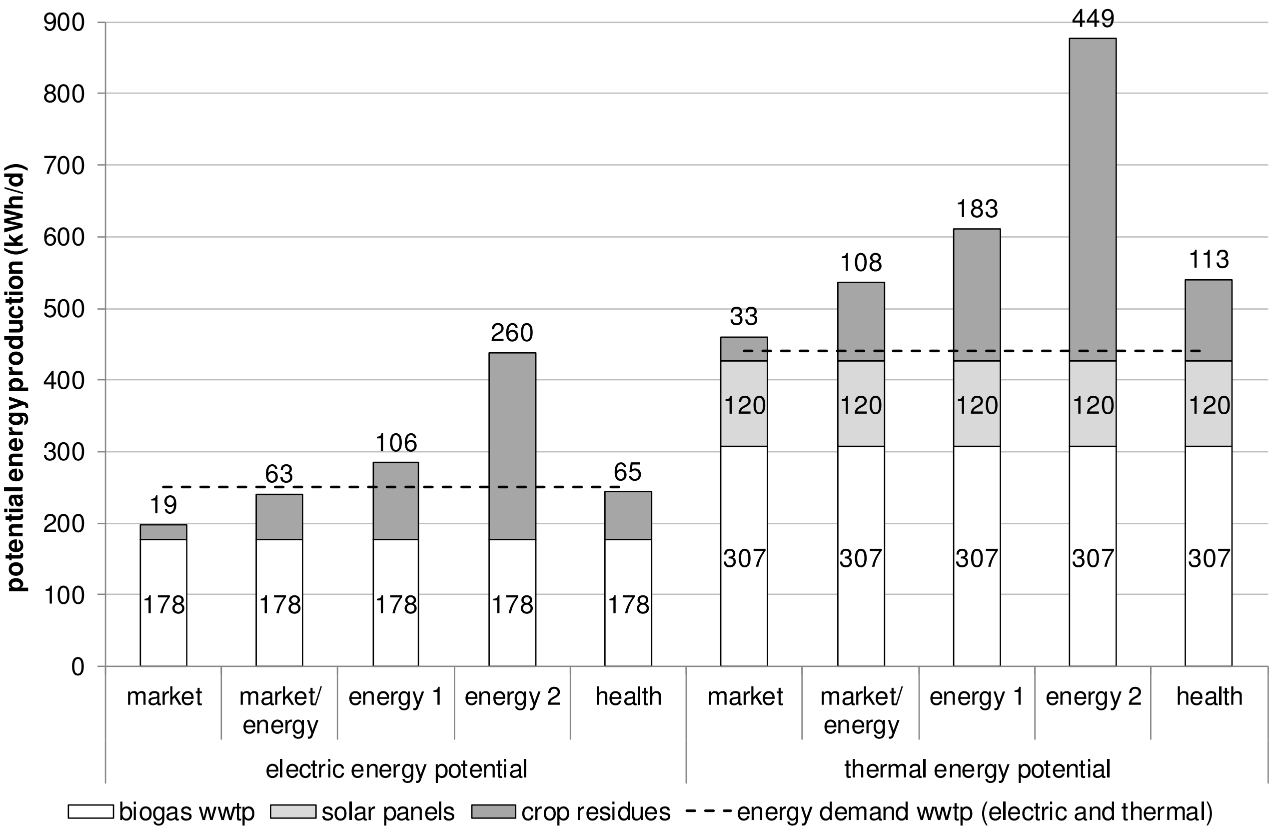

Figure 81 Producible electrical and thermal energy on 3 ha for the examined scenarios, wwtp = wastewater treatment plant

Table 43 Assumptions for calculation of methane yields from UASB reactors and digested excess sludge, η_{el} = electrical efficiency, η_{therm} = thermal efficiency

	value	unit	source
TCOD loads UASB reactors			
influent	150	kg TCOD/d	planning data
reduction due to anaerobic digestion	0.66	-	Chernicharo (2007)
reduction due to anaerobic digestion	99.0	kg TCOD/d	calculated
TSS loads UASB reactors			
influent	90.0	kg TSS/d	planning data
reduction	0.64	-	ATV-DVWK (2000)
reduction	57.6	kg TSS/d	calculated
VS/TS ratio	0.40	-	Tchobanoglous *et al.* (2004)
primary sludge production assumption: TSS ≈ TS	23.0	kg VSS/d	calculated
CH_4 yield UASB reactors			
CH_4 yield per COD converted	0.35	L CH_4/g COD	Tchobanoglous *et al.* (2004)
CH_4 yield	34.7	m³ CH_4/d	calculated
excess sludge UASB reactors			
specific excess sludge production	0.15	kg TSS/ kg COD applied	Chernicharo (2007)
TSS/VSS ratio of the bacterial cell	0.8	-	Sperling (2007a)
excess sludge production	22.5	kg TSS/d	calculated
excess sludge production	18.0	kg VSS/d	calculated
TSS loads RBCs and microscreen			
influent	32.4	kg TSS/d	planning data
effluent	0.5	kg TSS/d	planning data
reduction	31.9	kg TSS/d	calculated
VS/TS ratio	0.4	-	Tchobanoglous *et al.* (2004)
sludge production assumption: TSS ≈ TS	12.8	kg VSS/d	calculated
BOD UASB reactors and RBCs			
BOD untreated wastewater	75.0	kg/d	planning data
reduction in UASB reactors	0.74	-	Chernicharo (2007)
BOD influent RBC	19.5	kg/d	planning data
BOD effluent	2.7	kg/d	planning data
excess sludge RBC and MS			
specific excess sludge production	0.9	kg TSS/kg BOD removed	Sperling (2007a)
TSS/VSS ratio of the bacterial cell	0.8	-	Sperling (2007a)
excess sludge	15.1	kg TSS/d	calculated
	12.1	kg VSS/d	calculated
methane yields from sludge digestion			
biogas production	0.450	m³ biogas/ kg VS	Gujer (2007a)
CH_4 content	0.65	-	Haberkern *et al.* (2008)
sludge UASB (from TSS)	6.7	m³ CH_4/d	calculated
excess sludge UASB	5.3	m³ CH_4/d	calculated
sludge RBC and MS (from TSS)	3.7	m³ CH_4/d	calculated
excess sludge RBC	3.5	m³ CH_4/d	calculated
electrical and thermal energy			
total methane yield	53.9	m³/d	calculated
energy content methane	10	kWh/m³	Bischofsberger (2005)
η_{el}	0.33	-	ASUE (1999)
η_{therm}	0.57	-	ASUE (1999)
potential for electrical energy	178	kWh/d	calculated
potential for thermal energy	307	kWh/d	calculated

Table 44 Required additional areas and irrigation water quantities to achieve calculated electricity self-sufficiency, estimated irrigation requirement: based on Doorenbos (1979) and Woltersdorf *et al.* (2015), assumed electricity demand: estimate provided by Bilfinger Water Technologies, producible electricity from wastewater (UASB reactors and anaerobic digester): Table 43, producible electricity from residues: Figure 79

		scenario				
parameter	**unit**	**market**	**market/energy**	**energy 1**	**energy 2**	**health**
assumed electricity demand	kWh/d	250	250	250	250	250
available water quantity	m³/d	90	90	90	90	90
producible electricity from ww	kWh/d	178	178	178	178	178
	kWh/m³	2.0	2.0	2.0	2.0	2.0
estimated irrigation requirement of cropping pattern	m³/(ha×a)	17,760	17,760	13,000	13,000	17,760
	m³/(ha×d)	49	49	36	36	49
producible electricity from residues	kWh/(ha×d)	6	21	35	87	22
	kWh/m³	0.13	0.43	0.99	2.43	0.45
irrigation water requirement	m³/d	119	104	84	57	103
required agricultural area for energy self-sufficiency	ha	2.4	2.1	2.4	1.6	2.1
leaching requirement	m³/d	18	16	13	9	15
irrigation + leaching requirement	m³/d	136	119	97	65	119

250 kWh/d. The producible electricity from the wastewater is 178 kWh/d (Table 43). Referred to the water quantity of 90 m³/d, this is 2.0 kWh/m³ (Table 44).

When crops are irrigated with the reclaimed water, the producible electric energy from the crop residues varies between 6 kWh/(ha×d) ("market") and up to 87 kWh/(ha×d) ("energy 2") (Figure 79). Referred to the irrigation demand per ha, between 0.13 and 2.43 kWh/m³ can be produced from irrigated crops.

For each scenario, there is a minimum water quantity required for production of a sufficient amount of electricity from the wastewater constituents and the irrigated crops and crop residues. In the scenario "market", this water quantity is 119 m³/d (≈ 250 kWh/d ÷ (2.0 kWh/m³ + 0.13 kWh/m³)). 104 m³/d are required for the scenario "market/energy", 84 m³/d for "energy 1, 57 m³/d for "energy 2" and 103 m³/d for "health" (Table 44, Figure 82).

The areas required to cover 100% of the assumed electricity demand are 2.4 ha (scenario "market"), 2.1 ha (scenario "market/energy"), 2.4 ha (scenario "energy 1"), 1.6 ha (scenario "energy 2") and 2.1 ha (scenario "health"). The irrigation demand of each scenario was roughly estimated using the data in Doorenbos (1979) (see Table 27, page 126) and Woltersdorf *et al.* (2015) (Table 28, page 139). Assuming a leaching fraction of 15% (Table 28, page 139), between 65 m³/d and 136 m³/d of water need to be treated to provide the required water quantity. The monitored mean water quantity (30.3 m³/d) would not be sufficient to reach energy self-sufficiency in any of the scenarios. The sanitation concept was designed for a water quantity of 90 m³/d. The required irrigation water quantity could be provided for the scenario "energy 2".

The previous calculations are theoretical because the combined heat power unit only went operational for a test phase from February 2015 to July 2015 (Figure 83, page 172). Maize straw and sewage sludge were fed to the anaerobic digester. This study was carried out by Bilfinger

Water Technologies. During this test phase, between, on average, 3.8 kWh/d to 9.9 kWh/d were generated per month from February to May 2015. In June and July 2015, this figure was 0.4 kWh/d and 0.8 kWh/d. From February to May 2015, about 3% of the electricity demand could be covered (average demand between February to May 2015: 242 kWh/d). In June and July 2015, the produced electricity corresponded to only 0.2% of the demand of 202 kWh/d (June) 0.4% of the demand of and 203 kWh/d (July).

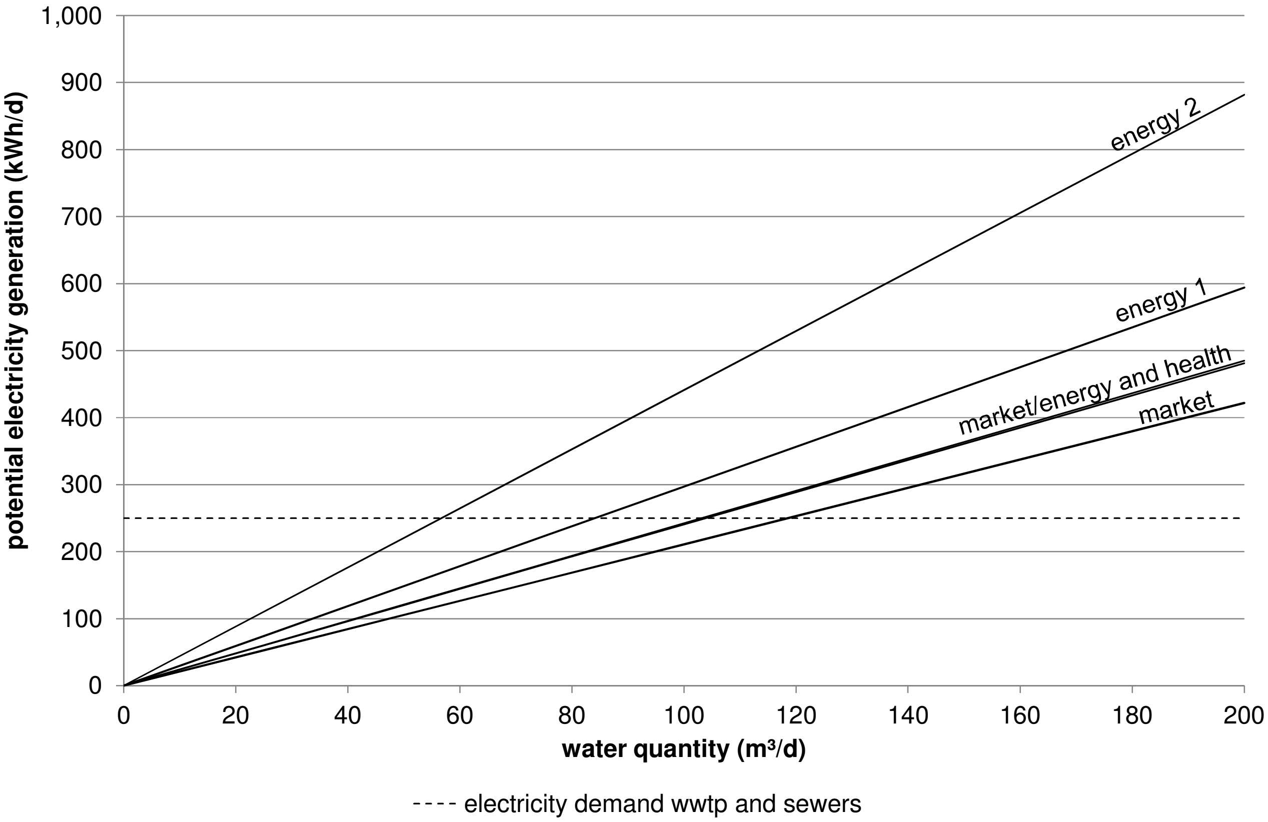

Figure 82 Potential electricity generation for the considered scenarios, dashed line = requirement for wastewater treatment plant and vacuum sewers, wwtp = wastewater treatment plant

Thus, the generated electricity did not substantially contribute to covering the electricity demand of the sanitation system. This was mainly due to the lower amounts of sewage sludge and agricultural residues available for co-digestion. Surplus electricity was fed to the local electricity grid but was not credited, because the tariff structure of the local supply entity did not include a tariff for customers consuming and generating electricity at the same point of supply. More details on this aspect are provided in Section 4.7.3 (page 183*ff.*).

4.7.1.3 Barriers to the implementation of co-digestion and co-generation

In addition to the unfavorable tariff structure, a number of other obstacles arose during the attempt to implement the thermophilic co-digestion of sewage sludge and crop residues. TS and TCOD contents of the untreated wastewater were much lower than planned (see Chapter 4.2.10, page 86*ff.*). Thus, sewage sludge production was lower. Overall wastewater quantities were also lower than expected (on average 30.3 (±11.8) m^3/d instead of 90.0 m^3/d). As a consequence, irrigable areas were smaller, and crop production and the amount of crop residues were lower.

The cooperation between operators of the wastewater treatment plant and the irrigation site was difficult to maintain. For instance, responsibilities for transfer of the crop residues from the irrigation site to the digester, or for chopping the residues and feeding procedure remained unclear or were not fulfilled.

The monetary value of the vegetables sold at local markets was much higher than the monetary value of producible electricity from these crops. For instance, the market price for maize, pumpkins, watermelons, and sweet melons ranged from 3 to 5 NAD/kg (Zimmermann *et al.* 2017b), which is about 0.26 to 0.43 EUR/kg (average currency exchange rate of 11.53 NAD/EUR during the project period from 2010 to 2015, www.oanda.com).

1 kg of maize cobs can be used to produce 291 L of methane (= 1 kg × 79.7% TS × 94.8% VS × 0.385 m^3 CH_4/kg VS, assumptions for TS, VS and specific methane yield: KTBL (2015)). The charge for energy from the local electricity supplier NORED (Northern Regional Electricity Distribution Company) ranges from 0.05 EUR/kWh up to 0.16 EUR/kWh (depending on the time of the year and the time of the day, see Table 46, page 185). With an energy potential of 10 kWh per m^3 methane and a conversion efficiency of 33%, the value of the producible electricity is between 0.05 EUR/kg and 0.15 EUR/kg (0.15 EUR/kg = 0.291 m^3 CH_4/kg × 10 kWh/m^3 CH_4 × 33% × 0.16 EUR/kWh). This is much less than the obtainable market price of 0.26 to 0.43 NAD/kg when selling the maize cobs. Because no credit is received when feeding electricity to the grid, these savings can only be achieved if the electricity generation and electricity demand of the wastewater treatment and vacuum sewers are synchronized.

Tomatoes, spinach, and peppers can be sold at higher prices of 8 to 12 NAD/kg (Zimmermann *et al.* 2017b) or 0.69 to 1.04 EUR/kg (average currency exchange rate of 11.53 NAD/EUR during the project period from 2010 to 2015, www.oanda.com). Because TS and VS contents and specific methane yields of these crops are lower (Table 41, Figure 78), the producible electricity would also be less. Thus, the difference between the potential revenues when selling the crops at local markets, compared to production of electricity, is even higher.

In addition, food and fodder is urgently needed in the region (FAO 2014b; Rukandema *et al.* 2009). Considering this, co-digestion of whole maize plants, including cobs, but also of the residual biomass is questionable. This was actually the decisive point in the project for not utilizing maize cobs for co-digestion.

Instead, the capacity of the anaerobic digestion unit could be used as a disposal option, e.g., for the contents of fat separators or similar wastes, provided that the produced electricity could be used or compensated for by the electricity supplier. An extensive discussion of the tariff structure and its significance for on-site electricity generation is provided in Section 4.7.3 (page 183*ff.*)

All in all, these challenges could not be overcome during the project period. The energetic concept could not be implemented as intended. Finally, only the relatively low quantity of sewage sludge was processed, as described above, together with the available residual maize biomass during the test phase of the co-generation unit. The excess heat produced by the electricity generator was not sufficient to operate a thermophilic digestion (50-57°C) but only a mesophilic digestion step (35-40°C). The relatively small quantities of sewage sludge were dried by the sun on the sludge beds.

4.7.2 Electricity consumption

4.7.2.1 Total electricity consumption

In this project, the electricity for operation of the vacuum sewers and wastewater treatment plant could be covered by three options: through the network of the public electricity provider NORED, the emergency generator, and a combined heat power unit.

The mean total power consumption was 212 kWh/d during the entire monitoring period from August 2013 to July 2015 (Figure 83). Until mid-September 2013, power was supplied via the emergency generator. After the switch to commercial power, on average, only 1.5 kWh/d were provided by the emergency generator and 210 kWh/d by the local power grid. The energy that was provided by the emergency generator constituted only a small fraction of the total demand. However, the emergency generator was required to meet the electricity demand during blackouts. From October 2013 to July 2015, there were only 5 months in which the emergency generator was not required.

The actually consumed electricity was 16% lower than the demand anticipated during planning (250 kWh/d). However, the volume of treated water was 66% lower than the planned value and the loads contained in the water were 80% to 92% lower (see Chapter 4.2.10, page 86*ff.*).

The power consumption of the individual components of the wastewater treatment plant and the vacuum sewers are shown in Figure 84. The vacuum pumps consumed the largest proportion of the supplied electricity (mean 31.0%), followed by the RBCs (17.0%) and sewage pumps (including chopper, 16.1%). “Miscellaneous” includes the energy consumption for the microscreen, service water supply, UV disinfection, and the pumps of the sludge liquor tank (recirculation). This value was, on average, 11.1%. The power consumption of the component “biogas” arose, for instance, from the inflation of the gas storage (3.5%). Under “sludge”, the main consumers of the sludge treatment and recirculation (e.g., the horizontal sludge agitator in the anaerobic digester, sludge circulation) are summarized. Their share of the electricity consumption was 6.9%. The power assignments can be viewed in the appendix (Table 48, page 232).

Energy was also required for pumping the heating water (between the water tank and solar panels, between the water tank and the heating installations for the untreated water and the anaerobic digester). This proportion was, however, very low with an average value of 0.3%.

The difference between the sum of the power consumption of each component and the total power consumption results from unrecognized smaller power consumers (e.g., laboratory equipment, lighting, level measurements, Figure 84).

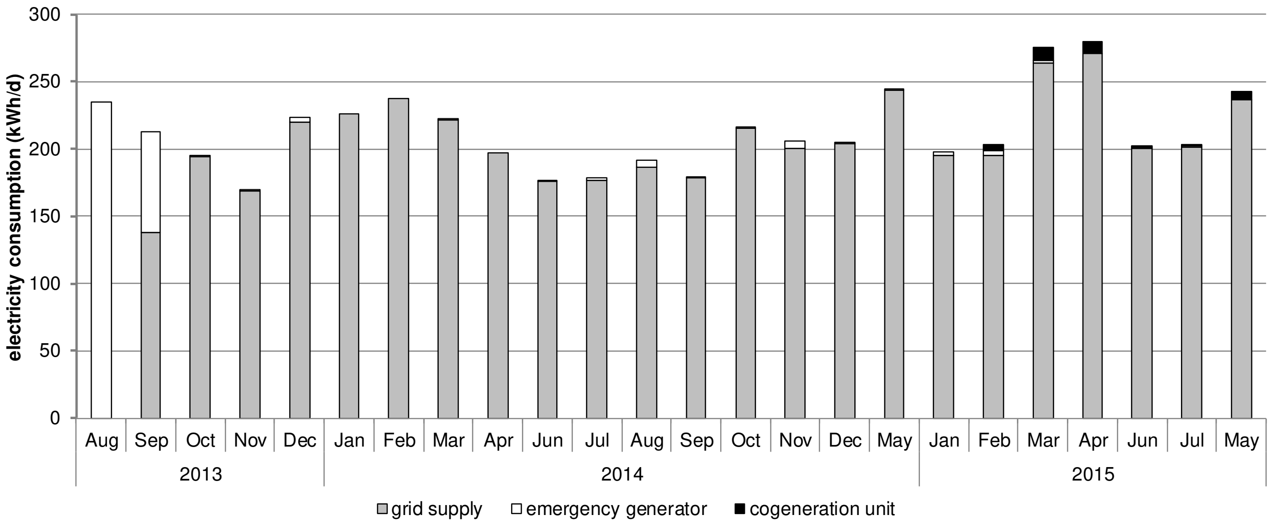

Figure 83 Average power consumption per day and origin of the electricity

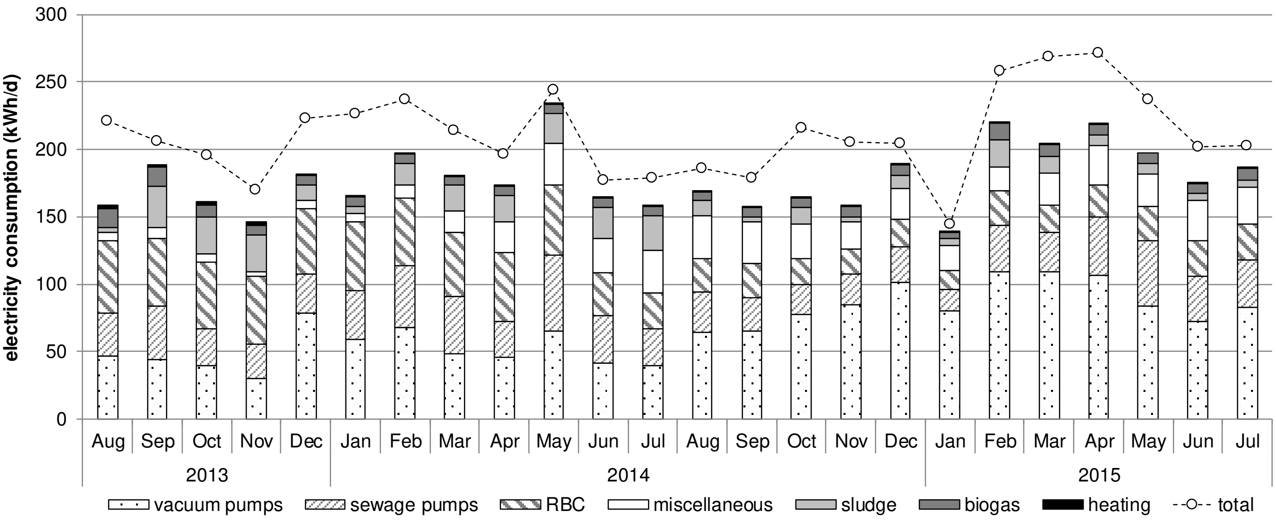

Figure 84 Total power consumption and power consumption of single components

4.7.2.2 RBCs, vacuum sewers and wastewater pumps

The aerobic treatment step of the wastewater treatment plant consisted of two RBCs operated in parallel. Because the water quantities were lower than expected, one of the RBCs was decommissioned on 8 June 2014.

As a second step, the remaining RBC was operated intermittently during the night (8:00 pm to 6:00 am) from October 2014 up to and including April 2015. Very small quantities of wastewater had to be transported at night. They could be stored in the vacuum tank. For this purpose, the critical water level was increased to 1.15 m inside the tank. This water level controlled the sewage pumps. After the sewage pumps had pumped the water for the last time to the treatment plant at night, the RBCs remained in continuous operation for one more hour. Subsequently, interval operation started (20 seconds of rotation, followed by a 10-minute break). If one of the sewage pumps or recirculation pumps resumed operation, the RBCs were put into continuous operation again.

From August 2013 to May 2014, the mean power consumption of the RBCs was 50.8 kWh/d (Figure 85). From June to September 2014, the energy requirement of the RBC was reduced to 27.4 kWh/d by the shutdown of one line. During the time period with intermittent operation at night (October 2014 to April 2015), electricity consumption decreased further to 20.2 kWh/d (Figure 86).

While changing the operational mode of the RBCs at night, the operation of the UV disinfection was also modified. During the intermittent operation of the RBCs, the UV disinfection was turned off. The energy requirements of the UV disinfection, microscreen, and recirculation ("miscellaneous") were 30.0 kWh/d from June 2014 to September 2014 and 22.5 kWh/d from October 2014 to April 2015 (Figure 87, Figure 88). The strong increase in the power consumption of this component in March/April 2014 is because UV disinfection was initially not included in its power consumption. Overall, during nightly operation, an energy saving of about 7 kWh/d was achieved for the RBCs and savings of about 6 kWh/d by turning off the UV disinfection.

The power consumption of the vacuum pumps remained at the same level until September 2014 and was proportional to the amount of treated water (Figure 89, Figure 90). As of October 2014, their power consumption continuously increased and showed a pronounced fluctuation range. This variation occurred at the same time as a higher fluctuation of the relative pressure in the vacuum tank was registered (Figure 91, Figure 92).

The variations began in October 2014 and were particularly pronounced after December 2014. After May 2015, the power consumption of the vacuum pumps dropped again, but the variation remained high.

In June 2015 and July 2015, the vacuum pumps consumed about 77.2 kWh/d, in March and April 2015, the average electricity demand was 108 kWh/d. The lower consumption of the vacuum pumps may be connected to the water level in the vacuum tank. Figure 93 shows the power demand of the vacuum pump and the water level in the vacuum tank during daytime operation (6:00 to 20:00). Between October 2014 and May 2015, the water level was higher than in the periods before and after. The power demand of the vacuum pump also reached higher values (Figure 94).

Figure 94 shows the power demand of the vacuum pump and the water level in the vacuum tank during nighttime operation (8:00 pm to 6:00 am). Between October 2014 and May 2015, the water level was higher than in the periods before and after, but the power demand remained at the same level.

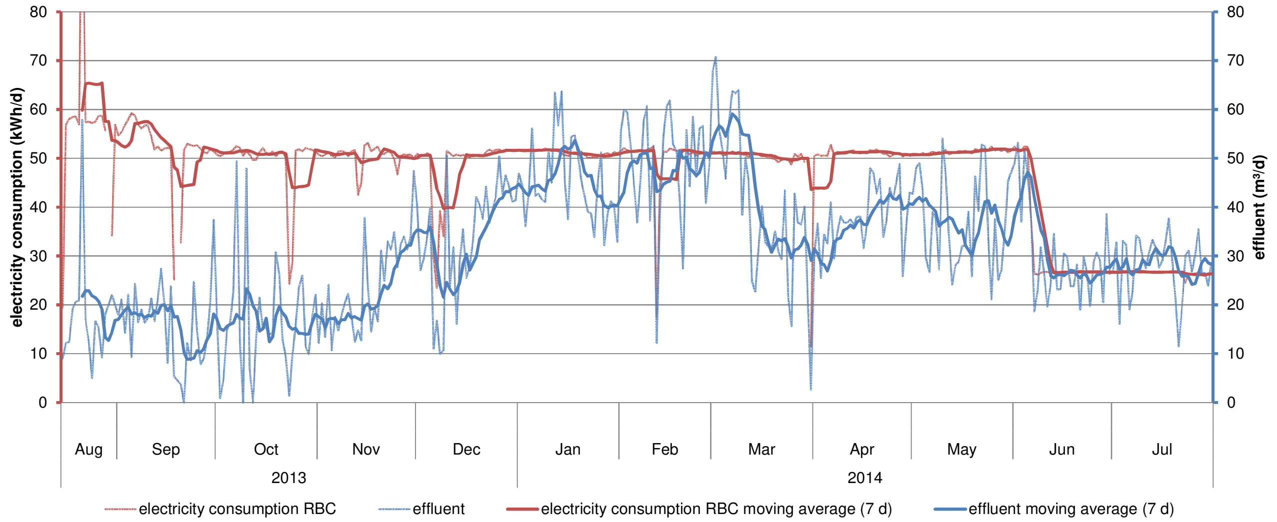

Figure 85 Electricity consumption of the RBCs and treated water quantities (August 2013 to July 2014)

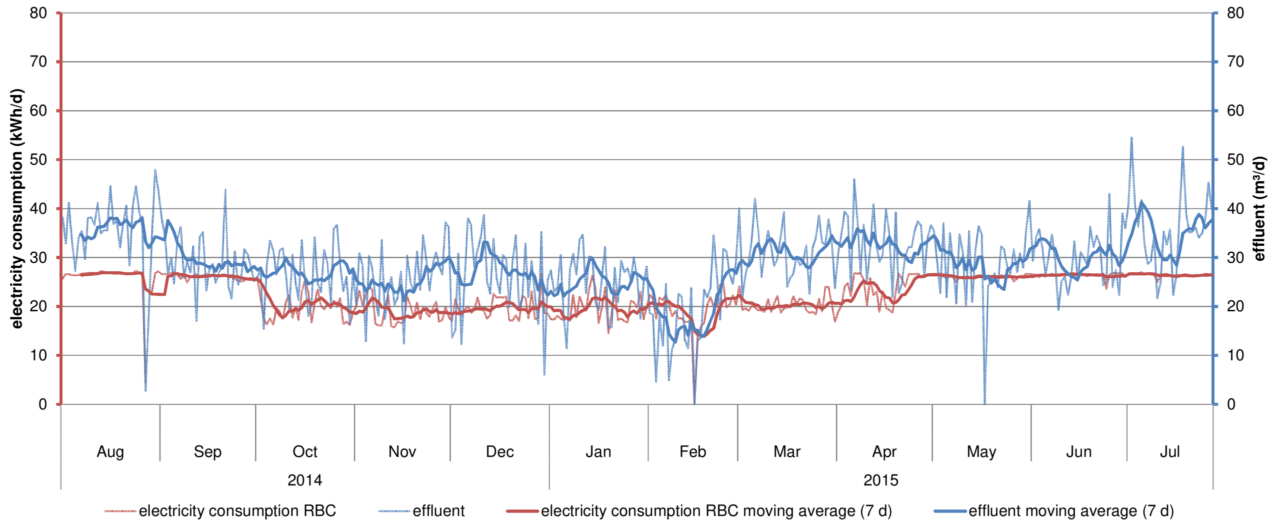

Figure 86 Electricity consumption of the RBCs and treated water quantities (August 2014 to July 2015)

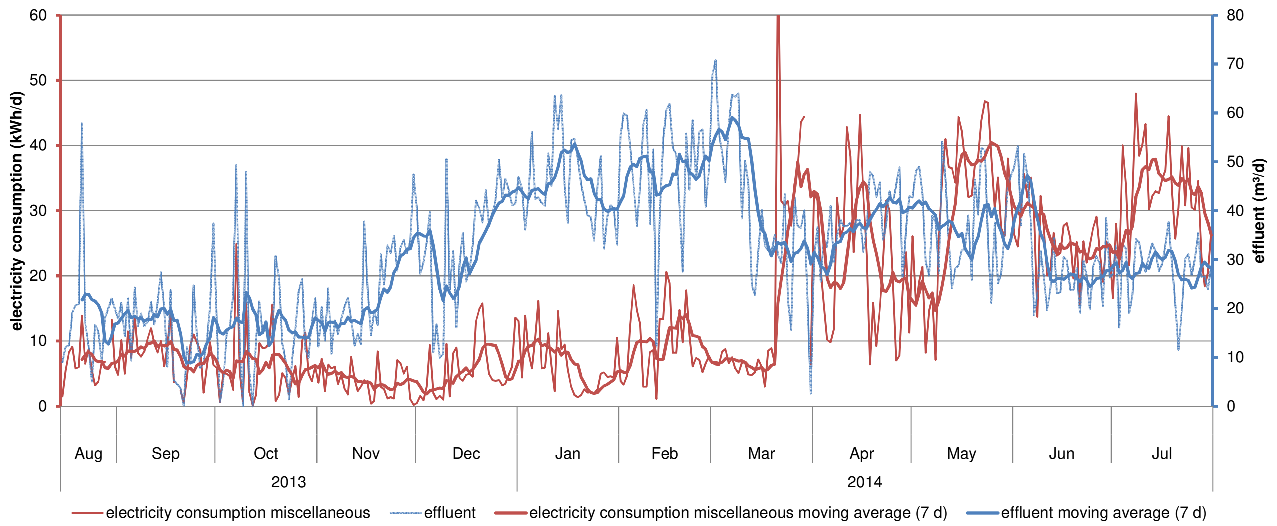

Figure 87 Electricity consumption for operation of the microscreen, UV disinfection, recirculation, and service water provision and treated water quantities (August 2013 to July 2014)

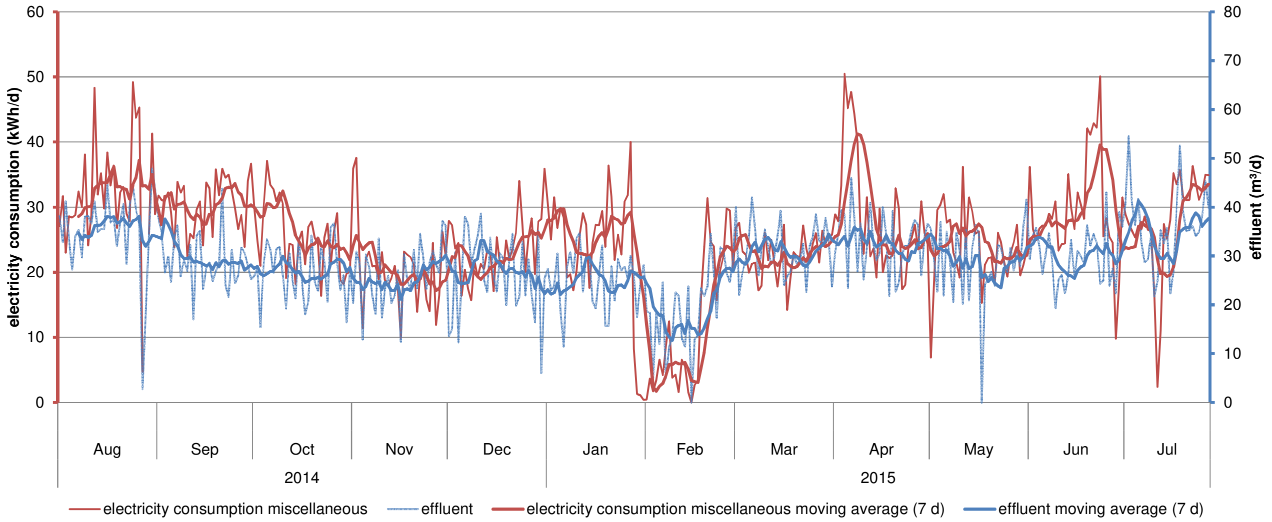

Figure 88 Electricity consumption for the operation of the microscreen, UV disinfection, recirculation, and service water provision and treated water quantities (August 2014 to July 2015)

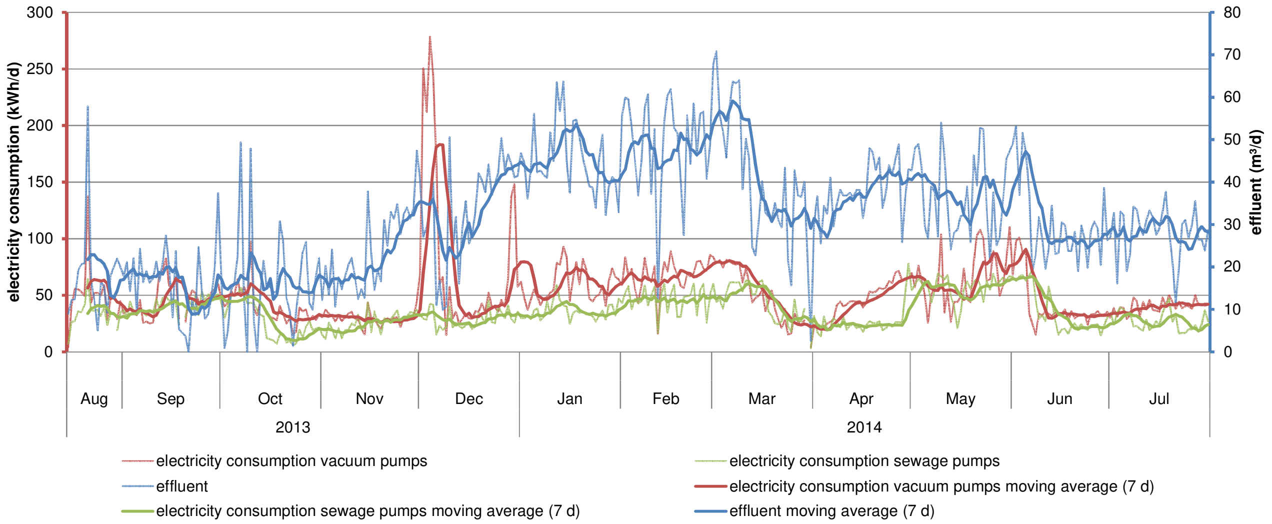

Figure 89 Electricity consumption of the vacuum and sewage pumps, treated water quantities (August 2013 to July 2014)

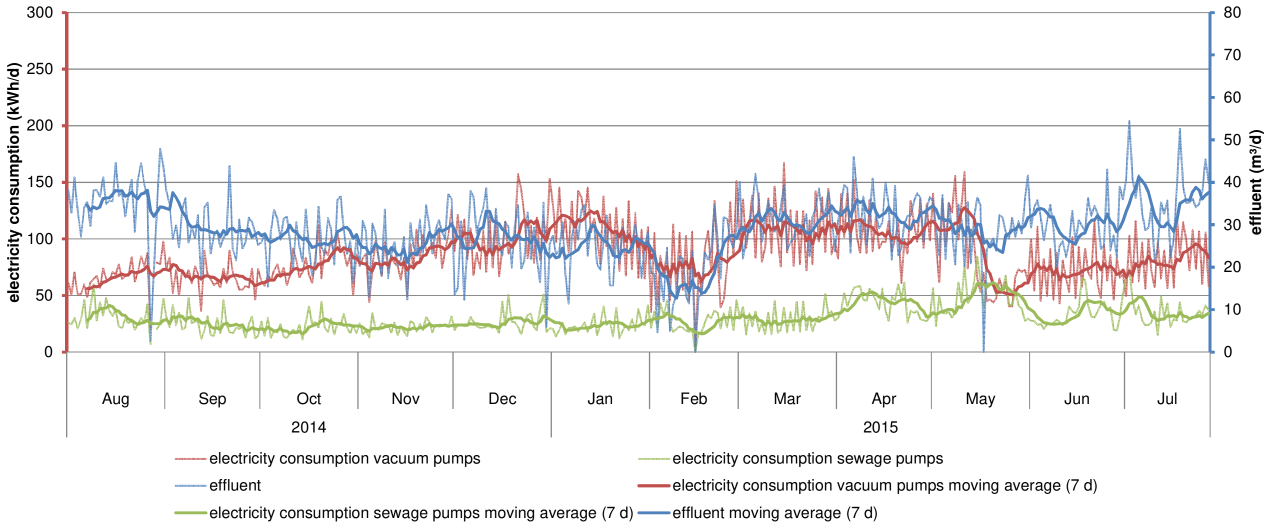

Figure 90 Electricity consumption of the vacuum and sewage pumps, treated water quantities (August 2014 to July 2015)

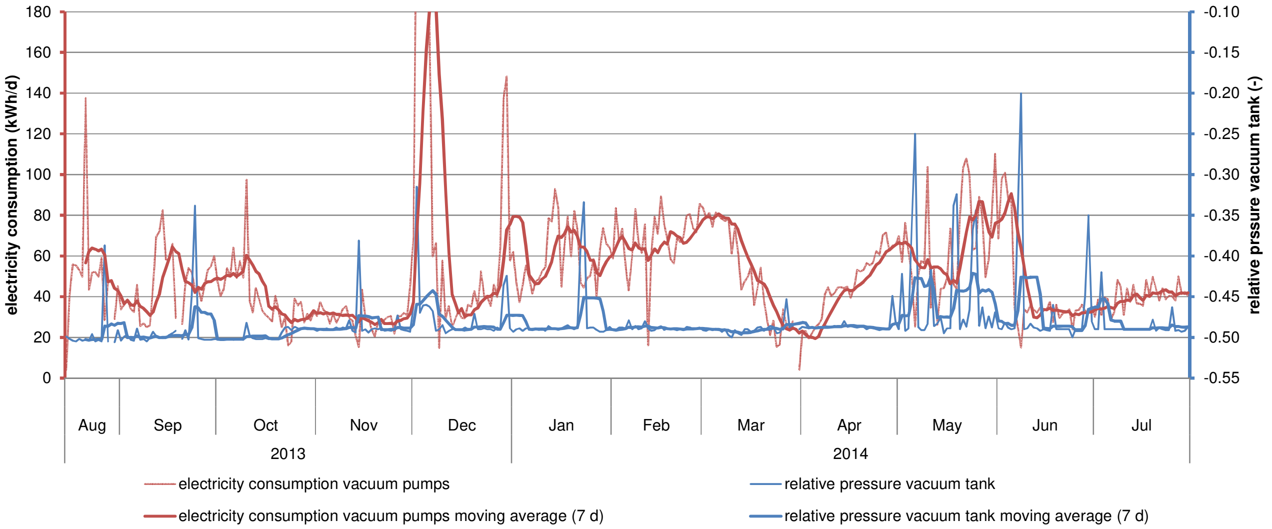

Figure 91 Electricity consumption of the vacuum pumps and relative pressure in the vacuum tank (August 2013 to July 2014)

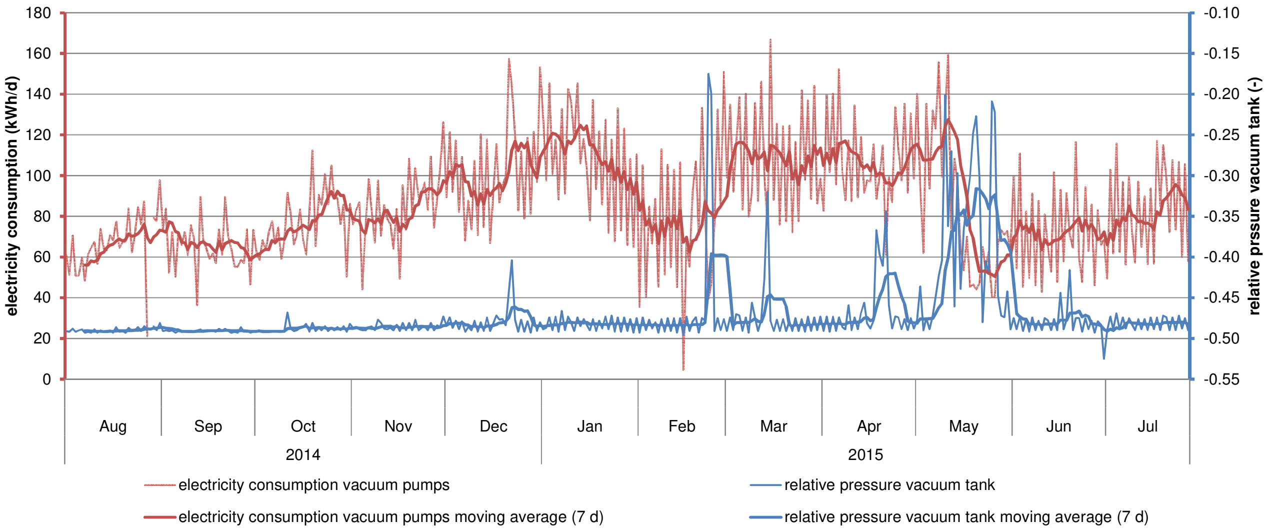

Figure 92 Electricity consumption of the vacuum pumps and relative pressure in the vacuum tank (August 2014 to July 2015)

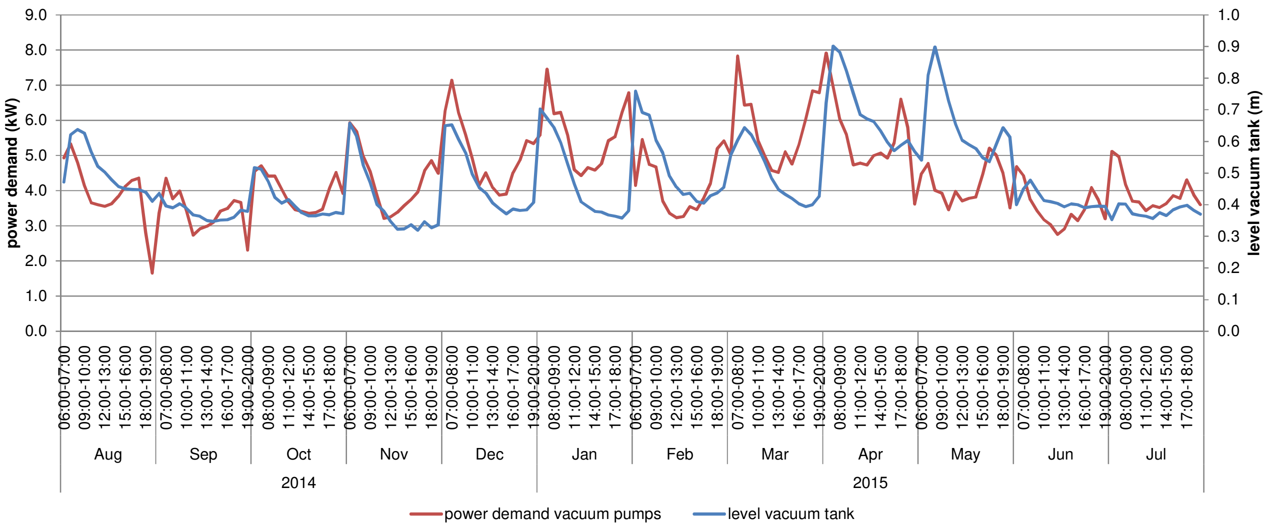

Figure 93 Power demand of the vacuum pumps and the water level in the vacuum tank during daytime operation from 6:00 am to 8:00 pm (August 2014 to July 2015)

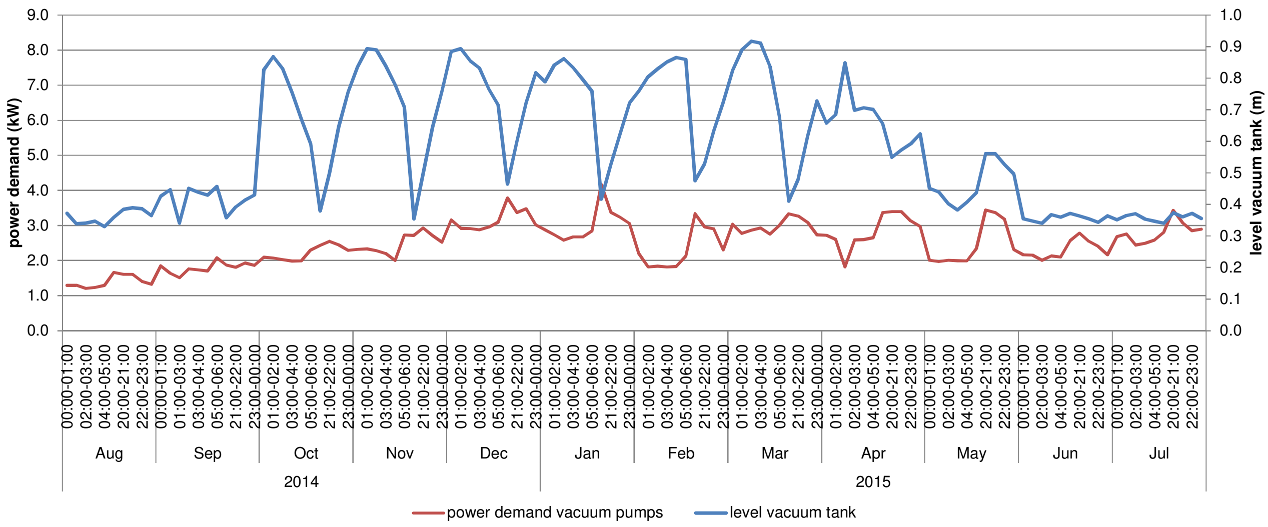

Figure 94 Power demand of the vacuum pumps and the water level in the vacuum tank during nighttime operation from 8:00 pm to 6:00 am (August 2014 to July 2015)

At the end of May 2015, the night operation mode was terminated, so that the water level in the vacuum tank declined. Simultaneously, the power consumption of the vacuum pumps decreased while the treated water volumes remained at the same level. It appears that, during the night operation mode, the higher water level in the vacuum tank led to a higher power consumption of the vacuum pumps.

4.7.2.3 Electricity demand per population equivalent

For comparison, the electricity consumption was referred to the population equivalents expressed by the TCOD load in the untreated water. One population equivalent corresponds to 100 g TCOD, which is considered a typical specific load in developing countries (Sperling 2007c).

In Outapi, the TCOD load increased during the first year of operation because the sanitation facilities went into operation consecutively. After May 2014, the TCOD loads remained relatively constant (Figure 95). On average, the TCOD loads calculated for the time period from May 2014 to June 2015 in the influent of the wastewater treatment plant corresponded to 332 (±131) population equivalents (pe).

The specific electricity consumption was 293 kWh/(pe×a). Of this, 105 kWh/(pe×a) were used for operation of the vacuum pumps and 67 kWh/(pe×a) for the operation of wastewater treatment (RBCs, microscreen, UV disinfection, service water supply, and recirculation).

The power consumption of the UV disinfection was not monitored separately, but can be estimated with reference to the lamp output. The UV system contains 12 emitters with a power of 70 W (Xylem Water Solutions 2012). For a daily operation time of 24 hours, this results in a power consumption of 20 kWh/d (= 70 W/emitter × 12 emitters × 24 h/d ÷ 1,000 Wh/kWh). Thus, the specific power consumption per inhabitant per year would decrease by 22 kWh

(= 20 kWh/d × 365 d/a ÷ 331 pe) to 44.9 kWh/(pe×a) for wastewater treatment and to 166 kWh for total electricity consumption, excluding the vacuum sewers (= 293 kWh/(pe×a) - 105 kWh/(pe×a) - 22 kWh/(pe×a)). For the following comparison with usual values in German wastewater treatment plants, the power consumption of the UV disinfection was excluded. However, this did not change the specific electricity consumption considerably.

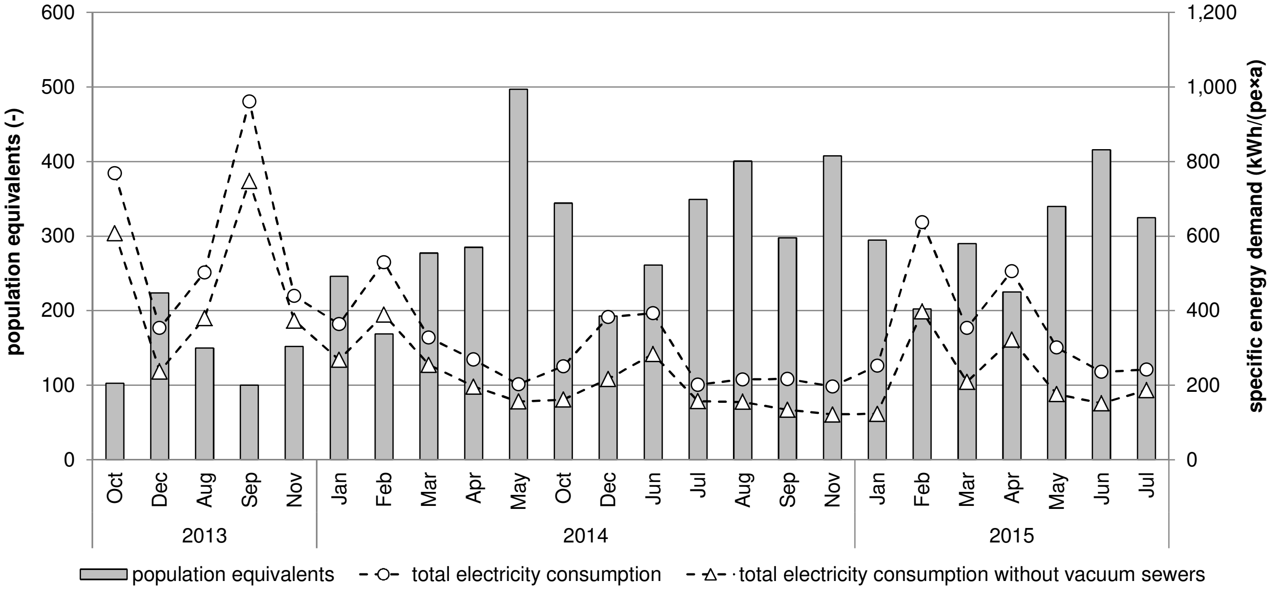

Figure 95 Population equivalents and power consumption (total power consumption without vacuum pumps) per population equivalent (pe)

German wastewater regulations distinguish between five size ranges of wastewater treatment plants, based on the BOD_5 load (AbwV 2014). A specific BOD_5 load of 60 g/(pe×d) is assumed (ATV-DVWK 2000). Plants with a daily BOD_5 load of < 60 kg/d (category 1), 60 to 300 kg/d (category 2), 300 to 600 kg/d (category 3), 600 to 6,000 kg/d (category 4) and > 6,000 kg/d (category 5) are considered (AbwV 2014).

The specific TCOD load is twice the specific BOD_5 load (ATV-DVWK 2000; Sperling 2007c). Hence, the plant in Outapi corresponds to category 1, which is characterized by a daily BOD_5 load below 60 kg (AbwV 2014) or a daily TCOD load below 120 kg/d, or less than 1,000 pe.

In German wastewater treatment plants the size of category 1, the average specific power consumption is 75 kWh/(pe×a) (Haberkern *et al.* 2008). In a more recent publication, an even lower average of 54.1 kWh/(pe×a) or a median value of 58.2 kWh/(pe×a) is estimated for this plant size (DWA 2011). It is acknowledged that the range of the energy consumption in wastewater treatment plants in the size range of categories 1 to 2 (< 5,000 pe) is very high, regardless of the kind of treatment steps applied (Haberkern *et al.* 2008). For the investigated wastewater treatment plants in Germany, this is mainly due to the stronger influence of special aggregates and less to the aggregates used (e.g., lower degree of efficiency of smaller motors and pumps and sub-optimal regulation and control of smaller devices) (Haberkern *et al.* 2008).

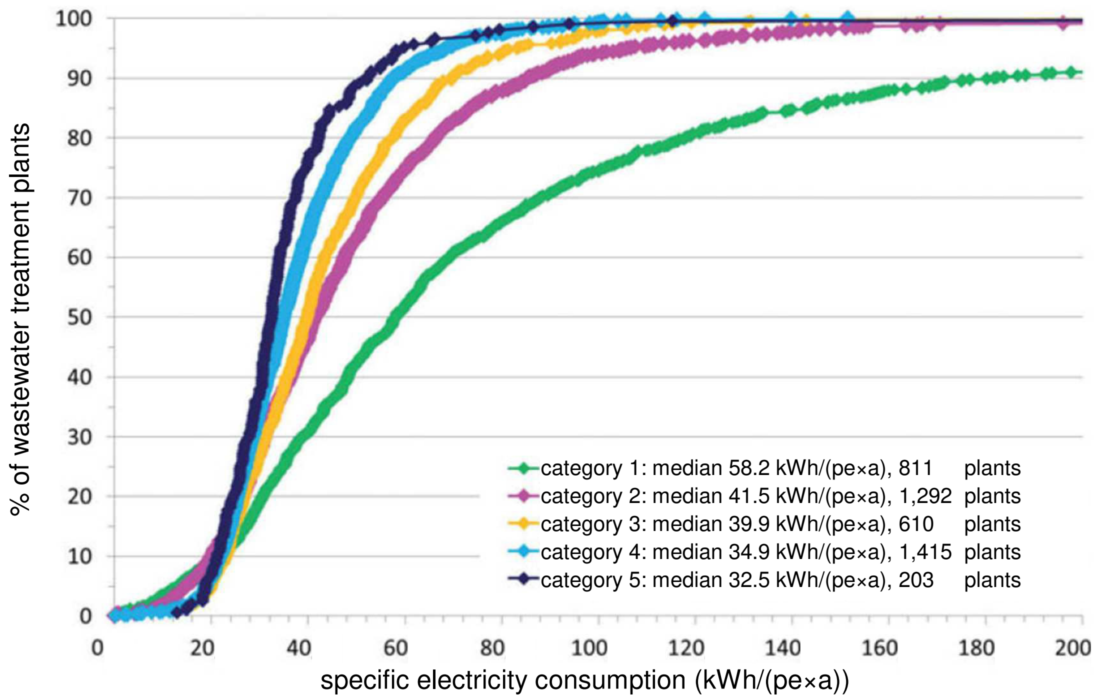

Figure 96 Cumulative frequency of the specific power consumption of a total of 4,331 wastewater treatment plants in Germany, of which 811 belong to the size range of category 1 (DWA (2011), modified), pe = population equivalents

The specific power consumption of the plant in Outapi is relatively high compared to these literature values. Its specific power consumptions of 188 kWh/(pe×a) (with UV disinfection, without vacuum sewers) and 166 kWh/(pe×a) (without UV disinfection, without vacuum sewers) are among the 10% of plants with the highest specific power consumption (Figure 96). During planning, a total electricity consumption of 250 kWh/d, including vacuum sewers, was assumed for 1,500 pe. Accordingly, the specific electricity consumption would be 60.8 kWh/(pe×a), which is very close to the median value for category 1 (Figure 96). However, the monitored specific energy consumption is three times higher than assumed during planning. This is mostly due to the smaller water quantities (see Figure 101, page 182).

4.7.2.4 Electricity demand per cubic meter of treated water

The power consumption increases with increasing water quantities (Figure 97, Figure 98). The variation of the gap between the lines marking water quantities and power consumption indicates whether the power consumption per cubic meter is increased or decreased.

After start-up of the communal washhouse and the wastewater treatment plant, the electricity consumption was relatively high, compared to the treated water quantities. The water quantities increased continuously from November 2013, due to commissioning of the cluster units, resulting in a more favorable ratio between treated water quantities and electricity consumption (Figure 99). From December 2013 to January 2015, the balance between power and treated water volume remained roughly the same. From September 2014, the treated water quantities

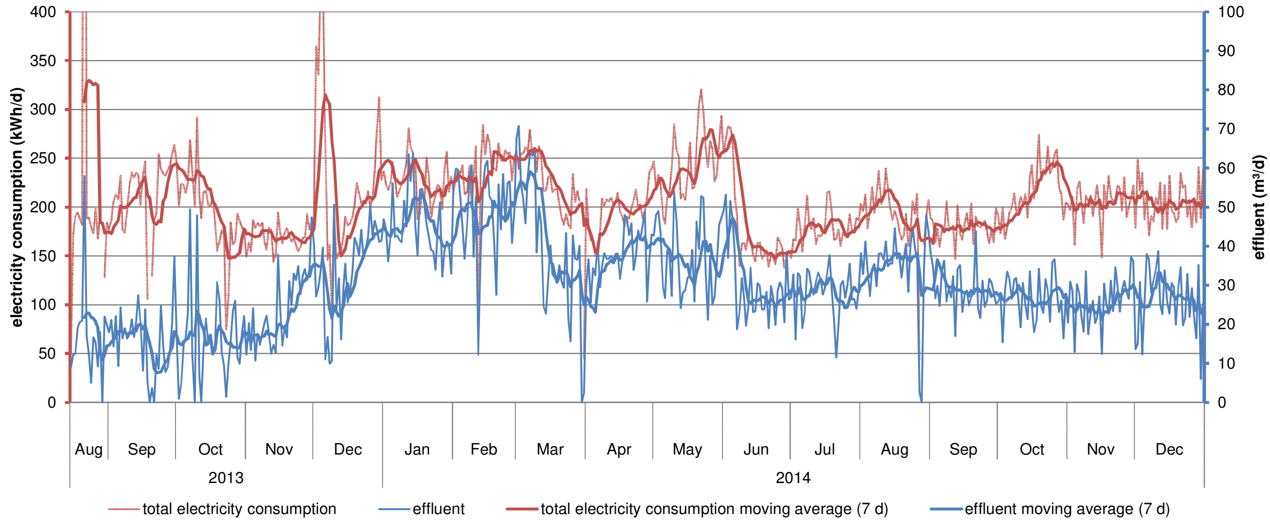

Figure 97 Total power consumption and treated water quantities from August 2013 to July 2014

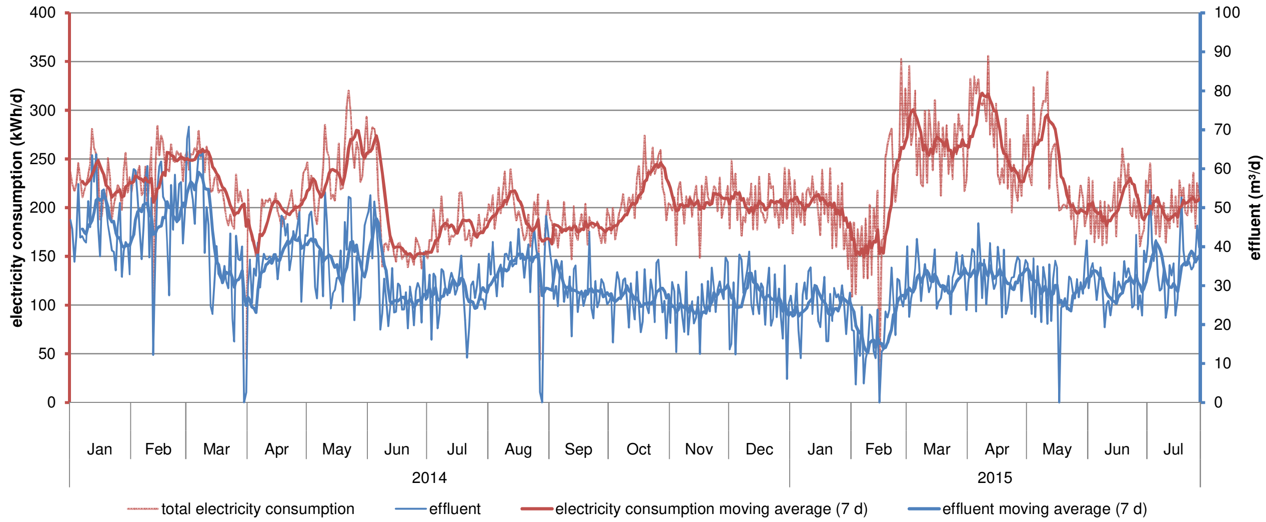

Figure 98 Total power consumption and treated water quantities from August 2014 to July 2015

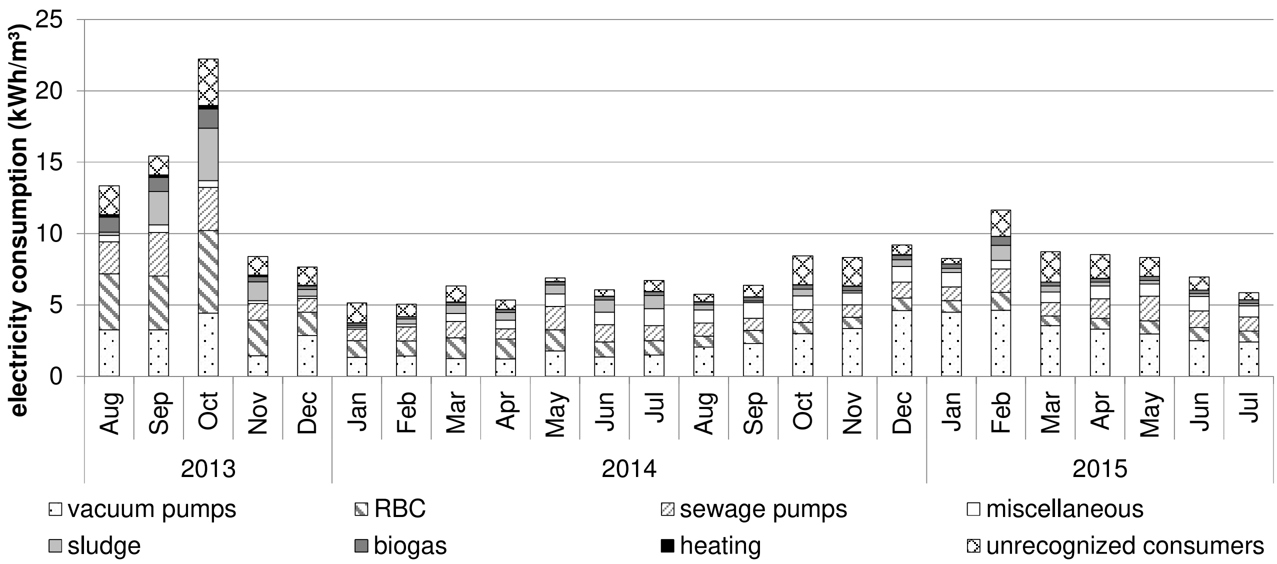

Figure 99 Power consumption of the individual components per cubic meter of treated water

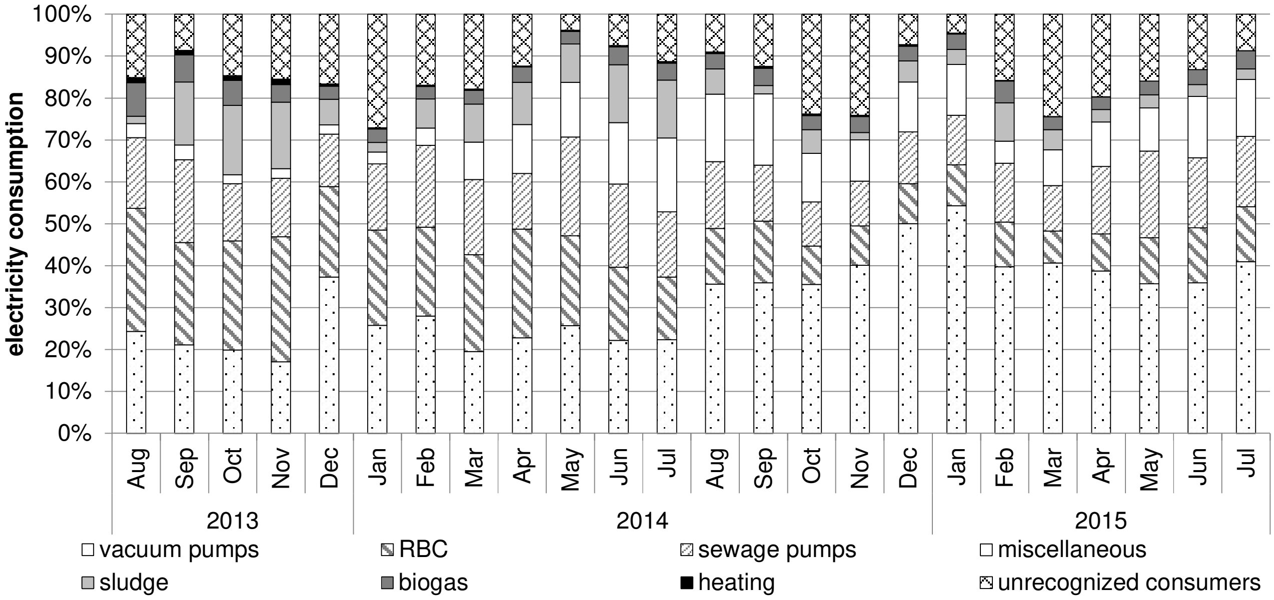

Figure 100 Percentage of the electricity consumption for groups of power consumers per cubic meter of treated water

decreased slightly, but the electricity consumption remained at the same level. From February 2015 to May 2015, the power consumption per treated cubic meter of water was relatively high and the electricity consumption per cubic meter of treated water also increased. At the end of May 2015, the power consumption decreased to values achieved before February 2015. At the same time, the amounts of water increased slightly; thus, the electricity consumption per cubic meter decreased slightly.

On average, the power consumption was 8.4 kWh/m³, whereby 2.7 kWh/m³ or 32% were on account of the vacuum pumps, 1.3 kWh/m³ or 15% were on account of the sewage pumps, and 2.2 kWh/m³ or 26% were consumed by wastewater treatment and recirculation (Figure 99).

The months with the lowest power consumption per cubic meter of treated water were January 2014, February 2014, April 2014, August 2014 and July 2015 (Figure 99). During these months, the specific power consumption was 5.0 to 5.9 kWh/m³ (mean: 5.4 kWh/m³) and the amount of water treated was relatively high (mean: 40.7 m³/d). To keep the specific power consumption permanently relatively low, the quantity of treated water needs to be relatively high. This could be achieved by connecting additional households or sanitation facilities to the sanitation system. Figure 101 shows the specific power consumption and the amount of treated water. Under optimal conditions, power consumptions of 3 to 4 kWh/m³ could be achieved.

Further savings could result from the decommissioning of unnecessary components, such as the devices for biogas storage and biogas processing facilities, sludge treatment, and heating water. During the project period, these components consumed 3.5%, 6.9% and 0.3% of the electricity, respectively. Thus, roughly 10% of the power consumption could be reduced by decommissioning (Figure 100).

Less energy was needed for the operation of the laboratory after completion of the technical monitoring. The power consumption of the laboratory is not monitored separately, but can be estimated from the power consumption during the vacations of the laboratory staff, compared to work periods. Between 16 August 2014 and 31 August 2014, the power consumption was 8.9 kWh/d (vacation time). Previously (16 July 2014 to 15 August 2014) and afterwards (1 September 2014 to 1 October 2014), it was at 23.8 and 24.2 kWh/d (working days). Thus, the operation of the laboratory equipment consumes about 15 kWh/d or 7% of the total power.

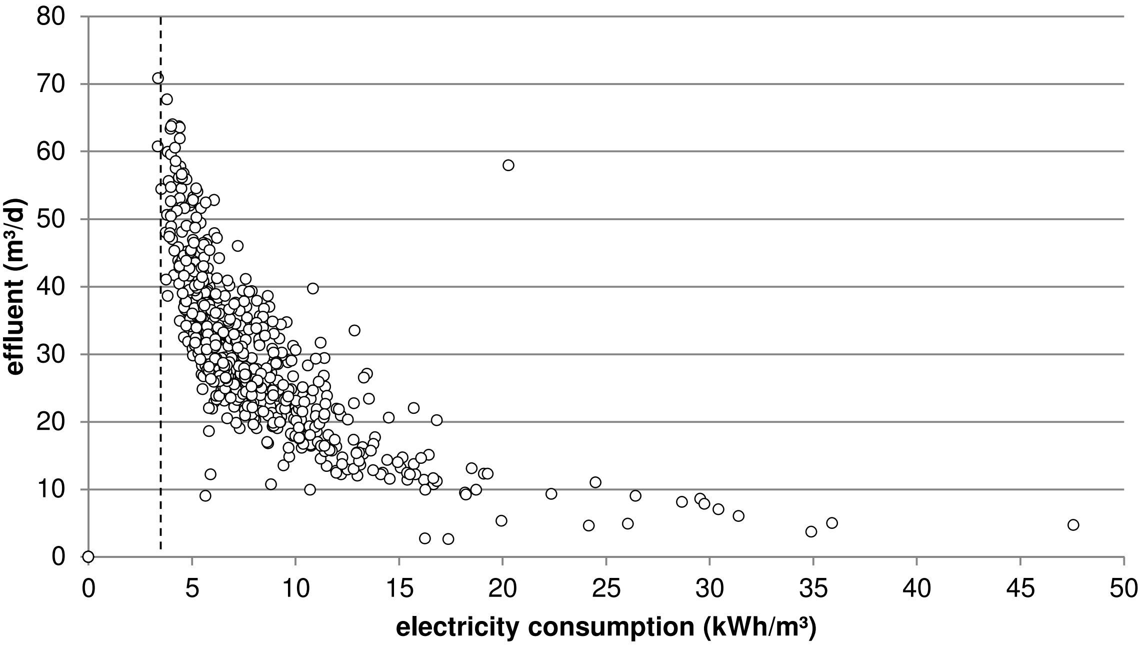

Figure 101 Relationship between the electricity consumption and water quantities

Considering unneeded plant components and the electricity requirement of the laboratory, 17% of the power consumption could be saved (10% from unneeded plant components and 7% from the laboratory). Hence, a specific power consumption of 2.5 to 3.3 kWh/m³ would be possible (2.5 kWh/m³ = 0.83 ×3 kWh/m³; 3.3 kWh/m³ = 0.83 × 4 kWh/m³). This is in accordance with the planned energy consumption of 2.8 kWh/m³, which is based on an electricity consumption of 250 kWh/d and a water demand of 90 m³/d. The previously determined power consumption of 188 kWh/(pe×a) (without vacuum pumps) could be reduced to between 103 to 139 kWh/(pe×a) by better utilization of the plant's hydraulic capacities. Through decommissioning unneeded components and the reduced laboratory consumption, a value of 86 to 116 kWh/(pe×a) could be achieved.

CEC (2005) consider 0.29 kWh/m³ a low energy intensity and a consumption of 1.2 kWh/m³ a high energy intensity for wastewater collection and treatment. Thus, even after optimization, the specific energy consumption of the plant in Outapi would be very high, compared to this Californian benchmark.

4.7.3 Regional tariff structures and their significance for energetic utilization of sewage sludge and agricultural residues

This section deals with the costs of the electricity consumed by the sanitation system in Outapi. First, the tariff structure of the regional electricity supply entity servicing North Namibia is presented. The fees imposed for Outapi are calculated. The tariff structure applied in Outapi was a major obstacle to implementation of co-generation. Hence, electricity tariffs of other supply entities in the region are examined to make a general statement if this finding can be generalized to the entire region or represents a characteristic specific to North Namibia.

The Namibian electricity distribution network is operated by NamPower, three major regional distributors, some municipalities and town councils (see Chapter 2.9.5, page 37). Most Namibian electricity suppliers charge customers via linear tariff concepts. This means that the unit price per kilowatt hour stays the same as consumption increases or decreases, in contrast to decreasing or increasing block tariffs (Briceno-Garmendia and Shkaratan 2011). An exception is ERONGORED (Erongo Regional Electricity Distributor) in West Namibia (e.g., Walfis Bay, Swakopmund and Henties Bay) that charges domestic customers with increasing block tariffs (ECB 2014a).

In the project region, the local electricity supplier, NORED, provides electricity to domestic users by a prepaid system with uniform charges per kilowatt hour and no fixed monthly charges (ECB 2014e). NORED does not offer postpaid billing to domestic customers. Other local electricity providers also offer post-paid tariffs to domestic customers with a fixed monthly fee and variable charges that depend on the consumed kilowatt hours, for instance CENORED (Central North Regional Electricity Distributor) in Grootfontein or Kamanjab (ECB 2014c) or OPE (Oshakati Premier Electric) in Oshakati (ECB 2014f).

The tariff system for uses other than domestic consumption is more complex. The tariffs for non-domestic users usually foresee a three-part cost structure, divided into a fixed monthly fee (network charge), a power price that is set on the basis of the peak power demand (capacity charge) and the fee for energy consumption (energy charge). The capacity charge is calculated with a contractually agreed maximum power to be provided.

Pricing for energy consumption of non-domestic consumers is further differentiated according to the time of use (Table 45). There is a low demand season from September to May and a high demand season from June to August (ECB 2014c). The prices per kilowatt hour are higher in the high demand season than in the low demand season (ECB 2014c). In addition, pricing for the energy consumption differs, depending on the times of the day. A distinction is drawn between peak time, standard time, and off-peak time (ECB 2014c). The time slots differ during high demand and low demand seasons and weekdays, Saturdays, and Sundays (ECB 2014c).

In addition, two fixed fees are charged for each kilowatt hour consumed. The ECB levy (ECB = Electricity Control Board) is paid directly to the ECB of Namibia, in order to provide sufficient funding for this institution (Cenored 2016). The NEF levy (NEF = National Energy Fund) was introduced in July 2013 by the Government of Namibia for financing of, e.g., electrification, soft loans for energy projects, and subsidies (MME 2016).

Table 45 Overview of relevant time periods for calculation of the energy charge in NORED`s time-of-use tariffs (ECB 2014e) and monitored average monthly power consumption of the vacuum sewers and wastewater treatment plant; the average values represent all monitoring data, i.e. also values collected in 2013

NORED tariff details for energy charge time periods for time-of-use tariffs 2014/2015				**monitored energy consumption** average 2013-2015 kWh/month	2014 kWh/month	2015 kWh/month
low season (September to May)	Mon-Fri	peak time	8:00 h -13:00 h 18:00 h - 21:00 h	1,773	1,718	1,995
		standard time	6:00 h - 8:00 h 13:00 h - 18:00 h 21:00 h - 22:00 h	1,772	1,743	1,951
		off-peak time	22:00 h - 6:00 h	1,176	1,204	1,122
	Sat	standard time	7:00 h - 12:00 h 18:00 h - 20:00 h	319	318	376
		off-peak time	0:00 h - 7:00 h 12:00 h - 18:00 h 20:00 h - 24:00 h	622	617	692
	Sun	off-peak time	0:00 h - 24:00 h	901	884	968
high season (June to August)	Mon-Fri	peak time	7:00 h -12:00 h 17:00 h - 20:00 h	1,335	1,432	1,532
		standard time	5:00 h - 7:00 h 12:00 h - 17:00 h 20:00 h - 21:00 h	1,453	1,416	1,397
		off-peak time	21:00 h - 5:00 h	972	985	1,203
	Sat	standard time	6:00 h - 11:00 h 17:00 h - 19:00 h	251	273	253
		off-peak time	0:00 h - 6:00 h 11:00 h - 17:00 h 19:00 h - 24:00 h	486	502	519
	Sun	off-peak time	0:00 h - 24:00 h	759	847	824

Table 46 is a survey of tariffs of several Namibian, one Botswanan (BPC = Botswana Power Corporation) and one South African (Eskom) supply entity and the respective energy costs for the average consumption of the Outapi vacuum sewers and wastewater treatment plant. The time periods for low and high seasons are identical in Namibia and South Africa. The time periods for peak, standard, and off-peak times are almost identical. These minor differences were neglected when calculating the energy charges in the second part of the table. Average currency exchange rates of 14.31 NAD/EUR, 14.31 ZAR/EUR and 11.05 BWP/EUR were assumed (average currency exchange rates in 2015, www.oanda.com, ZAR = South African Rand, BWP = Botswana Pula).

From the various tariffs offered by the supply entities, the one that seemed suitable was chosen for the calculations in Table 46. For the sanitation system in Outapi, NORED calculates with the tariff for “large power users”. Accordingly, this tariff was also chosen from the schedules

Table 46 Overview of charges and levies of several electricity supply entities in Namibia, Botswana (BPC) and South Africa (Eskom), OPE = Oshakati Premier Electric, AVC = Aranos Village Council, KaMu = Karasburg Municipality, KeMu = Keetmanshoop Municipality, MaMu = Mariental Municipality, BPC = Botswana Power Corporation, ECB = Electricity Control Board Namibia (except Eskom: "ancillary service charge"), NEF = National Energy Fund Namibia (except Eskom: "electrification and rural network subsidy charge", "affordability subsidy charge") (BPC 2015; ECB 2014a, 2014e, 2014c, 2014f, 2014d, 2014d, 2014b; Eskom 2015)

charges and levies			**electricity supply entity**									
description		**unit**	**NORED**	**CENO-RED**	**OPE**	**ERONGO-RED**	**AVC**	**KaMu**	**KeMu**	**MaMu**	**BPC**	**Eskom**
pricing structure												
energy charge low season	peak time	EUR/kWh	0.10	0.14	0.10	0.11	0.15	0.15	0.15	0.11	0.14	0.06
	standard time	EUR/kWh	0.08	0.12	0.09	0.09	0.12	0.09	0.09	0.10	0.14	0.04
	off-peak time	EUR/kWh	0.05	0.10	0.07	0.07	0.09	0.07	0.07	0.08	0.14	0.03
energy charge high season	peak time	EUR/kWh	0.16	0.21	0.17	0.18	0.20	0.20	0.20	0.18	0.14	0.20
	standard time	EUR/kWh	0.11	0.15	0.12	0.12	0.13	0.11	0.11	0.12	0.14	0.06
	off-peak time	EUR/kWh	0.07	0.12	0.09	0.09	0.11	0.09	0.09	0.10	0.14	0.03
network charge		EUR	63.69	103	74.30	108	285	285	285	-	6.02	34.12
capacity charge		EUR /(kVA ×month)	11.96	26.04	15.99	19.04	10.44	10.44	10.44	11.53	-	3.56
ECB levy		EUR/kWh	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	-	0.003
NEF levy		EUR/kWh	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	-	0.007
costs for average energy consumption of the sanitation system in Outapi during whole monitoring period (2013-2015)												
energy charge low season	peak time	EUR/month	172	246	183	191	260	260	260	198	254	114
	standard time	EUR/month	170	259	185	194	244	195	195	203	300	92.80
	off-peak time	EUR/month	145	275	180	191	241	191	191	202	387	75.99
energy charge high season	peak time	EUR/month	213	280	232	238	267	266	266	244	191	264
	standard time	EUR/month	182	258	198	205	228	180	180	212	244	102
	off-peak time	EUR/month	157	271	193	202	246	196	196	212	318	72.1
network charge		EUR/month	63.69	103	74.30	108	285	285	285	-	6.02	34.12
capacity charge		EUR/month	598	1,302	800	952	522	522	522	577	-	178
ECB levy		EUR/month	6.63	6.63	6.63	6.63	6.63	6.63	6.63	6.63	-	15.92
NEF levy		EUR/month	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	-	44.23
sum low season		EUR/month	1,160	2,196	1,433	1,647	1,563	1,464	1,464	1,192	947	555
sum high season		EUR/month	1,226	2,225	1,509	1,717	1,560	1,460	1,460	1,256	759	710

of approved tariffs for the Aranos Village Council (AVC), Karasburg Municipality (KaMu) and Keetmanshoop Municipality (KeMu). For CENORED, the chosen tariff was “institutional large power users”, for Oshakati Premier Electric it was “institutional (large) time of use”; for ERONGORED “institutional (bulk connections)” and for Mariental Municipality “bulk connections TOU” (TOU = time of use) were chosen.

The Botswana Power Corporation offers a special tariff for government and municipal installations (“government TOU 2”, TOU = type of user). This tariff has linear energy charges and does not require payment of a demand charge (BPC 2015).

The South African Eskom is the only energy supplier that offers a tariff category for customers that consume and provide energy at the same metering point (“genflex”, Eskom (2013)). The energy produced during peak, standard, and off-peak times is used to calculate a rebate that is subtracted from the network charges. However, the network charges cannot be less than zero (Eskom 2013). Thus, electricity generation cannot overcompensate costs.

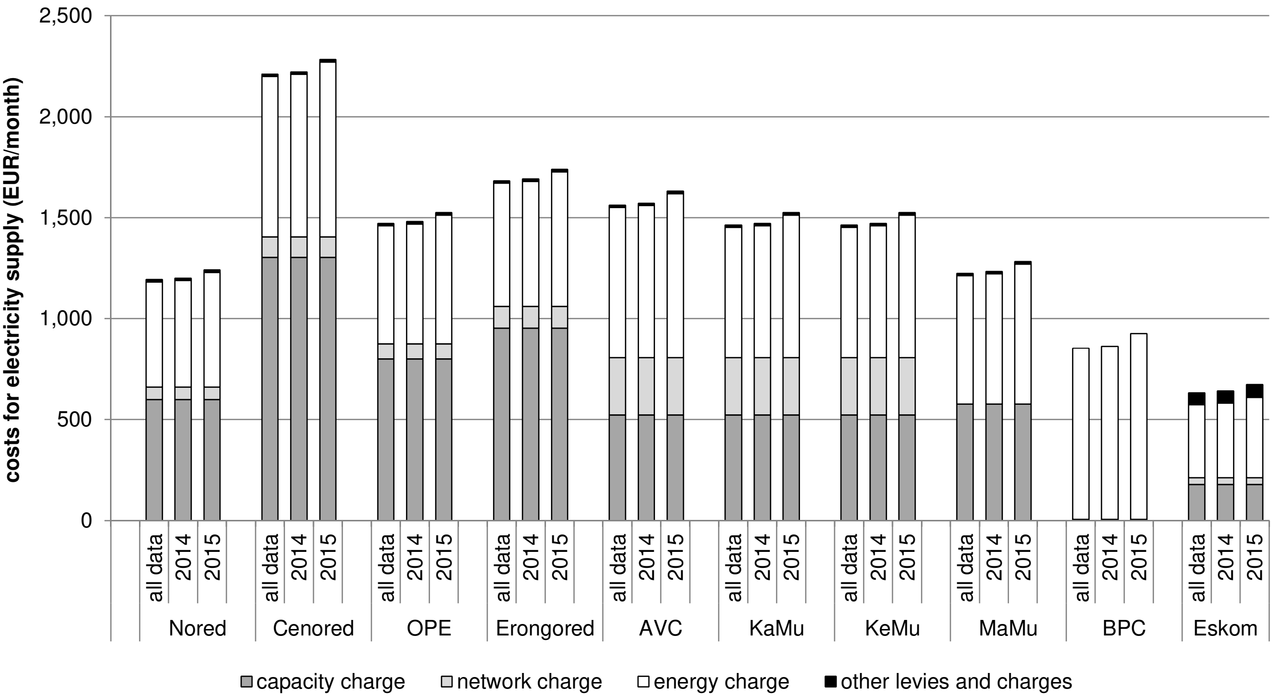

Figure 102 Calculated charges and levies for the monitored energy consumption in Outapi, based on the tariffs of several electrical supply entities in Namibia, Botswana, and South Africa, “all data” also includes electricity demand data collected in 2013; OPE = Oshakati Premier Electric, AVC = Aranos Village Council, KaMu = Karasburg Municipality, KeMu = Keetmanshoop Municipality, MaMu = Mariental Municipality, BPC = Botswana Power Corporation (BPC 2015; ECB 2014a, 2014e, 2014c, 2014f, 2014d, 2014d, 2014b; Eskom 2015), average currency exchange rate 2015: 14.31 NAD/EUR, 14.31 ZAR/EUR and 11.05 BWP/EUR (www.oanda.com)

Thus, electricity generation cannot overcompensate costs. The monitored electricity consumption (three right columns in Table 45) was used to calculate the corresponding energy charges of the energy suppliers in Figure 102. The energy costs vary from 633 EUR/month for the chosen Eskom tariff up to 2,200 EUR/month for the most expensive tariff (CENORED “institutional large power users”). Among the Namibian tariffs, the tariff applicable in Outapi (NORED “large power users”) is relatively low. However, the monthly costs for the Botswanan and South African tariff are considerably lower than the costs calculated for the Namibian tariffs.

Altogether, the Namibian costs are characterized by relatively high fixed costs (capacity charge, network charge), compared to the energy charges. The percentage of the fixed costs, compared to the total costs, varied between 45% and 64%. In South Africa, this percentage was only 33%. In Botswana, fixed costs were very low, due to the special tariff for municipalities (< 1% of the total costs). The costs for the consumed electricity were higher for the Namibian tariffs than for the South African tariff but lower than the Botswanan tariff. Because variable costs were relatively low in the Namibian cases, savings in kilowatt hours could reduce energy costs by maximally 36% to 54%, if all energy costs could be covered by co-generation. However, Eskom is the only energy supplier offering a tariff with credits for generated electricity. Thus, in the Outapi case, savings were only possible if the generated electricity matches the momentary demand of the sanitation system. Excess electricity that cannot be used by the plant is fed to the local grid but does not generate rebates.

Briceno-Garmendia and Shkaratan (2011) reviewed power tariffs for residential, commercial, and industrial purposes in Sub-Saharan African countries. They conclude that, for commercial and industrial consumers, the three-part tariff that includes a monthly fixed charge (e.g., network charge), a capacity charge and an energy charge is a very common model. Only a small number of countries uses a simpler linear tariff model (e.g., Botswana, BPC (2015)). Further differentiation by the time of use or a low and a high demand season, as in the Namibian and South African cases, is less common but also practiced by some countries. Briceno-Garmendia and Shkaratan (2011) further conclude that the "peak demand is a critical cost driver in the power sector, because it defines the amount of installed capacity needed to provide a given volume of electricity". Thus, even though only a limited number of tariffs was examined in this study, it is concluded that the basic challenges experienced in the Outapi case are also applicable in other Sub-Saharan countries. No credits or compensation for generated electricity and the high percentage of fixed costs, compared to the total electricity costs, represent important barriers to the implementation of on-site co-generation units at wastewater treatment plants in this region.

4.7.4 Conclusions

This section addressed the potential methane yields obtainable from agricultural crops, crops residues and sewage sludge, the monitored total and specific electricity consumption of the sanitation system, and the relevance of tariffs for implementation of co-generation units.

Leafy biomass has lower specific methane yields than the whole plant, because it often contains less proteins, carbohydrates, and fats than the harvested crop parts. The highest methane yield can be obtained from maize and wheat residues, because TS contents of the fresh matter are relatively high and harvest indices are relatively low, compared to other crops.

The contribution that co-digestion of agricultural residues can make towards achieving calculated energy self-sufficiency depends on the kind of cultivated crops and the available irrigation water quantities or the size of the irrigated area. In Outapi, the total size of the agricultural fields increased from 1 ha in the beginning of the project to 3 ha at its completion. On these relatively small areas, crop residues can contribute only to a limited extent towards achieving electricity self-sufficiency. Most of the producible electricity originates from the biogas potential of sewage sludge.

By calculation, the electricity demand could be covered to a high degree if the whole agricultural area of 3 ha were to be cultivated with crops that produce large amounts of residues, such as maize or wheat. On smaller areas, cultivation of energy crops and utilization of the whole plant for biogas production could achieve calculated energy self-sufficiency.

For each sanitation system, a minimum irrigation water quantity or minimum field size can be determined that is needed for achieving calculated electricity self-sufficiency via co-digestion of agricultural biomass with sewage sludge. This minimum water quantity or field size depends on the electricity requirement of the infrastructure, the producible biogas or electricity per cubic meter of wastewater, the kind of cultivated crops (irrigation requirement, specific methane yield) and whether the whole plant or only the residues are utilized for biogas production. In this case, the minimum required water quantities ranged from 66 m^3/d to 140 m^3/d. The electricity demand of the implemented sanitation system is relatively high. Thus, in theory, calculated electricity self-sufficiency could be achieved by relatively small sanitation systems that are operated with a lower electricity demand.

The thermal energy requirement can be covered to a high degree, even when a relatively small agricultural area is available. However, most of the thermal energy is contributed by sewage sludge.

Energy recovery from sewage sludge and biomass was not fully implemented in Outapi. The trade-off of biomass use – either as food for humans and livestock or as an energy source – led to the reduction of the biomass available for energy production. Because the TS and TCOD content and flows of the untreated wastewater were much lower than planned, less sewage sludge and biogas were produced. Furthermore, the energy price per kilowatt hour was low and it was not possible to feed the generated power into the grid of the local electricity supplier for revenue generation. Last, but not least, it was hard to maintain the interaction between the operators of the wastewater treatment plant and the operators of the irrigation site.

Blackouts were frequent in the project area. Only during 4 out of 24 monitored months, was an emergency generator not required for electricity supply. Thus, biogas utilization for electricity generation could be an option to bridge time periods without electricity supply from the local electricity provider.

The specific electricity consumption of the implemented sanitation system was relatively high, compared to German and Californian benchmarks. This was still the case after accounting for higher treated water quantities and decommissioning of components that will be no longer required after final handover to the OTC.

Optimal use of the co-generation unit was prevented by the tariff structure of the local electricity supply entity. The energy charge per kilowatt hour was relatively low, compared to the fixed charges, and it was not possible to receive a rebate for surplus energy fed into the grid. Thus, generated electricity is only beneficial if it is consumed immediately at the plant or if it can help to reduce the peak energy use, in order to achieve lower capacity charges.

A literature review on electricity tariffs in the region made clear that the impediments experienced in Outapi also exist in other regions in Namibia, South Africa, Botswana and many other countries in Sub-Saharan Africa. Consequently, to promote co-generation at wastewater treatment plants, electricity supply entities first need to adjust their current tariff structures.

5 Conclusions and outlook

In the light of a growing urban population and growing water scarcity, prior work in the field of sanitation and water reuse has emphasized the need for a holistic and integrated view of all involved components. This includes the hardware components (e.g., toilets and sewers) and the software components (e.g., community involvement, management issues). A systemic approach is needed to recover the resources contained in sewage and to maximize the benefits of sanitation systems. There is a consensus that this approach needs to be further promoted.

The main obstacles to the implementation of new approaches to urban water management are the lack of knowledge regarding inherent uncertainties and risks, practical management challenges, the available institutional capacities and capacities to facilitate community involvement, financial considerations, and institutional and personal biases that act as barriers (Marlow *et al.* 2013).

5.1 Project summary

This dissertation explored the main challenges and knowledge gaps encountered during implementation of a sanitation system in North Namibia. The objective of this research project was to develop and implement a sanitation system that responds to the water and sanitation challenges in the region. Together with local stakeholders, the city of Outapi was chosen for the implementation of a pilot project. The infrastructure that was provided includes three types of sanitation facilities in three differently developed informal settlements (communal washhouse, cluster units, and households with individual water and sewage connections), a vacuum sewer system, a wastewater treatment plant, a storage pond, and an agricultural irrigation site.

In order to match the local environmental and socioeconomic conditions and to fulfill the prerequisites of the Namibian Sanitation Strategy, the sanitation system had to comply with numerous requirements. On the technical side, the sanitation system needed to assure safe transport of excreta without contamination of the environment during the rainy season. It should be a flexible system that takes into account the dynamic population development in the region. Water and nutrients should be reused for agricultural irrigation. Organics contained in the wastewater and residues of the agricultural fields should be utilized energetically. Stabilized biosolids should be used as soil conditioners. To operate the infrastructure, a tariff and management system that fits the requirements and capacities of the OTC had to be developed. The revenues generated by agricultural reuse of the treated water should, in some way, flow back to subsidize operation and maintenance costs. On the societal side, a community-based approach, modification of hygiene behavior, and capacity development measures for local staff were required to facilitate implementation of the hardware components.

On the technical side, most of the components were implemented. The sanitation facilities were fully operational and well maintained. The vacuum sewer system and wastewater treatment steps functioned reliably. Water quality objectives were met. The energy recovery concept could not be fully implemented. The hydraulic capacity of the vacuum sewers and wastewater treatment plant was not fully utilized due to lower utilization and water use of the connected sanitation facilities.

5.2 Summary of specific findings

The most important (perceived) risks or points of discussion during planning and implementation were (1) the risk of vandalism and misuse of the sanitation facilities, (2) damage from garbage and other foreign objects in the water, (3) loads and concentrations of substances in the untreated water, (4) water quality objectives for agricultural irrigation, (5) measures for management of salts and nutrients, and (6) the energetic utilization of organics contained in wastewater and agricultural residues.

These topics were discussed in detail in this dissertation. The first chapter described the planning process for wastewater collection and transport, and treatment facilities, as well as the rationale for choosing the finally implemented options. The next chapter presented the main monitoring results on the water quantities and quality. It continued with determination of loads and water use for the three types of sanitation facilities and under which conditions recovery of operation and maintenance costs could be achieved. Capital costs were also addressed. The next part focused on the quality of the reclaimed water with emphasis on water quality requirements prior to UV disinfection and for drip irrigation systems, the nutrient requirements of cultivated crops, and the role of the storage pond. Planned and monitored salt and nutrient loads and concentrations were then presented. An assessment of possible management measures and their potential to remove salts and nutrients was carried out. The last part of this dissertation dealt with impediments to the implementation of the energetic concept of the project in Outapi. It focused on the co-digestion of sewage sludge with agricultural residues, planning and monitoring electricity consumption, and the significance of tariff arrangements for the realization of co-generation.

Major constraints for planning were the occurrence of floods during the rainy season and the need for a flexible system. As a consequence, vacuum sewers were chosen for sewage transport because they provide a watertight system that can be extended relatively easily. Despite reservations among Namibian stakeholders, the vacuum sewers worked well after installation. Close coordination between the project partners enabled acceptance of this sewer system and its successful implementation.

During the planning of the sanitation facilities, a major concern was choosing an appropriate system for collection of revenues. The structural design of a shared sanitation facility should consider possible future changes of the billing system and ensure that revenue collection can be carried out as intended.

Vandalism and misuse of the sanitation facilities could be prevented to a large degree via introduction of community health clubs and the demand-responsive approached used by ISOE and DRFN. More information on this topic is provided in Kramm and Deffner (2017) and Deffner and Mazambani (2010).

Garbage and foreign objects contained in the wastewater were mostly retained or crushed. However, stones of the eembe fruit (*Berchemia discolor*) were present in the wastewater, because this fruit is part of the diet of the local population. The stones passed the installed screens, the collection chambers, the vacuum tank, the stone trap and crusher and thus led to a high turnover of spare parts for the installed sewage pumps. Retrofitting was required to ensure removal of these stones to protect the wastewater pumps from damage. Hence, if the wastewater composition is uncertain, enough space should be foreseen for installation of additional equipment to remove problematic solid material.

After implementation, utilization of the sanitation facilities, water quantities, and loads were lower than planned. This affected the sanitation system in several ways. Most obviously, revenue collection from tariffs charged to the users was lower than planned. Thus, less money was available for covering the costs. Less irrigation water was available; thus, additional tap water had to be used to irrigate the agricultural fields. This meant lower revenues for the local farmers because tap water had to be bought from the OTC. In addition, this also meant lower revenues for the OTC that sold the reclaimed water to the farmers. Vice versa, less available irrigation water means, that a smaller area can be cultivated if another water source is not available or affordable. This lowers the revenues from the sale of crops and the amount of residues available for co-digestion. The specific electricity consumption per cubic meter of treated water was relatively high, due to the lower degree of efficiency of motors and pumps because their capacity was not fully used on account of the low hydraulic loading.

During planning, the TDS, TN, TP and K loads and concentrations were expected to be relatively high, and there was a high risk of salinization, overfertilization, and eutrophication when irrigating with the reclaimed water. After implementation, the loads and concentrations were considerably lower. On the whole, the nutrient and salt content of the irrigation water roughly met the requirements of crops and soil.

The lower TCOD content in the untreated water lead to a lower potential not only for biogas production in the UASB reactors but also in the anaerobic digestion unit that was intended to stabilize the sewage sludge. Because the RBCs were dimensioned for removal of higher TCOD loads, nitrogen was partly nitrified in the aerobic treatment step. This consumed most of the alkalinity of the water and led to a higher variability of the pH. The corrosive effluent could be harmful for downstream infrastructures.

In the storage pond, nitrogen losses due to ammonia volatilization were negligible because a considerable proportion of the ammonium nitrogen was oxidized during aerobic treatment. This was an advantage because ammonia gas causes acidification and eutrophication. Considerable ammonia emissions would have evolved during storage without nitrification in the aerobic treatment step. In addition, the volatilized ammonia would not be available for fertilization.

Reliable assumptions regarding loads and water quantities are not only required for correct dimensioning and planning of sewers and wastewater treatment but also for agricultural reuse, e.g., for assessing plant nutrition and soil protection parameters. Thus, the water quantities and loads from the individual and shared sanitation facilities were further examined.

The specific TCOD, TP, TN and TDS loads were determined for each sanitation facility. Substances contained mainly in feces and laundry detergents (TCOD, TP) had higher collection rates than substances found mainly in urine (TN, TDS). It appears that open urination remains widely practiced, even when toilets are provided in the immediate vicinity. More sociological or ethnographic studies are needed to understand the barriers to using toilets.

Tariff levels and billing modes have a strong effect on utilization intensity, specific water use, and the kinds of uses (urination, defecation, laundry washing, showering) and, thus, on the characteristics of the generated wastewater. Water use, collected loads and concentrations vary between the three types of sanitation facilities. Flat rate billing systems lead to relatively high quantities of weak to medium strength wastewater. Volumetric billing leads to relatively low quantities of more concentrated wastewater. The differing strength and volumes of the wastewater flows should be considered when choosing adequate transport and treatment steps. Comparisons with the literature suggest that utilization rates are also influenced by the population density in the area. The higher the population density, the higher the utilization and the higher the amount of collected excreta.

Because municipalities almost always operate under financial constraints, it was further explored under which conditions operation and maintenance costs of the shared sanitation facilities could be covered via tariffs. Operation and maintenance cost recovery could not be achieved for the solutions implemented in Outapi. Additional funding was required for the communal washhouse and the cluster units. Because the population density influences utilization, it also influences the amount of collected revenues. In areas with a low population density, utilization will be relatively low and collected revenues will be too low to cover the operation and maintenance costs. Under such conditions, caretaking and cleaning activities could be transferred to the users of the sanitation facilities, to reduce staff costs. However, in more densely populated areas, the costs of shared sanitation facilities could be covered entirely by the levied tariffs.

The project was implemented in a setting in which national regulations for reclaimed water quality did not yet exist. Thus, water quality criteria and objectives had to be defined. The FAO and WHO guidelines were used for monitoring irrigation water quality. In order to meet the requirements of water reuse projects, additional water quality objectives for turbidity, BOD_5, TCOD, TP, and K were suggested in this dissertation. For instance, anaerobic digestion in combination with (unintended) nitrification during aerobic treatment led to a relatively low and highly variable pH. This should be considered when reclaiming nutrient-rich water with low alkalinity. The storage pond was initially included in the sanitation system to balance supply and demand of the irrigation water. However, it turned out to be necessary to achieve the required water quality.

The risks of salinization and overfertilization were also assessed because they were regarded as major threats to sustainable agricultural irrigation at the planning stage. If there is no natural or artificial leaching of the agricultural fields, considerable amounts of TDS will accumulate there in the long term, in any case. Thus, measures for salt management have to be included. If excreta collection among the connected population is more or less complete or achieved to a high degree, the salt content of the water will only allow cultivation of crops with a certain salt tolerance, if measures for salinity reduction are not foreseen. Salinity needs to be reduced, already prior to water application in the fields, to allow irrigation of salt-sensitive crops. This could be achieved via dilution of the water or measures for salt reduction prior to or during wastewater treatment.

Because none of the salt reduction measures prior to and during wastewater treatment, or irrigation management practices, can completely prevent long-term accumulation of salts on the fields, a drainage system was implemented on the irrigation site in Outapi. Hence, most of the salt and nutrient loads are discharged via drainage water that is collected in an evaporation pond that acts as the final sink for these substances. A considerable proportion of the nutrients (up to 70% in the case of the applied K load) remains unused. More water would be needed to make use of these nutrients.

The last part of this dissertation dealt with energetic aspects that arose during planning and implementation. The obtainable methane yields and electricity generation from agricultural residuals were reviewed. Most of the producible electricity originates from the biogas potential of sewage sludge. By calculation, the electricity demand of vacuum sewers and wastewater treatment could be covered to a high degree if the whole agricultural area of 3 ha was to be cultivated with crops that produce large amounts of residues, such as maize or wheat. On smaller areas, cultivation of energy crops and utilization of the whole plant for biogas production could achieve calculated energy self-sufficiency. Minimum water quantities of 66 m^3/d to 140 m^3/d are required to irrigate an area that is sufficient for the production of crops or crop residues whose energy content is sufficient to produce the electricity required to operate the sewers and plant in Outapi (when co-digested with sewage sludge).

The energy recovery concept from sewage sludge and crop residues was difficult to implement, due to the low available quantities of organic matter contained in the collected sewage and in agricultural residues, the effort for transport, handling, and processing of the biomass, and the relatively high monetary value of vegetables on the local markets, compared to electricity.

Another impediment was the tariff structure of the local electricity supply entity. It was not possible to receive a rebate or credit for the electricity fed to the grid. Fixed costs also constituted a major part of the electricity costs. Thus, for the given tariff structure, co-generation can only reduce electricity costs if the produced electricity is immediately consumed to operate the vacuum sewers and wastewater treatment plant and if it can contribute to a reduction of the peak power demand.

Altogether, this work addressed the lack of knowledge regarding inherent uncertainties and risks as one of the main obstacles for implementing new approaches to sanitation systems in developing countries. It outlined the problems encountered during implementation of this pilot project in North Namibia and presented new results that provide a better understanding of the crucial points for planning and implementation and how the system components are connected.

This study presented the planning approach, the quantitative and qualitative data regarding water use, TCOD, TN, TP and TDS loads from individual and shared sanitation facilities, explored the potentials and limitations for recovery of operation and maintenance costs of shared sanitation facilities, discussed capital costs, provided water quality criteria and objectives for water reuse, clarified the potential of measures for salt and nutrient management, provided key figures regarding the electricity consumption of the presented infrastructure and highlighted the impediments to implementation of co-generation in the project region.

5.3 Application of the research

The dissertation was prepared within the framework of a project that considered the whole sanitation chain, including provision of tap water, toilets, showers and laundry washing facilities, as well as wastewater transport, treatment, reclamation and irrigation infrastructures. The technical implementation was completed by the provision of organizational and tariff structures needed for operation.

For the first time, a sanitation system has been analyzed from a holistic perspective providing detailed specifications on planning and monitoring data and influencing factors. This is a sound basis for better implementation of similar projects in the future. The knowledge gaps that caused misconceptions and challenges during implementation can now be avoided or at least be realistically assessed right from the outset.

The results in this dissertation are based on a practical project and not exclusively on theoretical considerations. They were elaborated within the framework of a project that is a tailor-made response to the sanitation challenges in North Namibia, but can also be transferred to other water-scarce, urban and peri-urban areas. The unexpected developments in this project could also occur elsewhere, although not necessarily in the same way or with solutions that are the same as in this case. An important feature of this work is that it identified the key points that need to be considered to avoid unpleasant surprises. Aspects that were not considered, initially, later turned out to be important. The presented project is a solution prepared by humans for humans and, hence, the outcome is not fully predictable. But for every point addressed in the following paragraphs, a substantial amount of information is available in the respective chapters of this dissertation that provides a sound foundation for making the outcome as predictable as possible. The following recommendations are made for the wider sanitation sector.

The detailed documentation and discussion of the overall approach for planning and implementation of wastewater collection, transport, and treatment steps reveal what needs to be considered during planning, what information is required, and the consequences of not following these recommendations.

When planning shared sanitation facilities, the future management and billing scheme need to be set as starting points. The physical layout has to match to the intended operation. An important point is the population density. It influences utilization and, hence, water use, collected loads and collected revenues. The reflections on potential and observed utilization rates at the communal washhouse made in this study, provide an approach that can be used to quantify the future utilization of shared sanitation facilities.

The figures on the specific water use and the specific loads allow a better prediction of the wastewater characteristics of shared and individual sanitation facilities in informal settlements. Altogether, the available literature data are sufficient to estimate the total per capita loads. The information provided here contributes to better estimate to what extent the loads are collectable.

An unexpected finding was that the smaller cluster units were cheaper, in terms of specific capital and operation and maintenance costs, than the larger communal washhouse. One would expect that the specific costs are lower for the communal washhouse because they are spread over a larger number of users. But even when considering optimum utilization, the cluster units were cheaper. This might be different in other locations with different cost structures. It is recommended to assess both possibilities thoroughly and not to suppose that economies of scale effects apply.

The objective of excreta collection conflicts with the objective to provide sanitation services at low cost. The smaller cluster units in this study were much cheaper in terms of capital and operation and maintenance costs, but less efficient in excreta collection. The larger communal washhouse was more efficient in excreta collection, but more expensive. This can be different in other locations. In other cities, cluster units might perform better regarding excreta collection or construction of a communal washhouse may be cheaper. For a cost situation comparable to the one in Outapi, the responsible authorities should set their priorities before planning and implementation. This means that decisions are required about whether the objective is to provide at least some service at minimum cost, or whether it is feasible to subsidize a possibly more expensive larger shared sanitation facility to provide water and sanitation services that collect a higher percentage of excreta.

This work suggests that the lower population density and tariff levels are reasons for the lower utilization of the shared sanitation facilities compared to planning data. However, other factors may also be relevant. Another finding is that open defecation remained widely practiced, even though toilets were kept clean and were freely accessible. In a South African example provided in the literature, toilets were also not used, even though they were available free of charge. Hence, there are barriers to toilet use that are not connected to accessibility and cleanliness.

These examples demonstrate the practical benefit that sociological or ethnographic studies can provide. The outcome of such studies would be useful to define measures to increase utilization and excreta collection. It is therefore recommended to involve social scientists to make sure that the provided infrastructure is fully used.

This dissertation also closes knowledge gaps in the field of water quality objectives for irrigation. It demonstrates how the available guidelines and further information from the literature can be used to define specific water quality objectives. The available knowledge is combined in a way that facilitates implementation of water reuse projects. The presented approach is useful for any setting in which national regulations are not available.

The salt and nutrient loads and concentrations were perceived as a very serious problem during planning. After implementation, the salt and nutrient content of the water was much lower than anticipated. This made implementation of sustainable agricultural irrigation much easier. However, this may not be the case elsewhere. Hence, the input data and calculations described in detail in this dissertation can be used as a basis for assessing salt and nutrient levels in all flows of sanitation systems. Suitable management measures can be selected with this approach.

Energetic utilization of organics contained in the wastewater and co-digestion with agricultural residues is only possible with suitable organizational frameworks. If the local electricity supply entity does not allow credit for electricity fed to the grid, costs and benefits need to be reconsidered.

5.4 Future work and outlook

Future work in Outapi should follow up on achieving higher hydraulic loadings by connecting additional households to the sanitation system. In this way, treatment would be more efficient regarding the resources used and effort made for treating the water and more water would be available for irrigation. Thus, revenues would increase for the OTC and for the farmers. Components that are not required should be decommissioned, to save resources.

The theoretical considerations on the fate and behavior of salts and nutrients should be backed up with more measurements. Long-term monitoring on the build-up of salts in the soil is required. The TDS, TN, TP and K contents and quantities of sewage sludge, algae, crops and drainage water should be quantified for comparison with the calculated data.

Further measurements on the specific water use and loads from the sanitation facilities should be carried out to determine whether the values have changed. Because achieving higher utilization rates is crucial for financial sustainability of the sanitation system, the sanitation facilities need further promotion among the residents. Sociological reasons for low utilization should be further studied. The examination of utilization rates and wastewater characteristics could also be carried out at other shared sanitation facilities. Data on the population density and socio-economic characteristics of the local residents could be compared to the findings in Outapi and be used to elaborate a better understanding of what influences the utilization, water use, and excreta collection of shared sanitation.

The idea of a tariff for consumption and generation of electricity at the same metering point should be discussed among Namibian stakeholders and, most importantly, among the Namibian electricity supply entities.

On a broader scale, more projects like the one in Outapi are needed to further promote the idea of holistic and systemic planning and implementation and to overcome the shortcomings of traditional water management approaches. The experiences and findings of this dissertation should be validated by the results from other projects. Obstacles to successful implementation could be prevented or avoided. Only by putting the theoretical framework into practice can success be achieved in closing the knowledge gaps that act as barriers to the realization of new approaches to water supply and sanitation and in reaching international development goals.

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7 Appendix

Table 47 Design data and process parameters of the wastewater and sludge treatment in Outapi (BWT 2013, 2011)

parameter	value	unit
UASB reactors		
volume	72	m³
upflow velocity water (max.)	0.7	m/h
hydraulic retention time (min.)	6.4	h
sludge retention time (min.)	6.5	d
space loading	1.54	kg COD/(m³×d)
sludge loading	5.2	kg COD/(kg VSS×d)
COD reduction	50	%
sludge production	≈ 100	kgTS/d
dry solids concentration in reactor (average)	1-5	%
biogas production	41	m³/d
methane production	27	m³/d
RBC and lamella clarifier		
diameter disks	2,000	mm
disk material	PP	
dimensions	2,000 x 2,000 x 5,000-10,000	mm
volume RBC	15	m³
volume lamella clarifier	3	m³
removal	carbon removal	
surface load	≈ 13	g BOD/(m²×d)
space load	1.3	kg BOD/(m³×d)
disk surface per unit	3,300	m²
surplus sludge production	≈ 41	kgTS/d
surface load lamella clarifier	1	m/h
microscreen		
flux	6	L/(m²×s)
mesh size	15	µm
volume	3	m³
dry solids content in outlet	< 10	mg/L
active sieve fabric surface	≈ 0.5	m²
matrial sieve fabric	Monodur Polyamid	
dimensions drum	500 x 500	mm
anaerobic digester		
input from RBC + UASB per day	≈ 60	kg VSS/d
volume	2.1	m³/d
dry solids input	44	kg/m³
input from maize silage per day	≈ 20	kg VSS/d
volume	0.06	m³/d
dry solids input	400	kg/m³
hydraulic retention time	20	d
volume anaerobic digester	45	m³
capacity recirculation pump	2-10	m³/h

heat-exchanger type	tube-in-tube, external	
heating capacity (max.)	≈ 7.5	kW thermal
horizontal mixer, length	≈ 9	m
diameter	≈ 2.2	M
drive power	≈ 2.2	kW
biogas and energy		
output Biogas from anaerobic digester	≈ 110	Nm³/d
thermal energy output	≈ 410	kWh/d
electrical energy output	≈ 205	kWh/d
biogas storage capacity	100	Nm³/d

Table 48 Assignment of power measurements (provided by Bilfinger Water Technologies)

designation	**power (kW)**	**CHP**	**biogas**	**miscellaneous**	**hea-ting**	**sewage pumps**	**RBC**	**sludge**	**vacuum pumps**
combined heat and power (CHP)	18	X							
biogas cooler	1.5		X						
biogas booster blower	0.25		X						
air blower biogas storage	0.37		X						
exhaust blower engine room biogas container	0.81		X						
exhaust blower gas room biogas container	0.32		X						
exhaust blower electric room biogas container	0.14		X						
drum motor micro screen	1.1			X					
sludge liquor pump	1.9			X					
sludge liquor pump	1.9			X					
booster pumping station	3			X					
UV disinfection	1.1			X (since March 2014)					
heating water pump UASB pre-heating	0.18				X				
heating water pump anaerobic digester heating	0.18				X				
heating water pump 1/2 solar panels feed	0.09				X				
chopper (sewage)	4					X			
feed pump 1 UASB	3					X			
feed pump 2 UASB	3					X			
feed pump 3 UASB	3					X			
control air compressor vacuum station	1.7					X (1/3)	X (1/3)	X (1/3)	
rotating disk drive RBC 1	2.2						X		
rotating disk drive RBC 2	2.2						X		
feed conveyor anaerobic digester	0.75							X	
discharge conveyor anaerobic digester	0.75							X	
circulation pump 1 anaerobic digester	3							X	
circulation pump 2 anaerobic digester	3							X	
horizontal agitator anaerobic digester	2.5							X	
control air compressor anaerobic digester	1.7							X	
exhaust ventilator pump room ventilator	0.15							X	
vacuum pump 1	5.5								X
vacuum pump 2	5.5								X

Table 49 Methane yields of a variety of plant types (part I)

crop	methane yield (m³/kg VS added)	source	mean	median
spinach	0.314	Knol *et al.* (1978)	-	-
cabbage	0.382	Zubr (1986)	0.32	0.32
	0.343	Zubr (1986)		
	0.309	Gunaseelan (2004)		
	0.291	Gunaseelan (2004)		
	0.277	Cho and Park (1995)		
	0.336	KTBL (2007)		
potatoes, stems and leaves	0.495	Deublein and Steinhauser (2011)	0.36	0.36
	0.110	Vlyssides *et al.* (2015)		
	0.606	Reinhold and Noack (1956)		
	0.229	Keymer (2016)		
potatoes, pulp	0.426	Stewart *et al.* (1984), cited in Gunaseelan (1997)	0.38	0.38
	0.335	Labatut *et al.* (2011)		
	0.366	Parawira *et al.* (2008)		
	0.411	Döhler (2005)		
	0.380	KTBL (2007)		
wheat straw	0.313	Hashimoto (1989)	0.28	0.31
	0.189	Amon *et al.* (2007)		
	0.154	Döhler (2005)		
	0.362	Sharma *et al.* (1988)		
	0.302	Tong *et al.* (1990)		
	0.333	Tong *et al.* (1990)		
	0.189	KTBL (2007)		
	0.367	Reinhold and Noack (1956)		
maize straw	0.351	Deublein and Steinhauser (2011)	0.31	0.33
	0.360	Tong *et al.* (1990)		
	0.214	Menardo and Balsari (2012)		
	0.317	Dinuccio *et al.* (2010)		
maize, whole plant	0.342	Badger *et al.* (1979), cited in Gunaseelan (1997)	0.37	0.39
	0.324	Deublein and Steinhauser (2011)		
	0.253	Stewart *et al.* (1984), cited in Gunaseelan (1997)		
	0.315	Weiland (2010)		
	0.390	Amon *et al.* (2007)		
	0.390	Oechsner *et al.* (2003)		
	0.400	Oechsner *et al.* (2003)		
	0.390	Oechsner *et al.* (2003)		
	0.400	Oechsner *et al.* (2003)		
	0.400	Oechsner *et al.* (2003)		
	0.370	Oechsner *et al.* (2003)		
	0.400	Oechsner *et al.* (2003)		
	0.390	Oechsner *et al.* (2003)		
	0.348	KTBL (2015)		

Table 50 Methane yields of a variety of plant types (part II)

crop	methane yield (m³/kg VS added)	source	mean	median
peppers, stems and leaves	0.368	Rhee *et al.* (2012)	0.35	0.35
	0.330	Rhee *et al.* (2012)		
beans, stems and leaves	0.265	Petersson *et al.* (2007)	0.28	0.27
	0.387	Pakarinen *et al.* (2011)		
	0.174	Lopez-Davila *et al.* (2012)		
	0.277	Keymer (2016)		
sugarbeet leaves	0.297	Badger *et al.* (1979), cited in Gunaseelan (1997)	0.33	0.34
	0.210	Amon *et al.* (2007)		
	0.340	Lehtomäki *et al.* (2008)		
	0.360	Zubr (1986)		
	0.381	Zubr (1986)		
	0.231	Gunaseelan (2004)		
	0.501	Reinhold and Noack (1956)		
	0.342	Keymer (2016)		
leaves: trees and shrubs	0.294	Deublein and Steinhauser (2011)	0.15	0.11
	0.123	Owens and Chynoweth (1993)		
	0.101	Döhler (2005)		
	0.098	Thomé-Kozmiensky (1995) and Brachtl (1998), cited in Ileleji *et al.* (2008)		
cauliflower leaves	0.520	Sharma *et al.* (1988)	0.35	0.34
	0.190	Gunaseelan (2004)		
	0.331	Gunaseelan (2004)		
	0.352	Zubr (1986)		
	0.341	Zubr (1986)		
Jerusalem artichoke	0.365	Lehtomäki *et al.* (2008)	0.31	0.30
	0.250	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.265	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.307	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.281	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.338	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.354	Gunnarson *et al.* (1985), cited in Gunaseelan (1997)		
	0.309	Zubr (1986)		
	0.301	Zubr (1986)		
rhubarb	0.405	Lehtomäki *et al.* (2008)	0.36	0.35
	0.316	Zubr (1986)		
	0.345	Zubr (1986)		

Table 51 Methane yields of a variety of plant types (part III)

crop	methane yield (m^3/kg VS added)	source	mean	median
other leafy crops	0.315	Lehtomäki *et al.* (2008)	0.29	0.31
	0.314	Gunaseelan (2004)		
	0.220	Lehtomäki *et al.* (2008)		
	0.315	Lehtomäki *et al.* (2008)		
grass	0.405	Deublein and Steinhauser (2011)	0.31	0.31
	0.270	Deublein and Steinhauser (2011)		
	0.305	Weiland (2010)		
	0.209	Owens and Chynoweth (1993)		
	0.315	KTBL (2015)		
	0.370	Lehtomäki *et al.* (2008)		
	0.300	Lehtomäki *et al.* (2008)		
	0.324	KTBL (2007)		

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*WA*R 68	Bewertung von Geruchsemissionen und –immissionen. 29. Darmstädter Seminar -Immissionsschutz- am 08.10.1992, TH Darmstadt, 1993	25,60 €
*WA*R 69	Möglichkeiten und Grenzen der Klärschlammentsorgung. 3. gemeinsames Seminar -Abwassertechnik- mit der Hochschule für Architektur und Bauwesen Weimar am 31.03. und 01.04.1993, TH Darmstadt, 1993	46,-- €
*WA*R 70	Sichere Wasserversorgung durch moderne Rohrleitungstechnik. 33. Darmstädter Seminar -Wasserversorgungstechnik- am 11.03.1993, TH Darmstadt, 1993	30,70 €

*WA*R 71	Aktuelle Aufgaben der Abwasserreinigung und Schlammbehandlung. 35. Darmstädter Seminar -Abwassertechnik- am 05. + 06.05.1993, TH Darmstadt, 1993	46,-- €
*WA*R 72	Raumordnungsverfahren mit Umweltverträglichkeitsprüfung und Umweltleitbilder für die Landes- und Regionalplanung. 28. und 30. Darmstädter Seminar -Raumplanung- am 17.09. und 05.11.1992, TH Darmstadt, 1993	40,90 €
*WA*R 73	Grohmann, Walter: Vergleichende Untersuchungen von Belüftungs- und Durchmischungssystemen zur bioverfahrenstechnischen Optimierung der aerob-thermophilen Stabilisation (ATS). Dissertation, FB 13, TH Darmstadt, 1993	35,80 €
*WA*R 74	Dioxinimmissionen und Quellen . 34. Darmstädter Seminar -Immissionsschutz- am 15.04.1993, TH Darmstadt, 1994	30,70 €
*WA*R 75	Betrieb von Abwasserbehandlungsanlagen Optimierung, Prozeßstabilität, Kosteneinsparung. 36. Darmstädter Seminar -Abwassertechnik- am 04.11.1993 in Darmstadt und 5. gemeinsames Seminar -Abwassertechnik- mit der Fakultät Bauingenieurwesen der Hochschule für Architektur und Bauwesen Weimar am 23. und 24.03.1994 in Weimar, TH Darmstadt, 1994	46,-- €
*WA*R 76	Umweltgerechte Ausweisung und Erschließung von Gewerbegebieten. 4. gemeinsames Seminar -Umwelt- und Raumplanung- mit der Fakultät Architektur, Stadt- und Regionalplanung der Hochschule für Architektur und Bauwesen Weimar am 08. und 09.09.1993 in Weimar, TH Darmstadt, 1994	vergriffen
*WA*R 77	Von der Umweltverträglichkeitsprüfung zum kooperativen Planungsmanagement. Das Scoping-Verfahren als erste Stufe!? 37. Darmstädter Seminar -Umwelt- und Raumplanung- am 11.11.1993, TH Darmstadt, 1994	vergriffen
*WA*R 78	Modellbildung und intelligente Steuerungssysteme in der Umwelttechnik. 38. Darmstädter Seminar -Abfalltechnik- am 24.02.1994, TH Darmstadt, 1994	25,60 €
*WA*R 79	Brauchwassernutzung in Haushalten und Gewerbebetrieben - Ein Gebot der Stunde? 39. Darmstädter Seminar -Wasserversorgungstechnik- am 17.03.1994, TH Darmstadt, 1994	25,60 €

*WA*R 80 Restabfallbehandlung in Hessen. — vergriffen
41. Darmstädter Seminar -Abfalltechnik- mit dem Hessischen Ministerium für Umwelt, Energie und Bundesangelegenheiten -HMUEB- am 16.06.1994,
TH Darmstadt, 1994

*WA*R 81 Umweltbeeinflussung durch biologische Abfallbehandlungsverfahren. — 46,-- €
42. Darmstädter Seminar -Abfalltechnik- mit dem Institut für Hygiene der FU Berlin und dem Institut für Meteorologie der TH Darmstadt am 08. und 09.09.1994 in Berlin,
TH Darmstadt, 1994

*WA*R 82 Zeitgemäße Planung von Anlagen der Ortsentwässerung - Kanäle, Bauwerke, Sonderbauwerke. — vergriffen
6. gemeinsames Seminar -Abwassertechnik- mit der Fakultät Bauingenieurwesen der Hochschule für Architektur und Bauwesen Weimar am 15. und 16.03.1995 in Weimar,
TH Darmstadt, 1995

*WA*R 83 Grundwasseranreicherung - Stand der Technik und neuere Entwicklungen. — 30,70 €
44. Darmstädter Seminar -Wasserversorgungstechnik- mit dem Verein des Gas- und Wasserfaches e.V. -DVGW- am 26.04.1994,
TH Darmstadt, 1995

*WA*R 84 Auswirkungen der Phosphorelimination auf die Schlammbehandlung. — 30,70 €
Theoretische Erkenntnisse und praktische Erfahrungen.
Workshop vom 24. bis 25. November 1994,
TH Darmstadt, 1995

*WA*R 85 Stickstoffelimination mit oder ohne externe Substrate ? — 35,80 €
- Erfahrungen und Überlegungen.
43. Darmstädter Seminar -Abwassertechnik- in Abstimmung mit der Abwassertechnischen Vereinigung e.V. (ATV) am 09.11.1994,
TH Darmstadt, 1995

*WA*R 85 Stickstoffelimination mit oder ohne externe Substrate ? — 35,80 €
- Erfahrungen und Überlegungen. 2. Auflage.
Wiederholung des 43. Darmstädter Seminars -Abwassertechnik- in Abstimmung mit der Abwassertechnischen Vereinigung e.V. (ATV) am 01.02.1996 in Düsseldorf,
TH Darmstadt, 1996

*WA*R 86 Möglichkeiten und Grenzen der Einsparung von Investitions- und Betriebskosten bei der Abwasserbehandlung. — 40,90 €
47. Darmstädter Seminar -Abwassertechnik- am 15.11.1995,
TH Darmstadt, 1995

*WA*R 87	Jardin, Norbert: Untersuchungen zum Einfluß der erhöhten biologischen Phosphor-elimination auf die Phosphordynamik bei der Schlammbehandlung. Dissertation, FB 13, TH Darmstadt, 1996	35,80 €
*WA*R 88	Thermische Restabfallbehandlung für kleine Planungsräume. 45. Darmstädter Seminar -Abfalltechnik- am 22.06.1995 in Hanau, TH Darmstadt, 1996	35,80 €
*WA*R 89	Ferber, Uwe: Aufbereitung und Revitalisierung industrieller Brachflächen in den traditionellen Industrieregionen Europas. Sonderprogramme im Vergleich. Dissertation, FB 13, TH Darmstadt 1996	25,60 €
*WA*R 90	Mechanisch-biologische Restabfallbehandlung unter Einbindung thermischer Verfahren für Teilfraktionen. 48. Darmstädter Seminar -Abfalltechnik- am 29.02.1996, TH Darmstadt, 1996	vergriffen
*WA*R 91	Neuere Erkenntnisse bei Planung, Bau, Ausrüstung und Betrieb von Abwasserbehandlungsanlagen. 7. gemeinsames Seminar -Abwassertechnik- mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar am 11. und 12.09.1996 in Weimar, TH Darmstadt, 1996	40,90 €
*WA*R 92	Hygiene in der Abfallwirtschaft. 50. Darmstädter Seminar -Abfalltechnik- am 17.10.1996, TH Darmstadt, 1996	30,70 €
*WA*R 93	Europäische Richtlinien und Normen zur Abwassertechnik - Konsequenzen und Folgerungen für die Praxis in Deutschland. 51. Darmstädter Seminar -Abwassertechnik- am 14.11.1996, TH Darmstadt, 1996	25,60 €
*WA*R 94	Dickhaut, Wolfgang: Möglichkeiten und Grenzen der Erarbeitung von Umwelt-qualitätszielkonzepten in kooperativen Planungsprozessen. Durchführung und Evaluierung von Projekten. Dissertation, FB 13, TH Darmstadt 1996	30,70 €
*WA*R 95	Lautner, Gerd: Einführung in das Bauordnungsrecht. 8. erw. und aktual. Auflage, TH Darmstadt, 1997	15,40 €
*WA*R 96	Reichert, Joachim: Bilanzierung des Sauerstoffeintrags und des Sauerstoffverbrauchs mit Hilfe der Abluftmethode. Dissertation, FB 13, TH Darmstadt 1997	46,-- €

*WA*R 97	Kuchta, Kerstin: Produktion von Qalitätsgütern in der Abfallbehandlung. Dargestellt am Beispiel der Produktion in der thermischen Abfallbehandlung. Dissertation, FB 13, TH Darmstadt 1997	30,70 €
*WA*R 98	Görg, Horst: Entwicklung eines Prognosemodells für Bauabfälle als Baustein von Stoffstrombetrachtungen zur Kreislaufwirtschaft im Bauwesen. Dissertation, FB 13, TH Darmstadt, 1997	46,-- €
*WA*R 99	Tiebel-Pahlke, Christoph: Abfallentsorgungsplanung – Beeinflussung der Umweltauswirkungen von Deponien. Dissertation, FB 13, TH Darmstadt, 1997	30,70 €
*WA*R 100	Wagner, Martin: Sauerstoffeintrag und Sauerstoffertrag von Belüftungssystemen und deren Bestimmung mit modernen Meßmethoden. Habilitation, FB 13, TH Darmstadt, 1997	vergriffen
*WA*R 101	Neue Trends bei der Behandlung und Entsorgung kommunaler und industrieller Klärschlämme. 8. gemeinsames Seminar -Abwassertechnik- mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar am 10. und 11.09.1997 in Weimar, TH Darmstadt, 1997	35,80 €
*WA*R 102	Senkung der Betriebskosten von Abwasserbehandlungsanlagen. 52. Darmstädter Seminar -Abwassertechnik- am 06.11.1997 in Darmstadt, TU Darmstadt, 1997	35,80 €
*WA*R 103	Sanierung und Rückbau von Bohrungen, Brunnen und Grundwasser-messstellen. 53. Darmstädter Seminar -Wasserversorgung- am 13.11.1997 in Darmstadt mit dem Deutschen Verein des Gas- und Wasserfaches e.V. (DVGW), TU Darmstadt, 1997	vergriffen
*WA*R 104	Wünschmann, Gabriele: Untersuchungen zur Kompostierbarkeit von Reststoffen der Papier-industrie und Altpapier unter besonderer Berücksichtigung von Schad-stoffbilanzierungen. Dissertation, FB 13, TU Darmstadt, 1997	25,60 €

*WA*R 105	Mechanisch-biologische Restabfallbehandlung unter Einbindung thermischer Verfahren für Teilfraktionen. 54. Darmstädter Seminar -Abfalltechnik- am 06.02.1998 in Darmstadt mit dem Hessischen Ministerium für Umwelt, Energie, Jugend, Familie und Gesundheit und der Südhessischen Arbeitsgemeinschaft Abfall-wirtschaft (SAGA), TU Darmstadt, 1998	40,90 €
*WA*R 106	Zentrale oder dezentrale Enthärtung von Trinkwasser – Konkurrenz oder sinnvolle Ergänzung ? 55. Darmstädter Seminar -Wasserversorgung- am 14.05.1998 in Darmstadt mit dem Deutschen Verein des Gas- und Wasserfaches e.V. (DVGW), TU Darmstadt, 1998	35,80 €
*WA*R 107	Dach, Joachim: Zur Deponiegas- und Temperaturentwicklung in Deponien mit Siedlungsabfällen nach mechanisch-biologischer Abfallbehandlung. Dissertation, FB 13, TU Darmstadt, 1998	35,80 €
*WA*R 108	Einsparung von Kosten für Betriebsmittel, Energie und Personal auf Abwasserbehandlungsanlagen. 9. gemeinsames Seminar -Abwassertechnik- am 16. und 17.09.1998 in Weimar mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar, TU Darmstadt, 1998	40,90 €
*WA*R 109	Fortschritte in der Abwassertechnik – 15 Jahre Forschungs- und Entwicklungstätigkeit von Prof. Dr.-Ing. H. Johannes Pöpel. 56. Darmstädter Seminar -Abwassertechnik- am 05.11.1998 in Darmstadt, TU Darmstadt, 1998	40,90 €
*WA*R 110	Qualitativer und Quantitativer Grundwasserschutz - Stand und Perspektiven. 57. Darmstädter Seminar -Wasserversorgung- am 10.06.1999 in Darmstadt mit dem Deutschen Verein des Gas- und Wasserfaches e.V. (DVGW), TU Darmstadt, 1999	35,80 €
*WA*R 111	Schwing, Elke: Bewertung der Emissionen der Kombination mechanisch-biologischer und thermischer Abfallbehandlungsverfahren in Südhessen. Dissertation, FB 13, TU Darmstadt, 1999	30,70 €
*WA*R 112	Schade, Bernd: Kostenplanung zur Analyse der Wirtschaftlichkeit von biologischen Restabfallbehandlungsanlagen. Dissertation, FB 13, TU Darmstadt, 1999	30,70 €

*WA*R 113	Lohf, Astrid: Modellierung der chemisch-physikalischen Vorgänge im Müllbett von Rostfeuerungsanlagen. Dissertation, FB 13, TU Darmstadt, 1999	25,60 €
*WA*R 114	Stackelberg, Daniel von: Biologische Festbettdenitrifikation von Grundwasser mit abbaubarem Trägermaterial. Dissertation, FB 13, TU Darmstadt, 1999	30,70 €
*WA*R 115	Folgerungen aus 10 Jahren Abwasserbeseitigung in den neuen Bundesländern - Erfahrungen und Perspektiven. 10. gemeinsames Seminar –Abwassertechnik- am 01. und 02.09.1999 in Weimar mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar, TU Darmstadt, 1999	40,90 €
*WA*R 116	Abwasserwiederverwendung in wasserarmen Regionen - Einsatzgebiete, Anforderungen, Lösungsmöglichkeiten. 58. Darmstädter Seminar –Abwassertechnik- am 11.11.1999 in Darmstadt, TU Darmstadt, 1999	vergriffen
*WA*R 117	Reinhardt, Tim: Untersuchungen zur Dynamik biologischer Prozesse in drei-Phasen-Systemen am Beispiel der Restabfallrotte unter besonderer Berücksichtigung anaerober Teilprozesse. Dissertation, FB 13, TU Darmstadt, 1999	30,70 €
*WA*R 118	Umweltfachpläne und Umweltgesetzbuch - Ein Beitrag zur Fortentwicklung des Umweltfachplanungssystems und „Von der Landschaftsplanung zur Umweltleitplanung?“ 46. Darmstädter Seminar -Umwelt- und Raumplanung- am 28.09.1995 in Darmstadt, TU Darmstadt, 1999	30,70 €
*WA*R 119	Herr, Christian: Innovative Analyse und primärseitige Prozeßführungsoptimierung thermischer Abfallbehandlungsprozesse - am Beispiel der Mülleingangsklassifizierung bei der Rostfeuerung. Dissertation, FB 13, TU Darmstadt, 2000	33,20 €
*WA*R 120	Neumüller, Jürgen: Wirksamkeit von Grundwasserabgaben für den Grundwasserschutz - am Beispiel des Bundeslandes Hessen. Dissertation, FB 13, TU Darmstadt, 2000	35,80 €
*WA*R 121	Hunklinger, Ralph: Abfalltechnische Kennzahlen zur umweltgerechten Produktentwicklung. Dissertation, FB 13, TU Darmstadt, 2000	30,70 €

*WA*R 122	Wie zukunftsfähig sind kleinere Wasserversorgungsunternehmen? 60. Darmstädter Seminar -Wasserversorgung- am 29. Juni 2000 in Darmstadt, TU Darmstadt, 2000	35,80 €
*WA*R 123	Maßnahmen zur Betriebsoptimierung von Pumpwerken, Kanalisationssystemen und Abwasserbehandlungsanlagen. 11. gemeinsames Seminar -Abwassertechnik- in Weimar am 20. und 21. September 2000 mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar, TU Darmstadt, FB 13, 2000	40,90 €
*WA*R 124	Mohr, Karin: Entwicklung einer on-line Emissionsmeßtechnik zur quasi-kontinuierlichen Bestimmung von Organohalogen-Verbindungen in Abgasen thermischer Prozesse. Dissertation, FB 13, TU Darmstadt, 2000	30,70 €
*WA*R 125	El-Labani, Mamoun: Optimierte Nutzung bestehender Abfallverbrennungsanlagen durch Errichtung vorgeschalteter Reaktoren zur Behandlung heizwertreicher Abfälle. Dissertation, FB 13, TU Darmstadt, 2000	25,60 €
*WA*R 126	Durth, Anke: Einfluß von Temperatur, Anlagenkonfiguration und Auslastung auf die Ablaufkonzentration bei der biologischen Abwasserreinigung. Dissertation, FB 13, TU Darmstadt, 2000	vergriffen
*WA*R 127	Meyer, Ulrich: Untersuchungen zum Einsatz von Fuzzy-Control zur Optimierung der Stickstoffelimination in Abwasserbehandlungsanlagen mit vorgeschalteter Denitrifikation. Dissertation, FB 13, TU Darmstadt, 2000	33,20 €
*WA*R 128	Kommunale Klärschlammbehandlung vor dem Hintergrund der neuen europäischen Klärschlammrichtlinie. 61. Darmstädter Seminar -Abwassertechnik- am 09.11.2000 in Darmstadt, TU Darmstadt, FB 13, 2000	35,80 €
*WA*R 129	Mengel, Andreas: Stringenz und Nachvollziehbarkeit in der fachbezogenen Umweltplanung. Dissertation, FB 13, TU Darmstadt, 2001	46,-- €

*WA*R 130	Kosteneinsparungen durch neuartige Automatisierungstechniken in der Wasserversorgung. 62. Darmstädter Seminar -Wasserversorgung- am 07.06.2001 in Darmstadt, TU Darmstadt, FB 13, 2001	30,70 €
*WA*R 131	Aktive Zukunftsgestaltung durch Umwelt- und Raumplanung. Festschrift zum 60. Geburtstag von Prof. Dr.-Ing. Hans Reiner Böhm. TU Darmstadt, FB 13, 2001	25,60 €
*WA*R 132	Aktuelle Ansätze bei der Klärschlammbehandlung und -entsorgung. 12. gemeinsames Seminar -Abwassertechnik- in Weimar am 05. und 06. September 2001 mit der Fakultät Bauingenieurwesen der Bauhaus-Universität Weimar, TU Darmstadt, FB 13, 2001	40,90 €
*WA*R 133	Zum Bodenwasser- und Stoffhaushalt auf unterschiedlich bewirtschafteten Flächen unter Einbeziehung ökonomischer Aspekte Interdisziplinäre Projektstudie der Technischen Universität Darmstadt (TUD) mit Partner. TU Darmstadt, FB 13, 2001	30,70 €
*WA*R 134	Neues zur Belüftungstechnik - Probleme, Lösungsmöglichkeiten, Entwicklungen. 64. Darmstädter Seminar -Abwassertechnik- am 15.11.2001 in Darmstadt, TU Darmstadt, FB 13, 2001	35,-- €
*WA*R 135	Auswirkungen der Verordnung über die umweltverträgliche Ablagerung von Siedlungsabfällen und über biologische Abfallbehandlungsanlagen. 63. Darmstädter Seminar -Abfalltechnik- am 12. und 13.11.2001 in Darmstadt, TU Darmstadt, FB 13, 2001	35,-- €
*WA*R 136	Bockreis, Anke: Infrarot-Thermographie zur Überwachung von Flächenbiofiltern. Dissertation, FB 13, TU Darmstadt, 2001	35,-- €
*WA*R 137	Luft, Cornelia: Luftgetragene mikrobielle Emissionen und Immissionen an aeroben mechanisch-biologischen Abfallbehandlungsanlagen. Dissertation, FB 13, TU Darmstadt, 2002	30,-- €
*WA*R 138	Danhamer, Harald: Emissionsprognosemodell für Deponien mit mechanisch-biologisch vorbehandelten Abfällen - Schwerpunkt: Modellierung des Gashaushaltes. Dissertation, FB 13, TU Darmstadt, 2002	25,-- €

*WA*R 139	Lieth, Sabine: Stickstoffelimination aus kommunalem Abwasser mit getauchten Festbetten nach Vorbehandlung mit HCR-Reaktoren. Dissertation, FB 13, TU Darmstadt, 2002	35,-- €
*WA*R 140	Streit, Hans-Ulrich: Optimierung des Kombinationsbetriebs eines Advanced Oxidation Process mit einer Stripp-Anlage zur Grundwassersanierung. Dissertation, FB 13, TU Darmstadt, 2002	vergriffen
*WA*R 141	Spura, Patrik: Ein Vergleich des anlagebezogenen tschechischen Luftreinehalterechts mit jenem der Europäischen Union vor dem Hintergrund des anstehenden Beitritts. Dissertation, Univ. Frankfurt a.M., 2002	40,-- €
*WA*R 142	Hilligardt, Jan: Nachhaltige Regionalentwicklung durch freiwillige regionale Kooperation - Faktoren einer erfolgreichen Initiierung untersucht an der Region Starkenburg. Dissertation, FB 13, TU Darmstadt, 2002	30,-- €
*WA*R 143	Heiland, Peter: Vorsorgender Hochwasserschutz durch Raumordnung, interregionale Kooperation und ökonomischen Lastenausgleich. Dissertation, FB 13, TU Darmstadt, 2002	vergriffen
*WA*R 144	Dapp, Klaus: Informationsmanagement in der Planung am Beispiel des vorsorgenden Hochwasserschutzes. Dissertation, FB 13, TU Darmstadt, 2002	vergriffen
*WA*R 145	Schüler, Doris: Untersuchungen an der Technikumsanlage VERONA zur Bildung und zum Abbau von polyhalogenierten Dioxinen und Furanen und anderen Organohalogenverbindungen in Verbrennungsprozessen. Dissertation, FB 13, TU Darmstadt, 2002	25,-- €
*WA*R 146	Grundwasserproblematik im Hessischen Ried : Eine unlösbare Aufgabe? 65. Darmstädter Seminar -Wasserversorgung- am 23.10.2002 in Darmstadt, TU Darmstadt, FB 13, 2002	30,-- €
*WA*R 147	Rückgewinnung von Phosphor aus Klärschlamm und Klärschlammasche. 66. Darmstädter Seminar -Abwassertechnik- am 07.11.2002 in Darmstadt, TU Darmstadt, FB 13, 2002	35,-- €

*WA*R 148	Schneider, Andreas: Role of LCA concepts at the Research and Development phase of a new process for waste treatment - The Trefoil Kiln process subject to IPPC and BAT requirements. Dissertation, FB 13, TU Darmstadt, 2002	25,-- €
*WA*R 149	Sonnenburg, Alexander: Untersuchungen zur Denitrifikation von Grundwasser in Schüttungen mit abbaubarem Trägermaterial. Dissertation, FB 13, TU Darmstadt, 2002	vergriffen
*WA*R 150	Emissionen aus der Abfallbehandlung. Energie - Emissionen – Messtechnik. 67. Darmstädter Seminar -Abfalltechnik- am 13. Februar 2003 in Darmstadt, TU Darmstadt, FB 13, 2003	35,-- €
*WA*R 151	Rationalisierungsmaßnahmen in der Wasserversorgung. Umsetzungsstatus und künftige Entwicklungen. 68. Darmstädter Seminar -Wasserversorgung- am 15. Oktober 2003 in Darmstadt, TU Darmstadt, FB 13, 2003	vergriffen
*WA*R 152	Verantwortungspartnerschaft beim vorsorgenden Hochwasserschutz. 69. Darmstädter Seminar - Umwelt- und Raumplanung - am 16. Oktober 2003 in Darmstadt, TU Darmstadt, FB 13, 2003	vergriffen
*WA*R 153	Biofiltration. Renaissance eines Verfahrens durch erhöhte Anforderungen im In- und Ausland ? 70. Darmstädter Seminar -Abwassertechnik- am 06. November 2003 in Darmstadt, TU Darmstadt, FB 13, 2003	35,-- €
*WA*R 154	Seiler, Kainan: Planung der Abwasserentsorgung im ländlichen Raum anhand von räumlichen Einflussfaktoren. Dissertation, FB 13, TU Darmstadt, 2004	30,-- €
*WA*R 155	Ludwig, Thomas: Entwicklung der Emissionsmessanlage DioxinCop. Dissertation, FB 13, TU Darmstadt, 2004	25,-- €
*WA*R 156	Haffner, Yvonne: Sozialwissenschaftliche Modellierung zur Privatisierung der Wasserversorgung. Dissertation, FB 2, TU Darmstadt, 2004	vergriffen

*WA*R 157	Geruch : Messung – Wirkung – Minderung. 71. Darmstädter Seminar -Abfalltechnik- am 24. Juni 2004 in Darmstadt, TU Darmstadt, FB 13, 2004	35,-- €
*WA*R 158	Qualitätssicherung bei Wassergewinnungsanlagen - Umsetzung und aktuelle Entwicklung im Regelwerk. 72. Darmstädter Seminar –Wasserversorgung– am 06.10.2004 in Darmstadt, TU Darmstadt, 2004	vergriffen
*WA*R 159	Wasserwiederverwendung - eine ökologische und ökonomische Notwendigkeit wasserwirtschaftlicher Planung weltweit ? 73. Darmstädter Seminar –Abwassertechnik– am 04.11.2004 in Darmstadt, TU Darmstadt, 2004	vergriffen
*WA*R 160	Weil, Marcel: Ressourcenschonung und Umweltentlastung bei der Betonherstellung durch Nutzung von Bau- und Abbruchabfällen. Dissertation, FB 13, TU Darmstadt, 2004	35,-- €
*WA*R 161	Unendlicher Wachstum auf unendlicher Fläche ? 74. Darmstädter Seminar –Umwelt- und Raumplanung– am 27.01.2005 in Darmstadt, TU Darmstadt, 2005	vergriffen
*WA*R 162	Gernuks, Marko: Entwicklung einer Methode zur Bewertung von Umweltaspekten mit der Ableitung von Umweltzielen im Rahmen von EMAS. Dissertation, FB 13, TU Darmstadt, 2004	vergriffen
*WA*R 163	Rother, Elmar: Optimising Design and Operation of the Biofiltration Process for Municipal Wastewater Treatment. Dissertation, FB 13, TU Darmstadt, 2005	35,-- €
*WA*R 164	Hilligardt, Jan: Regionale Kooperation der Landkreise, Städte und Gemeinden. Stand - Potenziale - Perspektiven. Habilitation, FB 13, TU Darmstadt, 2005	vergriffen
*WA*R 165	Gramel, Stefan: Privatisierung von Wasserversorgungsunternehmen - Auswirkungen auf den Umwelt- und Ressourcenschutz? Dissertation, FB 13, TU Darmstadt, 2004	35,-- €
*WA*R 166	Krause, Stefan: Untersuchungen zum Energiebedarf von Membranbelebungsanlagen. Dissertation, FB 13, TU Darmstadt, 2005	35,-- €

*WA*R 167 Rückgewinnung von Phosphor aus Abwasser und Klärschlamm. vergriffen
Konzepte - Verfahren - Entwicklungen.
75. Darmstädter Seminar –Abwassertechnik- am 12./13.12.2005 in Darmstadt,
TU Darmstadt, 2005

*WA*R 168 Hora, Maike: 30,-- €
Abfallverursacher Elektrogeräte. Ansätze zur prospektiven Bilanzierung von Abfallströmen in der umweltgerechten Produktentwicklung.
Dissertation, FB 13, TU Darmstadt, 2005

*WA*R 169 Zhang, Wensheng: 30,-- €
Ökologische siedlungswasserwirtschaftliche Konzepte für urbane Räume Chinas unter Berücksichtigung deutscher Techniken und Erfahrungen.
Dissertation, FB 13, TU Darmstadt, 2005

*WA*R 170 Steinberg, Iris: 30,-- €
Untersuchungen zur Effizienzsteigerung von biologischen und nicht-thermischen Abluftreinigungsverfahren bei der biologischen Abfall-behandlung.
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Telefon: +49 (0)6151 / 16 203 01
Fax: +49 (0)6151 / 16 203 05

Zur Autorin:

Katharina Müller studierte Geoökologie mit den Schwerpunkten Wasserchemie, Siedlungswasserwirtschaft, Hydrogeologie und Geoinformatik an der Universität Karlsruhe. Nach ihrem Abschluss arbeitete sie von 2009 bis 2015 als Wissenschaftliche Mitarbeiterin am Fachgebiet Abwassertechnik des Instituts IWAR der TU Darmstadt. Schwerpunkt ihrer Tätigkeit war ein Projekt zur Implementierung von Sanitärversorgung und landwirtschaftlicher Wasserwiederverwendung in Namibia, in dessen Rahmen die vorliegende Dissertation entstand.

Zum Inhalt:

Globales Bevölkerungswachstum, zunehmende Urbanisierung und steigende Wasserknappheit erfordern eine ganzheitliche, integrierte Vorgehensweise hinsichtlich Wasserver-, Abwasserentsorgung und Wasserwiederverwendung. Ein systemischer Ansatz ist notwendig, um im Abwasser enthaltene Ressourcen zurückzugewinnen und den durch Sanitärversorgung erzielbaren Mehrwert zu maximieren. Diese Vorgehensweise muss in Zukunft weitere Verbreitung finden. Hindernisse für die Implementierung von neuen Konzepten im urbanen Wassermanagement sind hauptsächlich fehlendes Wissen hinsichtlich systemimmanenter Unsicherheiten und Risiken, Managementherausforderungen in der Praxis, die zur Verfügung stehenden institutionellen Kapazitäten, die zur Verfügung stehenden Kapazitäten zur Einbindung der lokalen Bevölkerung, finanzielle Erwägungen sowie als Barrieren wirkende institutionelle und persönliche Neigungen. Die vorliegende Dissertation untersucht die wesentlichen Herausforderungen und Wissenslücken während der Implementierung eines Projekts zur Abwassersammlung, -behandlung und Wasserwiederverwendung in Nord-Namibia. Die im Rahmen dieses Projekts implementierte Infrastruktur umfasst verschiedene Arten von Sanitäranlagen, eine Vakuumkanalisation, eine Kläranlage mit Sedimentation und anaerober Vorbehandlung des Abwassers, aerober Behandlung mit Nachklärung, Mikrosiebung und UV Desinfektion. Das behandelte Abwasser wird in einem Becken gespeichert und mit oberirdischer Tröpfchenbewässerung auf landwirtschaftlichen Flächen aufgebracht. Das Wasser wird für die Produktion von Gemüse für den menschlichen Verzehr verwendet. Zum ersten Mal wurde ein System zur Abwassersammlung, -behandlung und Wasserwiederverwendung aus einer ganzheitlichen Perspektive analysiert und detaillierte Informationen hinsichtlich Planung, Monitoring und wichtiger Einflussgrößen während der Realisierung dargestellt. Das ist eine solide Grundlage für die bessere Planung und Implementierung von vergleichbaren Projekten. Wissenslücken, die zu falschen Annahmen und Schwierigkeiten während der Umsetzung führten, wurden geschlossen beziehungsweise adressiert und können nun von Anfang an realistisch eingeschätzt werden.

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