The Impact of Source Separation of Urine on BNR WWTPs

A simulation-based study of the impacts of urine separation on the effluent quality, capacity and configuration of BNR wastewater treatment plants.

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Acknowledgements

As I'm sure many people who have done a thesis before will agree, it is a monumental task, and one that is as much a journey of self-exploration as it is a journey of exploration into a specific research topic. Feeling alone, frustrated and overwhelmed are common feelings associated with undertaking a thesis and it is for these reasons that it means so much to have people to help you through it.

I would like to sincerely thank Professor George Ekama for his genuine enthusiasm and interest in my topic, for his invaluable contributions and for providing a never-ending wealth of information and knowledge (even when it felt like my brain was going to explode from information-overload). I would like to thank him for providing us with his cellphone number and telling us we could phone him anytime with questions: this type of attitude and genuine approachability is rare and extremely appreciated. I would like to thank him for being kind enough to allow me to borrow personal copies of two textbooks that he had, and for trusting me enough to offer me a spare laptop if I needed it. I would also like to thank Prof. Ekama for providing the direction and inspiration for this topic, which I could never have dreamt up on my own.

I would like to thank Simon Woolf, a fellow 4th year civil engineer with the same thesis topic. Our break-through meetings and a shared understanding of some of the frustrations of working with UCTPHO software helped ease the pain immeasurably.

Lastly, I would like to thank my parents, for providing me with the simple luxuries of my own computer and a fast internet connection. These made the world of difference.

"Live as if you were to die tomorrow. Learn as if you were to live forever." – Mahatma Ghandi

Terms of Reference

The BSc Eng (civil engineering) thesis topics were finalised on the 16th May 2012. Prof. Ekama requested a thesis investigating the effects of urine separation on WWTPs through computer simulation and hand-calculations only. Regular meetings were scheduled between September and November 2012 to provide chances for queries, feedback and guidance. A draft of this thesis was given to Prof. Ekama on the 26th of October and returned with comments on the 1st of November.

This final dissertation submission would make up 75% of the CIV4044S mark, with another 15% coming from the research proposal already handed in and the final 10% to be made up of the poster/e-portfolio submission (due on 16th November 2012).

Two copies of this report were to be made. An unbound copy of this report was due for submission to Mrs Ncube, while a bound copy was due for submission to Prof. Ekama, both by 10:00am on Monday 12th November 2012. A digital copy was not requested.

Executive Summary

It is evident from the 2011 and 2012 Green Drop reports published by the Department of Water Affairs and Forestry (DWAF) that South African WWTPs are not performing to the desired standards. Nationwide, the effluent from South Africa's WWTPs is largely below DWAF's guidelines. Action should be taken to rectify this situation and further investment in new WWTPs, upgrades to existing WWTPs and skills development of the WWTP operators should be encouraged. Urine separation has the potential to simplify existing and new WWTPs, making them easier to operate efficiently, while accommodating much larger catchment populations and producing better quality effluent.

This thesis investigated the effects of the different levels of urine on Nitrification-Denitrification Excess Biological Phosphorous Removal Activated Sludge (NDEBPRAS) WWTPs based on the UCT system setup. Different levels of urine separation were applied to a UCT WWTP setup and the diurnal WW loading patterns were simulated in wastewater computer software called UCTPHO. Two significant setups were modelled – one where the WWTP was not optimised (left unchanged) and one where the WWTP was optimised at each level of urine separation. These two setups produced significantly different results, but both gave insight into the effects of urine separation on BNR WWTPs.

Save for a few problems originating with denitrification in the anaerobic reactor of the optimised WWTP setup, the original hypothesis was largely proved correct. When not optimising a WWTP, the effluent quality improved in a direct relationship to the lower influent nutrient concentrations with increasing urine separation. This effectively represented a WWTP that was operating efficiently and below maximum flow capacity. The aeration requirements showed real decreases with increasing urine separation. However, the gains in capacity were not as significant when not optimising the WWTP.

When simulating urine separation on an optimised WWTP, the gains in capacity are significantly higher than when not optimising the plant. The gains in effluent quality were not as significant in the optimised WWTP as in the unoptimised WWTP, as the optimised WWTP was configured to be 'on the edge' with respect to nutrient removal. However, some denitrification in the anaerobic reactor resulted in unexpected improvements in the effluent nitrate of the optimised WWTP but high peak P effluent concentrations. The aeration requirements showed decreases in terms of the oxygen utilization rate, but showed increases in terms of the real mass of oxygen required per day. However, these aeration demand increases were a direct result of the massive gains in capacity and increase of catchment population size with the optimised WWTP. Above 80% urine separation, a two-reactor system could be implemented, facilitating significantly-simpler WWTPs, making them easier to build and operate efficiently, while providing significantly higher effluent quality.

The largest gains with urine separation technology would manifest as an increase in capacity. It was found that increasing urine separation had the effect of profoundly increasing

the capacity of an WWTP, showing a capacity increase of 234% (if the original capacity was considered as 100%) for a WWTP that was optimised for each level of urine separation. Either the capacity could be drastically increased (for a fixed size WWTP) or the size of new WWTPs could be drastically decreased (for a fixed population) based on this technology (when compared to the design without urine separation technology).

Through the implementation of low-flush diversion toilets and flush-less urinals, the implementation of (full) urine separation could potentially save up to 20% of the freshwater used by the public. This would have a drastic effect on the water resources of South Africa and could go a long way to alleviating freshwater shortages and the strain on water resources and infrastructure in this country.

There are significant benefits of urine separation to BNR WWTPs, but these need to be weighed up against the costs of implementing urine separation technology and constructing and running decentralised urine treatment facilities. While urine separation holds significant benefits as a technology on its own, it is most likely that these benefits would not be enough to justify a retrofit of existing toilets and sewers. This technology could however, play an important role in unlocking greater benefits when combined with other technologies, most notably when combined with seawater flushing technology. This combined technology is worthy of further research and could be a vital tool in humanity's approach to achieving global sustainability.

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Glossary of Terms and Abbreviations

Aerobic	Aerated: presence of dissolved Oxygen
Anaerobic	Unaerated: absence of dissolved Oxygen but presence of Nitrate (or other electron acceptor)
Anoxic	Unaerated: presence of other electron acceptor (i.e. Nitrate) but not dissolved Oxygen
AS	Activated Sludge
ANO	Autotrophic Nitrifying Organisms
BNR	Biological Nutrient Removal
BNRAS	Biological Nutrient Removal Activated Sludge
COD	Chemical Oxygen Demand
CRR	Cumulative Risk Rating
DO	Dissolved Oxygen
DWAF	Department of Water Affairs and Forestry (aka: Department of Water Affairs)
E or effl.	Effluent from WWTP
EBPR	Excess Biological Phosphorous Removal
ER	Endogenous Respiration
FSA	Free and Saline Ammonia
ISuspS (or ISS)	Inorganic Suspended Solids (both settleable and non-settleable solids)
ISetS	Inorganic Settleable Solids (only settleable)
N/A	Not Applicable
ND	Nitrification and Denitrification
NDEBPRAS	Nitrification Denitrification Excess Biological Phosphorous Removal Activated Sludge
OUR	Oxygen Utilization Rate

OHO	Ordinary Heterotrophic Organisms
PAO	Phosphate Accumulating Organisms
PST	Primary Settling Tank
SA	South Africa or South African (depending on context)
SST	Secondary Settling Tank
TSuspS (or TSS)	Total Suspended Solids (both settleable and non-settleable solids)
TSetS	Total Settleable Solids (only settleable)
UCT	University of Cape Town
VFA	Volatile Fatty Acids
VSS	Volatile Suspended Solids (both settleable and non-settleable solids)
WW	Wastewater
WWTP	Wastewater Treatment Plant

List of Symbols

%	Percentage
°C	Degrees Celcius
ℓ	Litre
Mℓ	Megalitre (1 000 000 litres)
x:y	Ratio of x to y
CO_2	Carbon Dioxide
O_2	Oxygen Gas
P	Phosphorous
OP	Ortho (dissolved) Phosphate
PP	Poly-Phosphate
N	Nitrogen
TP	Total Phosphorous
TN	Total Nitrogen (includes Nitrate)
TKN	Total Kjeldahl Nitrogen (does not include Nitrate)
FSA	Free and Saline Ammonia
N_{ae}	Effluent Ammonia
NO_2	Nitrite
NO_3	Nitrate
N_2	Nitrogen Gas
Mg	Magnesium
K	Potassium
>	Greater than
<	Less than
=	Equal to
COD_{TOTAL}	Total COD
COD_{RBSO}	COD that is readily biodegradable and soluble

COD_{BPO}	COD that is biodegradable and particulate
COD_{UPO}	COD that is unbiodegradable and particulate
COD_{USO} or S_{us}	COD that is unbiodegradable and soluble
	/p.d Per Person Per Day
mg	Milligrams
kg	Kilograms
h or hr	Hour
R_s	Sludge Age (Mean solids retention time)
fx	Unaerated sludge mass fraction
fxdm	Anoxic sludge mass fraction
b_{at}	Endogenous Respiration rate of nitrifiers at temperature T
μ_{AmT}	Maximum specific growth rate of nitrifiers at temperature T
K_{nT}	Half saturation coefficient of nitrifiers at temperature T

1. Chapter I – Introduction

1.1 Subject and Motivation

With the modern drive for humans to reduce their impact on the world around us and to develop in more sustainable ways, novel ideas to achieve these aims are being put forward. In the field of wastewater treatment, some innovative ideas to save water, reduce electricity consumption (thus reducing CO_2 emissions at power plants) and reduce the impact of wastewater on natural aquatic systems are being investigated. One such idea is that of urine separation. Toilet flushing represents 20–30% of domestic water consumption (Ekama, 2011b), and if this could be reduced or if another source of toilet flush-water could be found, freshwater could be saved and a noticeable impact on the urban water cycle could be observed. Eutrophication, pollution and the effects of some micro-pollutants in pharmaceuticals present noteworthy problems to the water bodies of many urban areas. Much of the Nitrogen, Phosphorous and micro-pollutants that cause these problems are contained within human urine. If these urine nutrients and micro-pollutants could be isolated from the main wastewater stream and kept concentrated, they could be removed more efficiently. One of the primary objectives of modern wastewater treatment plants is to remove these P and N nutrients (and the removal of micro-nutrients and micro-pollutants could be legislated in the future). If human urine could be separated from the wastewater stream and treated decentrally, then wastewater treatment plants (WWTPs) could increase their capacities, decrease their complexity and discharge higher quality effluent, all while reducing their energy requirements.

The nutrients contained in urine could be recovered and reused in agriculture. As time goes by and some of these nutrients (P and K for example) become scarcer, the opportunity exists to harvest these nutrients from the concentrated urine stream to make fertilizer at competitive costs. Urine separation is not a new technology and is well-known as an idea in the academic world. However, in terms of mainstream public knowledge, this idea is still relatively new. Urine separation has been implemented in rural areas to keep faeces dry and facilitate the development of dry faecal compost, but this technology also holds many other potential benefits for application in the dense urban environment. It is these effects of urine separation in the urban environment, and particularly at WWTPs, that will be investigated in this thesis.

1.2 Background to Investigation

The size of the Biological Nutrient Removal (BNR) WWTP is governed by the requirement to nitrify Ammonia to Nitrate, which imposes long sludge ages and hence large reactor volumes. Considering that urine contains about 80% of the N and 50% of the P in waterborne municipal wastewater but constitutes only the 1% the liquid volume, there is potential to drastically change the setup of BNR WWTPs with urine separation. If the urine were collected separately, would the influent TKN concentration be low enough to no longer require N removal by nitrification and denitrification? If yes, then the capacity of existing BNR plants could be significantly increased by reducing the sludge age. This thesis will explore this question with BNR activated sludge simulation models.

This topic was made available as a thesis research topic by Prof. George Ekama. Urine separation, along with seawater flushing technology, forms the body of a current research interest held by the supervisor of this thesis. The choice of topic therefore derives from the interest in the innovative research being done by Prof. Ekama and his associates (which was recently recognised by the receipt of a 2012 IWA Project Innovation Award). This research involves the integrated ideas of using seawater as a water resource for toilet-flushing and cooling and greywater for cooling purposes, and thus involves a Triple Water Supply (TWS) system. This TWS system has been implemented in a pilot study at Hong Kong International Airport and has resulted in significant water and electricity savings. When combining this technology with urine separation, there exists the opportunity to develop innovative WWTP systems, such as the 'sulphate reduction, autotrophic denitrification, nitrification integrated' (SANI) process, which requires no aeration inputs and produces minimal sludge waste.

This research into urine separation thus forms one component of a greater body of work and technological development currently being undertaken by Prof. Ekama.

1.3 Objectives of Thesis

- Through research and computer-aided simulations, show the impacts of varying degrees of urine separation on BNR WWTPs, including:
    - The impact on effluent Ammonia and N concentrations (i.e. show the effects of urine separation on nitrification and denitrification), and the impact on effluent P concentrations (i.e. effect on P removal in BNR WWTPs)
    - The impact on size, capacity, operational complexity and aeration requirements of BNR WWTPs.
- Investigate at what level (if this level exists) of urine separation nitrification will no longer need to be sustained in BNR WWTPs, showing the consequences of eliminating the need for nitrification and commenting on whether this level of urine separation is attainable.
- Show the potential benefits and drawbacks of using this technology by focussing on the impacts on WWTPs but also through illustrating an understanding of the broader impacts of urine separation on society.

If it is concluded that as a result of urine separation the capacity of existing WWTPs could be significantly increased, or new WWTPs could be significantly simpler and smaller, then this technology could be recommended to be implemented in the future.

1.4 Limitations and Scope of Investigation

Due to the physical constraints of UCT's WW labs having not been operational during the construction of the New Engineering Building during 2012, performing physical (batch) testing during this research was not possible. Even if these labs were operational, it is doubtful that there would have been the time or resources available to include a physical-testing aspect to this undergraduate thesis. This is one reason why the research in this thesis was 'limited' to computer simulations of the various WWTP configurations and variations in urine separation.

While there are quite clearly benefits to implementing urine separation with seawater toilet flushing (as will be discussed in the Literature Review), this thesis is primarily focussed on the idea of urine separation as a stand-alone option. The background to this investigation is indeed contained within broader solutions to water conservation and an integrated move towards sustainability. However, this thesis investigates the merits of urine separation as a technology on its own. This system could practically be implemented before the seawater flushing system is implemented, or could be implemented country-wide as a base technology while being combined with seawater flushing in coastal (and other applicable) areas of South Africa. It is for this reason that this technology should have stand-alone benefits for implementation, and hence this thesis investigates the stand-alone impacts of urine separation on WWTPs.

Although processes such as energy generation through methane recovery are important aspects to the functioning of WWTPs that employs anaerobic digestion, the *quantitative* effects of urine separation on these extraneous processes were not explored. This was deemed outside of the scope of this thesis.

While nutrient recovery from wastewater is discussed in the Literature Review of this thesis, no further attention was made to nutrient recovery in the simulation phase of this research. Nutrient recovery from concentrated urine streams presents interesting considerations and research potential, but was not looked at in any detail. This was deemed outside of the scope of this thesis.

This research was primarily concerned with the impacts of urine separation on WWTPs, while also assessing the impacts of urine separation on the potential catchment population of WWTPs. This research was not however concerned with addressing the problem of the separate urine treatment facilities, and the technology, costs and conceptual problems associated with the potential urine treatment facilities were not investigated further. This was deemed outside of the scope of this thesis.

1.5 Plan of Development

This thesis follows the following plan of development:

A literature review is presented, where information that is relevant background to this topic is provided. Expected results for urine separation based on previous research and experiments are given and the potential (expected) benefits of urine separation as a stand-alone technology are discussed.

The goals of the experimental procedure are then highlighted and a hypothesis is proposed. An explanation of the different testing phases and their assumptions (along with justifications) are given.

The results of the two testing phases are then presented and discussed. Conclusions are drawn from the resulting discussions, highlighting a source of possible simulation inaccuracy. Recommendations are then made based on these conclusions.

1.6 Methodology

This methodology section briefly describes the methodology of this thesis, and not of experiment in detail. For a detailed methodology of the experimental process that was followed, please refer to Chapter III – Simulation and Modelling.

The research for thesis involved regular consultations with Prof G. Ekama. Course notes for the post-graduate course offered at UCT were provided, and tutorials on designing influent data, designing a WWTP and using UCTPHO simulation software were given and completed. These tutorials were worked through with Prof. Ekama to ensure that the correct methods had been followed. Once a confidence in the calculations and processes had been established, the individual catchment population and influent WW characteristics for the 0% urine separation situation were generated. A base WWTP was designed by hand calculations (performed in Microsoft Excel) and the same WWTP was simulated in UCTPHO to compare the results.

From this point onwards, the different levels of urine separation technology were first simulated in an unchanged (unoptimised) WWTP and the data was captured and results were formulated. The different levels of urine separation were then simulated on a WWTP that was optimised for N removal at each level of urine separation (optimised by changing the sludge age and reactor mass fractions but keeping the total volume the same). When each level of urine separation was performed on the influent data, the catchment population was increased by an equivalent amount to maintain the TSS concentration in the aerobic reactor at the same level throughout. This was to simulate the capacity increase while maintaining the original setup of the WWTP (the design and operation of the SST is bound by the TSS concentration in the reactor and as such the TSS concentration had to remain constant throughout).

2. Chapter II - Literature Review

2.1 Constituents and Properties of Municipal Wastewater

The term 'wastewater' is used to describe refuse water (and any waste matter suspended in this water) that has been used and discarded. For the majority of the developed world, this wastewater is conveyed through buried, gravity-driven sewers to facilities that treat this water for discharge into natural water bodies. In order to facilitate waterborne sanitation, a prerequisite is that users are supplied with potable water.

Only a small quantity (between 1ℓ and 2ℓ per person per day) of the potable water supplied to users through water reticulation networks is physically consumed by public water users. The rest of the water that people utilise is generally used for cleaning, cooking and watering of gardens. Whether washing dishes, bodies, clothes or flushing toilets in a household environment, most of the water goes almost directly from the water reticulation network and into the sewer network. Domestic (residential) wastewater is thus made up of contributions from a variety of sources within the household environment.

'Municipal wastewater' is a term often used to define a combination of domestic and commercial wastewater (Mbaya, 2011). Municipal WW generally consists of blackwater (flush-water containing both urine and faeces) and greywater from bathrooms, kitchens and laundries (from sinks, baths, showers, dishwashers and washing machines).

2.1.1 Toilet Water

Blackwater (toilet flush-water with both urine and faeces) represents 20-30% of domestic water consumption (Ekama, 2011b). Generally, greater quantities of water are used to flush urine than to flush faeces. On average, urine flushes number around 5 per person per day, while faeces flushes number around 1 per person per day (Wilsenach and Loosdrecht, 2002).

2.1.1.1 Yellowwater

The term 'yellowwater' is used to describe that portion of blackwater that contains only urine and its associated flush-water. Around 35ℓ/p.d is used to flush urine (STOWA, 2002; Jonsson et al., 1997).

2.1.1.2 Brownwater

The term 'brownwater' is used to describe that portion of blackwater that contains only faeces, toilet paper and its associated flush-water. While estimates vary, around 7-10ℓ/p.d is used to flush faeces (STOWA, 2002; Jonsson et al., 1997), making up the brownwater contribution.

2.1.2 Greywater

Greywater makes up the bulk of the domestic wastewater stream. As the name implies, 'greywater' is water that appears only slightly tainted by detergents, soaps etc. and generally contains low concentrations of organics as it is fairly dilute. Greywater generally comes from two sources: bathroom greywater and kitchen greywater. Bathroom greywater includes all the water from bathroom sources excluding the toilet, i.e. shower, bath and basin refuse water. Kitchen greywater includes water used to wash dishes and clean clothes, as well as water used in various cooking processes. Bathroom greywater contains almost no nutrients or organics (measured as COD: Chemical Oxygen Demand), while water from kitchen greywater generally contains substantial concentrations of COD and some nutrients (Wilsenach, 2006).

2.1.3 Industrial Water

Depending on the location of the Wastewater Treatment Plant (WWTP), there can also be wastewater contributions from industrial processes, often containing significantly higher concentrations of chemicals than household wastewater. Water used in industrial cooling and cleaning processes could also enter the wastewater network, although this water could also find its way into stormwater systems (either through negligence or for convenience on the part of the wastewater producer).

2.1.4 Rainwater

Rainwater can be accommodated in the sewer network, as is done in most parts of Europe, using combined sewers where no separate stormwater network exists. In South Africa however, separate networks are used to transport wastewater and stormwater. Although this system is intended to keep wastewater and stormwater (rain and runoff water) separate, there is still some cross-contamination. This infiltration of stormwater into the sewer network leads to significantly higher wastewater flows in wet weather than in dry weather (CSIR, 2000).

2.2 Importance of Wastewater Treatment

Around 5 million people die annually due to water borne diseases (Wilsenach, 2006). Millions more get sick through contact or consumption of contaminated water. Generally, this problem is caused when humans discharge some form of untreated wastewater into receiving water bodies upstream of other water users. With some exceptions, this problem is particularly poignant in Africa and other parts of the developing world, where treatment of wastewater is limited, as explained by Esrey (2002), “In Africa virtually all sewage is discharged without treatment into receiving water bodies. The figure for Latin America, the Caribbean and Asia are not much better”.

Basic wastewater treatment is still desperately needed to reduce mortality and sickness in certain parts of the world. However, in Europe and other developed regions of the world, especially in the last decade, the focus of wastewater treatment has evolved from simply preventing human illness and mortality to preventing environmental damage and degradation (Wilsenach, 2006). Where there are already WWTPs that achieve the fundamental goal of removing pathogens effectively, they also seek to prevent pollution and ecological damage to the waters that they discharge into. In today’s society, human systems are no longer judged solely on their ability to not harm other people in their operation (this is and should be expected), but are judged also on their sustainability and impact on the ‘natural world’ in which we live.

2.3 Current Objectives of Wastewater

The most basic objective of wastewater treatment is to remove pathogens and prevent disease or death to people downstream (if discharging into a river) or nearby (if discharging to a coastal environment). This is most often achieved in the biological reactors or by retaining the WWTP effluent in maturation ponds for about 30 days, where the pathogens die off (Ekama, 2012). Other methods to ensure that the pathogens are removed from the effluent are disinfection or purification via Chlorination or Ultra-Violet light. Once this objective has been achieved (which is *supposed* to be done by all modern wastewater treatment plants) there are other objectives that focus on reducing the impact of the WWTP effluent on natural aquatic systems. These objectives include:

1) The removal of organic material (proteins, carbohydrates or fats) to reduce receiving water deoxygenation
2) The reduction of Ammonia (NH_3) – the generally available form of N – to minimise toxicity and deoxygenation of the receiving water bodies
3) The removal of nutrients Nitrogen (N) and Phosphorous (P) to reduce eutrophication

The effluent standards on Ammonia (number 2 above) in WWTP effluent are set to prevent Ammonia making the receiving water body toxic to aquatic life – which it can do in high enough concentrations (i.e. effluent Free and Saline Ammonia (FSA) > 10 mg/ℓ) (Mbaya, 2011). The effluent standards on N, P and organic material are set to prevent eutrophication and deoxygenation.

Some relevant DWAF effluent guidelines for WWTP discharge are given in Table 2.1 below (Department of Environmental Affairs and Fisheries, 1984).

Table 2.1 South African WWTP effluent guidelines (DWAF, 1984)

Effluent Parameter	Units	General Standard	"Special" Standard
COD	mgCOD/ℓ	75	30
Ammonia (as N)	mgN/ℓ	10	1
Nitrate (as N)	mgN/ℓ	10	1.5
Ortho-P (as P)	mgP/ℓ	1	1

Currently, no effluent standards exist for micro-pollutants such as pharmaceuticals, and accordingly, the removal of micro-pollutants (endocrine disruptors and pharmaceuticals) is not currently an objective of South African WWTPs (nor indeed in most other parts of the world). However, increasing attention has recently been paid to the presence of micro-pollutants in WWTP effluent due to the increasing likelihood of effluent water reuse in water-scarce areas and due to the documented environmental effects of these micro-pollutants in the aquatic environment (Joss et al, 2005).

2.3.1 Preventing Eutrophication

High levels of organics or N and P nutrients in the effluent of a WWTP can cause an imbalance in natural aquatic ecosystems and cause eutrophication. Eutrophication has negative side-effects such as causing harmful algal blooms in lakes and rivers and 'dead zones' in coastal marine ecosystems. Algal blooms can kill off natural, indigenous plant and animal life by utilising the available dissolved oxygen in the water (deoxygenation) and also by smothering other plant life and preventing the penetration of sunlight. Examples of the visual and physical effects of eutrophication are shown in Figure 1 below.

Figure 1: (a) Eutrophication (algal bloom) at Hartebeesfontein Dam, November 2010 [photo: R Ingle] and (b) Removing macroalgal blooms at the Olympic Sailing venue, Beijing, China, 2008 [source: Conley et al, 2009]

The best way to prevent eutrophication is to limit the dissolved (ortho-) phosphate in the WWTP effluent that discharges into the receiving water bodies. "Phosphorous is the key element to remove from aquatic environments to limit growth of aquatic plants and algae, and thus, to control Eutrophication" (Henze, van Loosdrecht, Ekama and Brdjanovic, 2008). However, Nitrogen is also a crucial eutrophication nutrient, especially in coastal environments (Conley et al, 2009).

2.4 Achieving the Goals of COD and Nutrient Removal from Wastewater ("How WWTPs Work")

With today's state-of-the-art nitrification-denitrification excess biological phosphate removal activated sludge (NDEBPRAS) systems such as the UCT system, good effluent qualities are obtainable e.g. Chemical Oxygen Demand (COD) < 50 mg/ℓ; N_{tot} or Total Kjeldahl Nitrogen (TKN) < 10 mg/ℓ; NH_3< 1 mg/ℓ and TP (P_{tot}) (PO_4) < 1.0 mg/ℓ (Mbaya, 2011). Generally, the higher the TKN/COD and TP/COD ratios of the influent wastewater, the higher the cost and operational complexity required at the WWTP to achieve these effluent qualities.

The following sections outline how the goals of Organics, Nitrogen and Phosphorous removal from the influent wastewater are achieved at these NDEBPRAS WWTPs and generally show how the complexity of these plants has to increase to accommodate high influent loads in order to get good effluent qualities. For readers who are familiar with the operations of typical NDEBPRAS WWTPs, the following sections (2.4.1, 2.4.2, 2.4.3, 2.4.4 and 2.4.5) may seem superfluous and a brief skim of the following sections is thus advised.

2.4.1 Physical Separation: Screening, Degritting and Primary Sedimentation

These processes are all physical processes that remove particulate matter from the wastewater based on either particle size or density of the removable matter. These processes assist the biological processes by ensuring that large (and often unbiodegradable) pollutants and litter do not interfere with the mechanical equipment or impair the biological processes later on.

2.4.1.1 Screening

It is normal practice to have coarse screens at the entrance of the WWTP to prevent any large objects entering the main WWTP operations. These large, often inorganic objects, if allowed to pass through the coarse screens, may interfere with the operation of mechanical cleaning equipment associated with the main or fine bar screens. Main screens are provided to remove any gross solids passing through the coarse screens and which may interfere with the operation of pumps and cause blockages in pipelines. At smaller plants the screened material is often buried, while at larger plants the material is often disintegrated and added to the works further on or washed and sent to landfill (Ekama, 2012). These screens can either be cleaned manually or mechanically (usually at larger plants). Figure 2 below shows examples of manual and mechanical methods for cleaning the objects trapped on the primary screens.

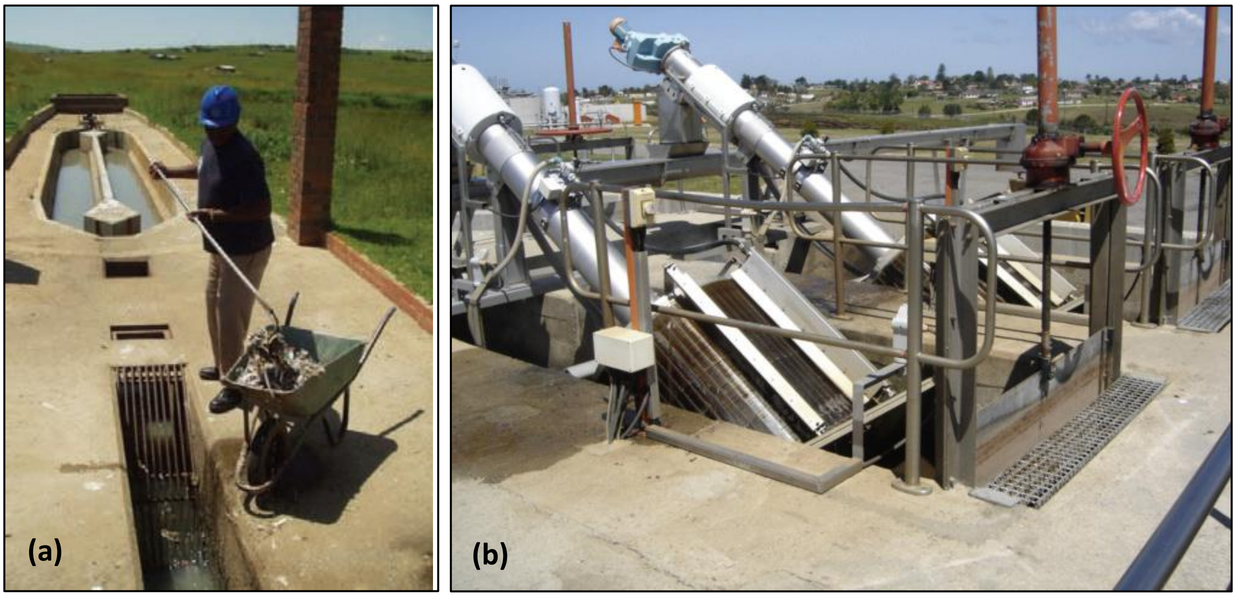

Figure 2: (a) Manual cleaning of screens and (b) Mechanical cleaning of screens [Nozaic et al, 2009]

2.4.1.2 Degritting

Sewers are generally designed to flow at a water velocity faster than 1m/s to ensure that solid material is swept along to prevent blockages in the sewers (Ekama, 2012). The result of this design is that a lot of grit and dense solids arrive at the head of the WWTP. The purpose of degritters is to remove 'grit' consisting of sand, gravel or solid (inorganic) material that have specific gravities greater than that of organic solids in wastewater. At large plants, vortex degritters are used where water is swirled around a cylindrical tank causing the heavier materials to accumulate in the centre of the 'vortex' for removal. Degritters are provided to protect mechanical equipment further down the plant from abrasion and abnormal wear. Cleaned grit is often buried. The organic matter that is kept in suspension is passed onto the subsequent unit processes of the treatment system (Ekama, 2012).

2.4.1.3 Primary Sedimentation

Primary settling tanks are employed to remove the remaining settleable solids that pass through the primary screens and degritters. Primary settling tanks reduce the flow sufficiently to allow sedimentation of the denser particulate matter, while the clearer liquid escapes over the wall of the settling tank. The majority of the particulate matter settled here is organic (Ekama, 2012). Removal of these solids before biological treatment reduces the organic (COD) load on the biological reactor, resulting in savings in biological reactor size, aeration power input and secondary sludge production (although the primary sludge has to be treated, stabilised and safely disposed of and these costs have to be considered) (Ekama, 2012). Figure 3 below shows the size and setup of a Primary Settling Tank (PST) at Athlone WWTP.

Figure 3: Temporarily unused PSTs at Athlone WWTP (July 2012) [Photo: M Grüter]

If the WWTP has primary sedimentation, then the biological reactors treat ‘settled sewage’, while if there is no primary sedimentation then the influent is referred to as ‘raw sewage’. Primary sedimentation reduces the COD, settleable solids and total solids loads by about 35%, 90% and 45% respectively (Ekama, 2012).

2.4.2 Biological Removal of Organics

The biodegradable portion of the influent organics is biologically used in the growth process of ordinary heterotrophic organisms (OHOs) in the aerated (oxygenated) part of the biological reactor. Even suspended (non-settleable) solids that have passed through the PSTs and are unbiodegradable are enmeshed in the activated sludge and prevented from escaping in the effluent. The aim of Activated Sludge WWTPs is to remove all organics except that portion which is unbiodegradable and soluble. The activated sludge (AS), after proceeding through the biological reactors, is passed through the secondary settling tanks (SSTs) where (if designed correctly) the solid sludge mass settles to the bottom and is recycled to the bioreactor, and only liquid (and soluble matter) passes over the SST weir and escapes in the effluent.

Through metabolism (a combination of catabolism and anabolism), the OHOs use oxygen and biodegradable substrate to grow, giving off Carbon Dioxide and water. The molecular formula for OHOs can be approximated as $C_5H_7O_2N_{0.8}P_{0.09}$ (Henze et al, 2008). From the formula, it is clear that both Nitrogen and Phosphorous are required for biological OHO growth in the bioreactor. This assists in both biological Nitrogen and Phosphorous removal, explained in Section 2.4.3 “Removal of Nitrogen” and Section 2.4.4 “Removal of Phosphorous” further on.

For an AS system, a certain quantity of sludge must be harvested from the bioreactors to avoid the biomass concentration in the reactors becoming too high. The sludge that is taken out daily (in SA directly from the biological reactor) is waste sludge or secondary sludge and is dealt with separately at the sludge disposal stage of the treatment works (Ekama, 2012). The biological growth process combined with this solids-removal process ensures that solid organic (and inorganic) materials are removed from the wastewater and do not escape with the effluent and into the receiving water bodies. Figure 4 on the next page shows an example of an aerated biological reactor.

Figure 4: Fine-bubble aeration technology in an aerobic biological reactor at Athlone WWTP (July 2012) [Photo: M Grüter]

2.4.3 Removal of Nitrogen

Most of the nitrogen in sewage is in the form of Ammonia (NH_4^+) (Ekama, 2012). The nitrogen that is locked up in organic materials is released as Ammonia when the organics are broken down by the OHOs in the bioreactor (called ammonification). There is a limit to the amount of Nitrogen that can be taken up by OHO sludge growth, and this is limited by factors such as reactor size, sludge age and aeration inputs.

Nitrification and denitrification are biological processes utilised at WWTPs to lower the nitrogen content in the WWTP effluent. As explained by Ekama (2011a), the term "nitrification" describes the biological process whereby free and saline Ammonia (FSA) is oxidised to Nitrite and then Nitrate. The nitrifying organisms that facilitate this process obtain their carbon (required for growth) from dissolved CO_2 and obtain their energy requirements either from the oxidation of Ammonia to Nitrite or from Nitrite to Nitrate (Ekama, 2011a). Nitrification happens in two steps as a result of the processes of two different types of organisms. The ANOs convert FSA to Nitrite (NO_2) and Nitrite oxidising organisms convert Nitrite to Nitrate (NO_3) (Ekama, 2011a). This two-step nitrification process can be explained by the basic stoichiometric redox reactions on the next page:

$$NH_4^{\ +}\ (FSA) + \frac{3}{2}O_2 \rightarrow NO_2^{\ -} + H_2O + 2H^+ \qquad \textbf{Equation 1}$$

$$NO_2^{\ -} + \frac{1}{2}O_2 \rightarrow NO_3^{\ -} \qquad \textbf{Equation 2}$$

ANOs are obligate aerobes, which means they can only grow under aerobic conditions (i.e. active aeration in the biological reactor is required) (Ekama, 2011a). This therefore adds to the aeration costs (electrical costs) at WWTPs.

With nitrification, a minor fraction of the Nitrogen is utilised in the growth process of the ANOs and becomes part of the solid AS mass. In this process, most of the Nitrogen does not leave the wastewater, but simply changes from one form to another (from FSA to Nitrate). This is beneficial in reducing the concentration of Ammonia in the effluent, which can be toxic to aquatic life in the receiving water bodies. However, biological N removal, where N is removed by transferring it from the liquid phase to gas phase, requires another process. This process is denitrification, where Nitrates are used as an electron acceptor by facultative aerobes and the Nitrogen is removed from the wastewater, with much of the Nitrogen being released as Nitrogen gas. This process is given by the following denitrification equation:

$$\frac{1}{5}NO_3^{\ -} + \frac{1}{4}CH_2O + \frac{1}{5}H^+ \rightarrow \frac{1}{10}N_2 + \frac{1}{4}CO_2 + \frac{7}{20}H_2O \qquad \textbf{Equation 3}$$

Where CH_2O (formaldehyde) above represented a specific carbon source but could be replaced by a different carbon source. As shown in Equation 3 above, denitrification occurs under anoxic conditions, where dissolved oxygen is not present and NO_3^- acts as the electron acceptor.

In this way, Nitrogen is released in the gas phase, removing it from the wastewater. The facultative heterotrophs that perform this function also capture some of the Nitrogen from the wastewater as solid mass during the growth process. Denitrification becomes possible once nitrification takes place, and occurs in zones of the biological reactors that are intentionally not aerated (Ekama, 2011a). Incorporating denitrification into a WWTP allows a reduction in the oxygen demand of the biological reactor, because under anoxic (oxygen-deficient) conditions, Nitrates serve as the electron acceptor instead of dissolved oxygen (DO) in the breakdown of organics by (facultative) heterotrophic organisms (Ekama, 2011a). In this way, aeration costs can be reduced. In any case, it is these important processes of nitrification and denitrification that account for the majority of the Nitrogen removal at WWTPs. There are also other ways to ensure Nitrogen removal, such as post-denitrification units, where filters are used and methanol is dosed as a carbon source (Wilsenach, 2007).

Figure 5 on the next page shows the exit routes of influent Nitrogen at a single sludge ND activated sludge system, also highlighting the percentages of these different exit routes.

Clearly in these ND AS systems, the removal of influent N via denitrification (converting soluble N to gaseous N) plays a large role in reducing the Total N in the effluent.

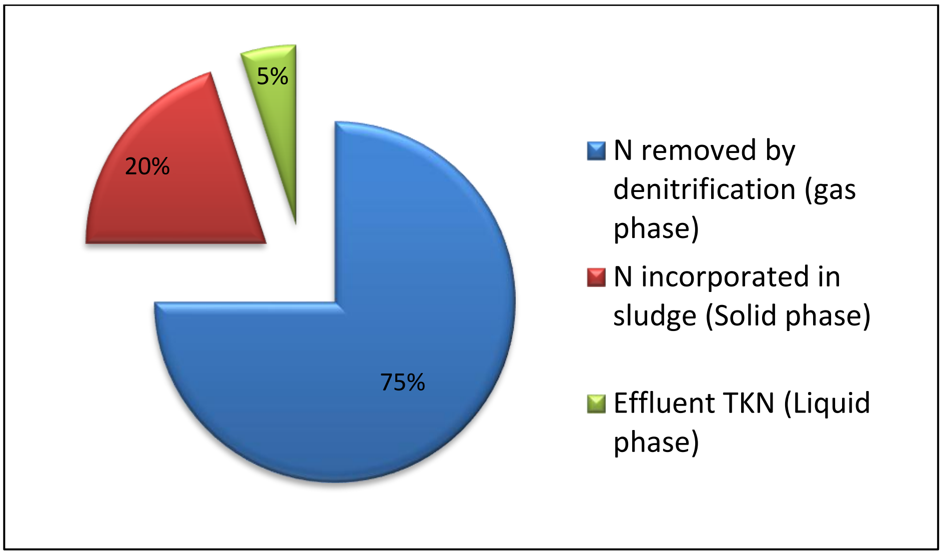

Figure 5: Typical exit routes for influent N in a single sludge ND AS system [adapted from Henze et al, 2008]

2.4.4 Removal of Phosphorous

Phosphorus is removed from wastewater by transforming it from the dissolved liquid phase to the solid phase. This can be done chemically or biologically, or both together (Ekama, 2012). When done chemically, Aluminium (Al) sulphate or Iron (Fe) salts are added to the water and the Al or Fe precipitates with the phosphate, leaving the sulphate or chloride in solution (Ekama, 2012). The precipitated solid material becomes part of the sludge mass which is separated in secondary settling tanks and eventually wasted via the waste flow.

Biological Phosphorous removal happens on a small scale in the growth process of OHOs (as explained in Section 2.4.2 “Biological Removal of Organics”) and even in the growth of facultative aerobes in the denitrification process.

However, in the same way that Nitrogen cannot always be removed via OHO sludge mass, this is often not sufficient to achieve acceptable or full P removal either (conversion from dissolved phase to solid sludge mass). For this reason, the Excess Biological Phosphorous Removal (EBPR) processes are often utilised to achieve the necessary P removal at WWTPs to ensure that the effluent P quality meets the effluent guidelines. When excess Phosphorous removal is done biologically, a special group of bacteria are encouraged to grow in anaerobic (no oxygen or Nitrate) zones in the activated sludge system. These organisms, called Phosphorous Accumulating Organisms (PAOs), take up large concentrations of phosphorus, much larger than normal OHOs (up to 12 times more P – in terms of mgP/mgVSS - than OHOs). If one cannot grow enough of these bacteria (it depends on the readily biodegradable COD fraction of the influent wastewater) to remove all the phosphorus then one can supplement the removal by the chemical (rather than biological) methods (Ekama, 2012).

2.4.5 Secondary Settlement (Clarification)

The mixed liquor (sludge and water) from the biological system is discharged to secondary settling tanks (SSTs). In these tanks, the calm hydraulic conditions allow the solid material (sludge) to settle to the bottom, much the same way as the influent solids settle to the bottom of the PSTs. The sludge that settles in these PSTs is recycled to the bioreactor system. The clarified water overflows around edges of the settling tanks and becomes the effluent from the treatment plant, which may or may not then proceed to maturation ponds, purification or disinfection, where the majority of pathogens (e.g. E. coli) are destroyed (Ekama, 2012).

In Figure 6 (a) below, the sludge blanket at the bottom of the SST has been disturbed during peak wet-weather flow and solid matter is overflowing the weir of the SST (as evident by the dark flocculant matter), while in Figure 6 (b) much clearier SST effluent is observed.

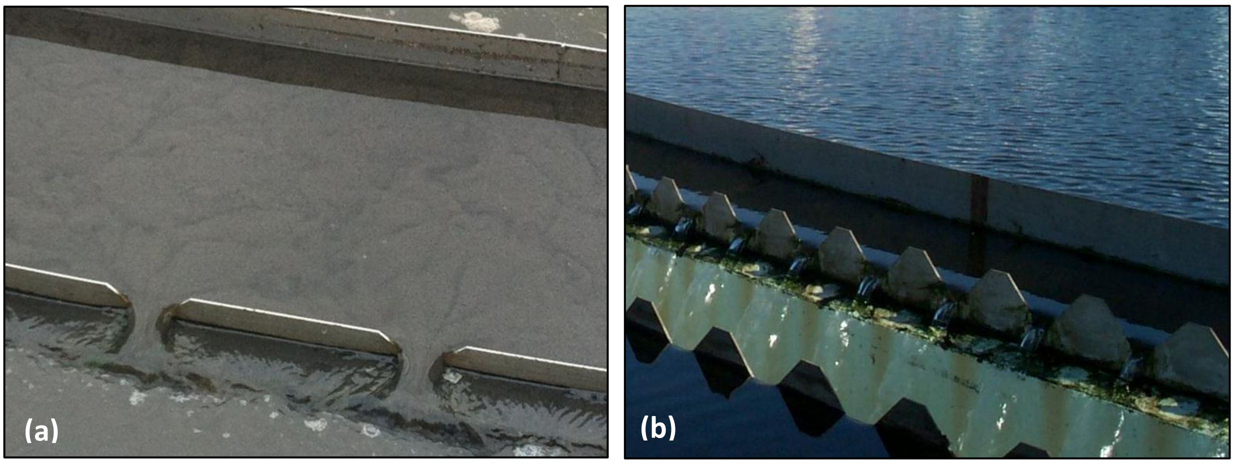

Figure 6: (a) An SST at Malmesbury WWTP not performing well during peak wet-weather flow (July 2012) [Photo: M Grüter] and (b) An SST at a WWTP with clear overflow effluent [souce: Karia et al., 2006]

2.5 Introducing the UCT System as a Type of WWTP Setup

There are many systems of operational setup that can be used to achieve the N and P removal goals of wastewater treatment at WWTPs, such as the Ludzack-Ettinger system, the four-stage Barenpho system, the BCFS system, the JHB system, the UCT system and the Modified-UCT system (Henze et al., 2008). While all of these systems are interesting and have different benefits and drawbacks for different applications, the system that this thesis will focus on is the UCT system, whose setup is shown below in Figure 7.

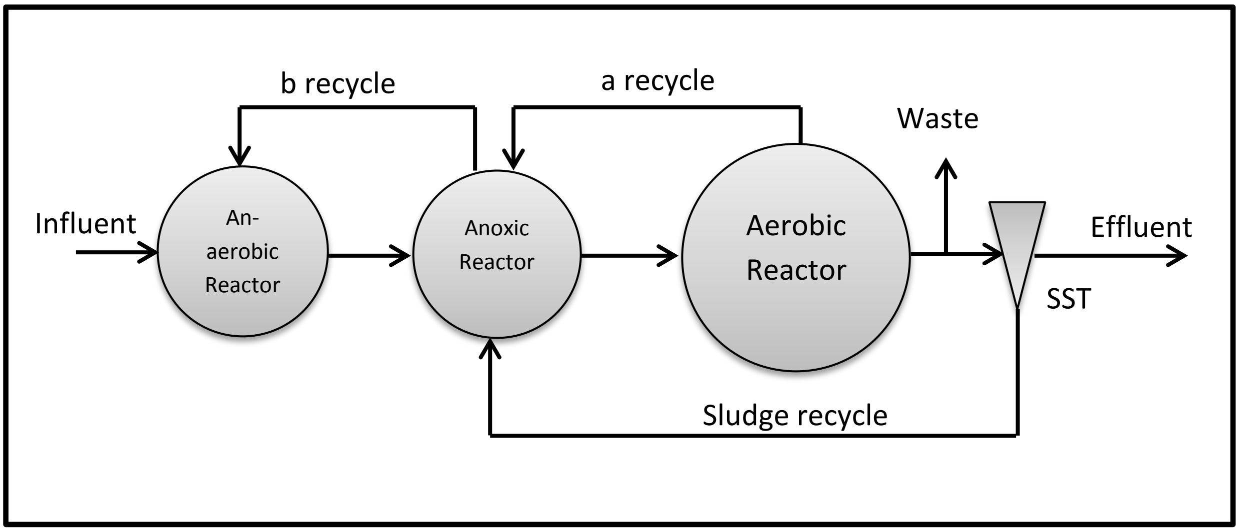

Figure 7: Schematic setup of UCT WWTP system [adapted from Mbaya, 2011]

The UCT system is simple to model and achieves the desired goals of ND and EBPR and is thus a good reference setup for this research. Also, most WWTPS can be setup to operate as a UCT system (Mbaya, 2011), so it is certainly worth investigating. This system is a NDEBPR system that has three basic reactors, one of which is aerated (aerobic reactor) and two are unaerated (anoxic and anaerobic reactors). This system allows for aerobic growth of OHOs and nitrification (by ANOs) in the aerobic reactor, denitrification in the anoxic reactor and EBPR in the anaerobic reactor. (This research modelled the varying degrees of urine separation in a simulated UCT system – see Section 3.3.4 “WWTP Model: Setup, Explanation and Assumptions Made” for further details.) Table 2.2 on the next page is a good aid in explaining the different biological activities in the different zones of the UCT WWTP system.

Table 2.2: Summary of the Organism Groups, their Biological Processes and the respective Zones of the WWTP where these functions are utilized (Adapted from Henze et al., 2008)

Organism Group	Biological Process	Zone
Ordinary heterotrophic organisms (OHOs), which are unable to accumulate polyphosphate	COD removal and ammonification (organic degradation, release of organic N as Ammonia, NH_4^+) Denitrification - removal of N via liquid-to-gas conversion (organic degradation, ammonification, reduction of Nitrate Nitrite – $NO_3^- \rightarrow NO_2^- \rightarrow N_2$) Fermentation (conversion of FBSO to VFA)	Aerobic (presence of dissolved oxygen and Nitrate/Nitrite) Anoxic (zero DO but presence of Nitrate/Nitrite) Anaerobic (zero DO or Nitrate/Nitrite)
Autotrophic nitrifying organisms (ANOs)	Nitrification - removal of Ammonia ($NH_4^+ \rightarrow NO_2^- \rightarrow NO_3^-$; DO uptake)	Aerobic
Phosphorous accumulating organisms (PAOs) which can accumulate polyphosphate.	P release – VFA uptake; PHA storage. P release – VFA uptake; P storage. P uptake – PHA degradation. P uptake, P removal – PA degradation; DO uptake.	Anaerobic Anoxic Aerobic

2.6 Using Wastewater Treatment to Move Towards a More Sustainable Future

The terms 'sustainability' and 'sustainable development' may have different meanings to different people, at different times and in different fields. While these terms may be vague in broader society, it is accepted that in the wastewater treatment field, some good indicators of moving towards more sustainable operations are: reducing fresh water consumption, improving effluent quality from WWTPs, recovering and reusing nutrients such as N, P, K and Mg, and decreasing energy requirements at WWTPs (Ekama, 2011b). When issues of human health are at hand, and the very direct impacts of damaging delicate aquatic systems are at stake, then reductions in CO_2 emissions should not take priority. While obviously trying to develop systems that reduce CO_2 emissions or reuse as much methane as possible to power the operations, the goal of reducing GHG emissions should not result in sacrificing the other important objectives of WWTPs.

2.6.1 Saving Water

South Africa is a water-scarce country, as shown in Figure 8 on the following page (where dark yellow represents a country/region where there is 'stress' on the freshwater resources).

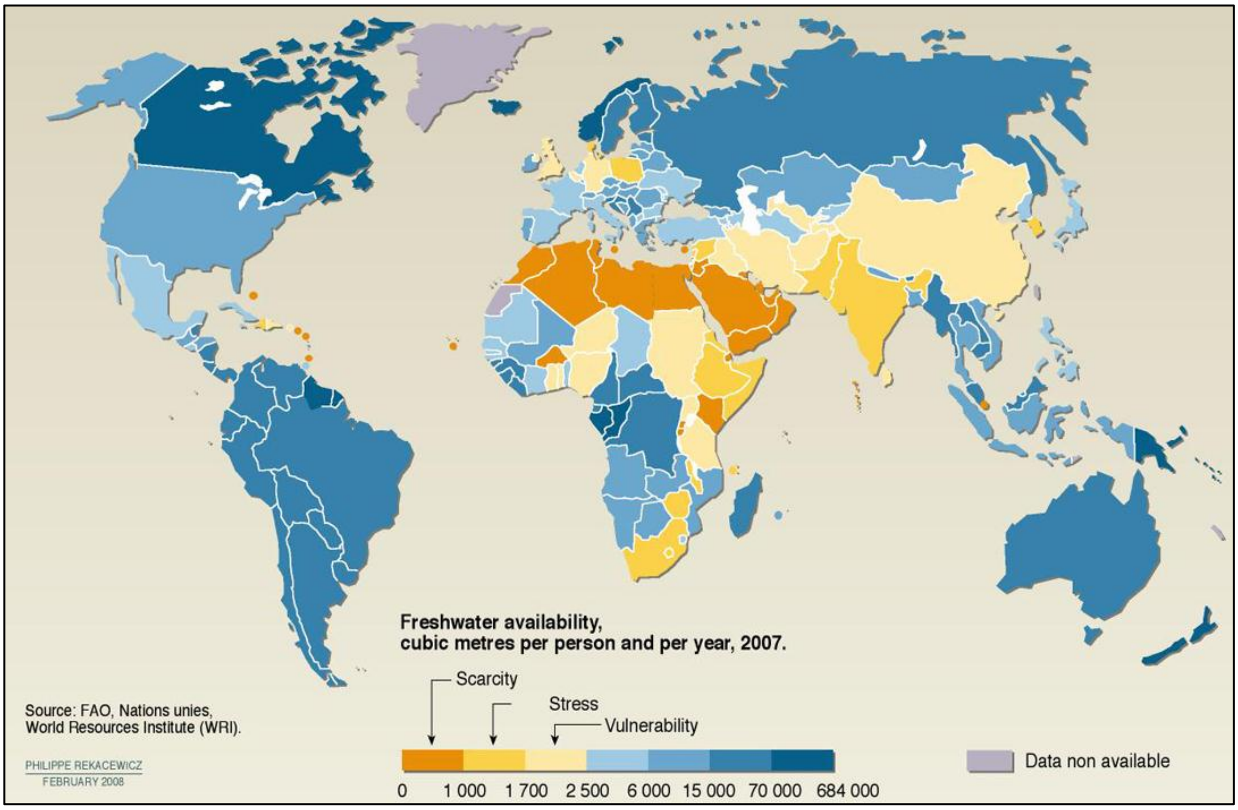

Figure 8: Map showing estimated global freshwater availability [United Nations, 2008]

Despite being a water-scarce country, over 70% of residents in South Africa receive potable water that is safe for human consumption (StatsSA, 2009). A lot of effort, in terms of both energy and infrastructure, is used to ensure that the water that South Africans receive is of drinkable quality. Rapid urbanisation and limited viable space for new dams has put strain on the water resources of the country. An example of this is Johannesburg, where the municipality has to import water from Lesotho (Lesotho Highlands Project). There are many possible options to maximise water use without impacting on our natural water resources, such as rainwater harvesting and greywater reuse programs, and further explanations of these can be found in Armitage et al. (2007).

In coastal areas where potable water is needed, desalination (converting sea water to drinking water) is an option, as evidenced by the 15Mℓ/day desalination plant that was constructed in Mossel Bay in 2011. This technology is seen as unsustainable by many due to its high energy consumption (energy that is largely generated from fossil fuels in South Africa), giving it both high costs and contributing to CO_2 emissions. An alternative to 'producing' freshwater in this way is to conserve the freshwater that consumers already use. While strategies like campaigns to get the public to save water and implementing water restrictions have their place, there are other proactive approaches that municipalities can adopt to encourage water saving. Some of these include promoting the use of dual-flush toilets, waterless urinals or low-flow urine separation toilets, which reduce the quantity of flush-water used. Another of these ideas to save water is to implement the use of seawater to flush toilets, which will be discussed in Section 2.8.2 "Seawater Toilet Flushing" further on.

2.7 Status of WWTPs in South Africa and Motivation for Improving WWTPs

2.7.1 Status of WWTPs

According to the 2012 Green Drop Progress Report by the Department of Water Affairs and Forestry, there are currently 831 WWTP facilitates in South Africa (DWAF, 2012). Of these 831 WWTPs, the percentage split between different sizes of WWTPs is shown in Figure 9 below.

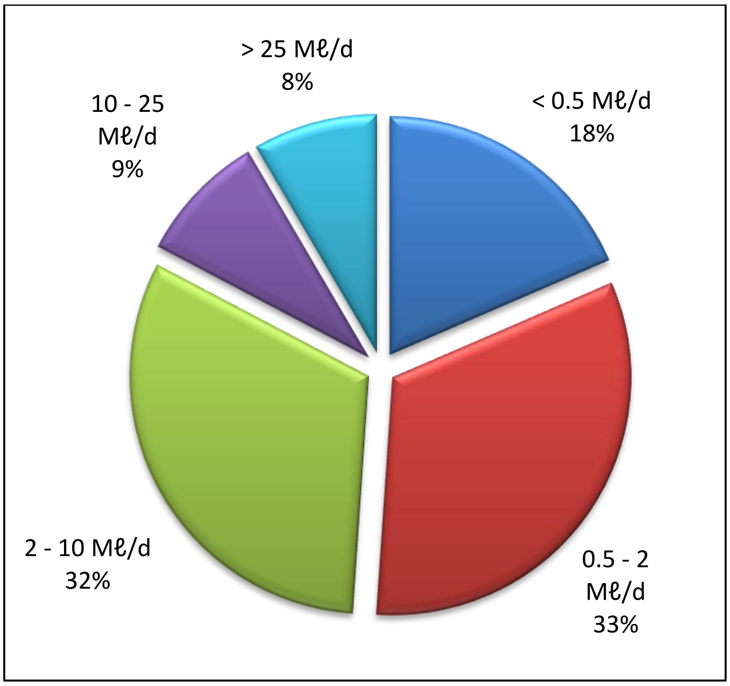

Figure 9: Percentage make-up of WWTPs in South Africa based on average daily influent flow [adapted from DWAF, 2012]

As shown in Figure 9 above, micro size plants (treating less than 0.5Mℓ per day) constitute approximately one fifth of all treatment plant facilities in South Africa. This provides the perspective that in terms of selecting appropriate technology, management, operational and maintenance support, the numerous micro plants should not be neglected (DWAF, 2011). Small plants (treating between 0.5 and 2Mℓ per day) are also numerous, and make up around a third of all wastewater treatment plants in South Africa. This again constitutes a large number of plants which fill a specific make in terms of management, operations and maintenance. The medium (2 - 10Mℓ per day) constitute about another third of the total WWTPs, while the large (10 – 25Mℓ per day) and macro plants (more 25Mℓ per day) make up the other sixth of the wastewater treatment facilities in South Africa. The large and macro plants would typically have access to better management, operations and maintenance resources (DWAF, 2011a). Importantly, while the macro-sized plants make up only 8% of the total number of WWTPs in South Africa, they account for roughly two-thirds of the total wastewater flow treated in this country (DWAF, 2011b).

In terms of the provincial spread of WWTP facilities, the following conclusions can be made:

- The Western Cape's spread of wastewater treatment plant sizes is similar to the national situation (DWAF, 2011a). The Western Cape accounts for 155 out of the 831 WWTPs (19%) in South Africa and makes up 16% of the national daily operational flow (DWAF, 2011b).
- Gauteng province has a relatively high number of medium and large WWTPs, with fewer micro and small size plants (DWAF, 2011a). Gauteng accounts for only 7% of the national WWTPs but handles 49% of the national daily operational flow (DWAF, 2011b).
- The Eastern Cape, Northern Cape, Mpumalanga and Limpopo provinces mainly have micro and small size plants (DWAF, 2011a).
- The other provinces, including North West, KwaZulu Natal and Free State have a wider spread of WWTP sizes across all the plant size categories (DWAF, 2011a).

The overall performance of South Africa's WWTPs, according to DWAF (2012), is summarised in Figure 10 below, where the WWTPs are grouped into performance percentage categories with simple descriptions (i.e. 'Excellent', 'Good', 'Average', 'Very Poor' and 'Critical State') for easy interpretation.

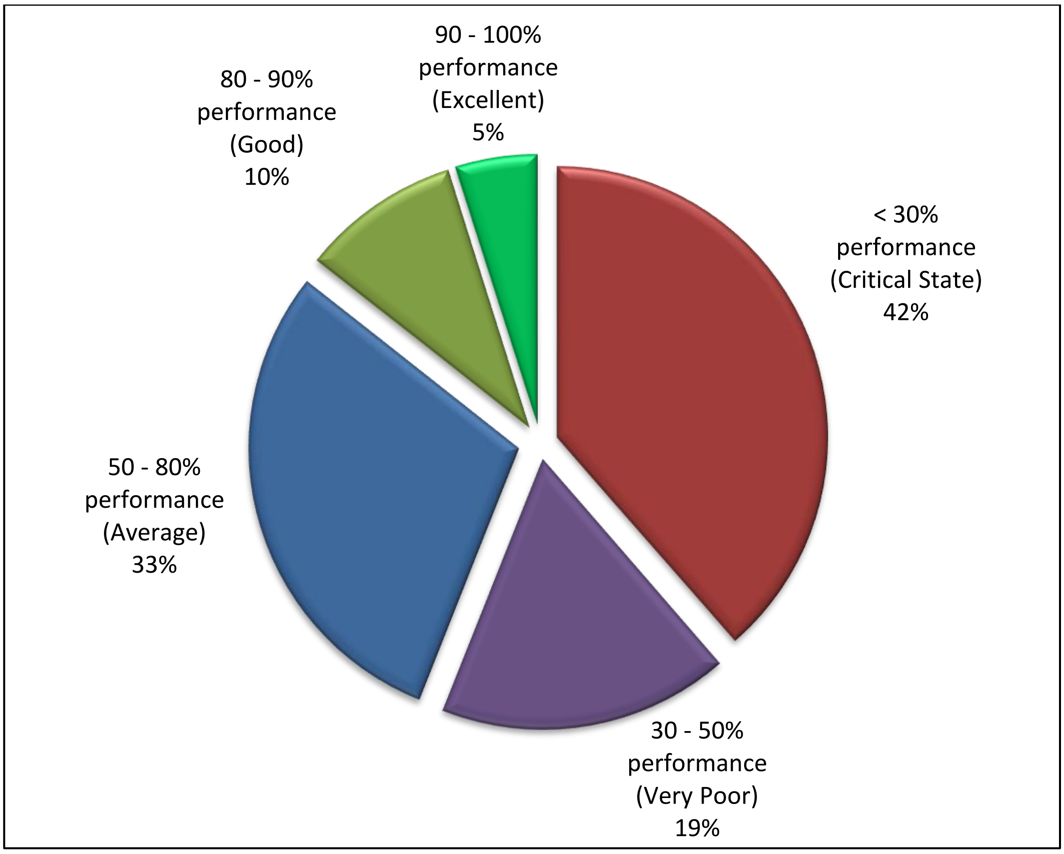

Figure 10: Assessing the performance of South African WWTPs by placing them in different performance categories [adapted from DWAF, 2011b]

The overall impression from Figure 10 above is that 61% of South Africa's WWTPs are deemed to be operating below 'Very Poor' performance. This is clearly cause for concern. What is both simultaneously encouraging and discouraging is that this dire situation actually represents an *i*mprovement from the previous Green Drop Report. The data in the 2012 Green Drop Progress Report shows that the country as a whole has improved in the effluent quality and technical skill categories in comparison to the previous year (DWAF, 2012).

While mainly focussing on the effluent performance of WWTPs, the Green Drop Report also assigns a risk-based rating for each plant. This allows the municipalities to identify and prioritise critical risk areas and take measures to mitigate these risks. This is done by calculating a Cumulative Risk Rating (CRR) for each plant, which incorporates compliance in terms of technical skills, effluent quality and includes whether the operational flow is above or below the design capacity, as given by Equation 4 below (DWAF, 2012):

$$CRR = A \times B + C + D \qquad \textbf{Equation 4}$$

Where:

A = Design Capacity of plant

B = Operational flow

C = Number of non-compliance trends in terms of effluent quality

D = Compliance or non-compliance in terms of technical skills

As shown in Figure 11 below, the CRR ratings of WWTPs in SA had generally risen (become worse) from 2008 to 2011. This is evidenced by the fact that there are fewer WWTPs in the "low risk" category and more in the "high risk" and "critical risk" categories in 2011 when compared to 2008. This presents a worrying trend and reveals that even though WWTP performance (mostly based on effluent quality) may have improved (DWAF, 2012), there are other risk factors that are making the overall trend of South Africa's WWTPs a negative one.

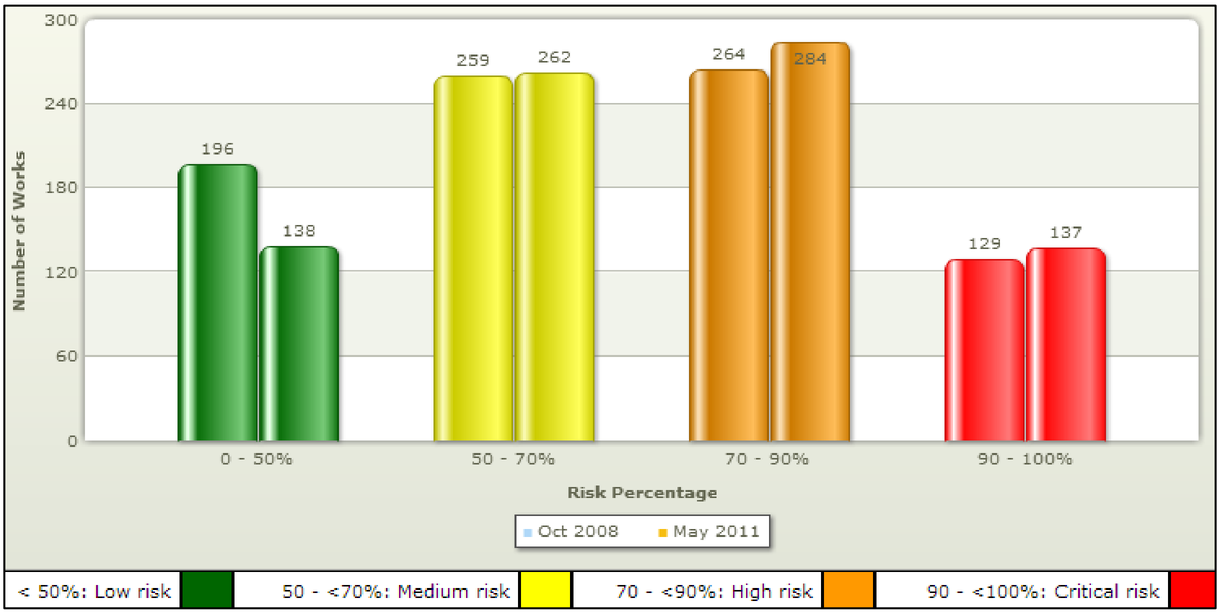

Figure 11: CRRs of all South African WWTPs in 2008 and 2011 [Source: DWAF, 2011b]

An interesting observation is that the national trends are by no means evenly distributed around the country, and there are definitely areas that are more critical than others. This spatially-unequal phenomenon is shown in Figure 12 below, where the provincial performance profiles are the summation of the respective municipal performances (DWAF, 2011b).

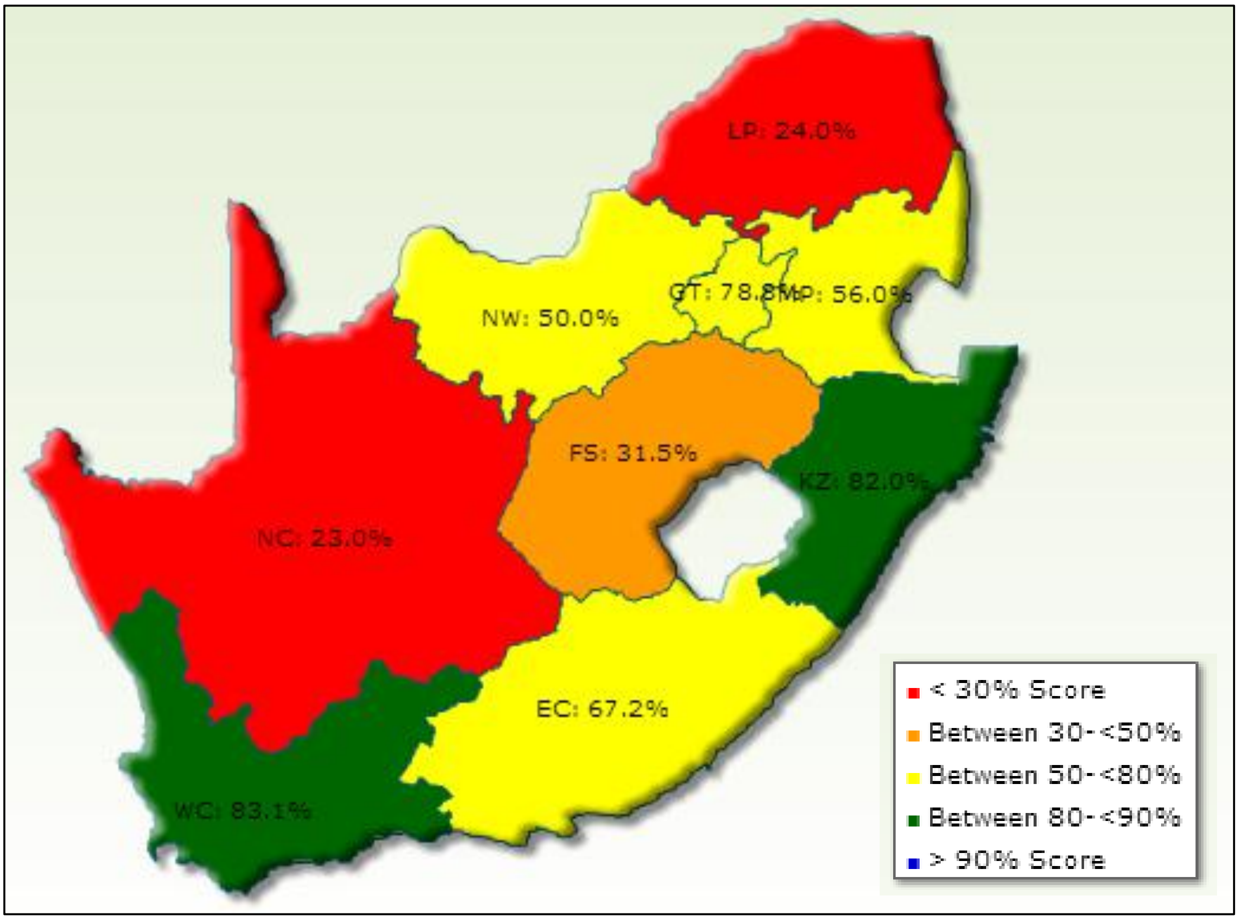

Figure 12: Provincial distribution of SA WWTP performance [source: DWAF, 2011b]

Clearly, there are regions that for political, social, spatial, technical or other reasons generally have better performing WWTPs than other regions. The Western Cape has the highest-performing WWTPs, and also the highest percentage of Green Drop Certification for its WWTPs (DWAF, 2011a).

Various conclusions regarding the state of the nation's WWTPs and wastewater treatment industry will now be discussed. Some of the detailed information was only readily available for the Western Cape's WWTPs (DWAF, 2011a), but because this province is the best-performing province, it can be inferred that worse conditions exist throughout the rest of the country.

2.7.2 Motivation for Improving SA's WWTPs

Based on the overall status of SA's WWTPs as highlighted above, there is definitely cause for concern regarding the state of the wastewater treatment industry in SA. The current status of

SA's WWTPs should provide enough motivation and justification for improving SA's WWTPs. The specific problems with SA's WWTPs are outlined below in Sections 2.7.2.1, 2.7.2.2 and 2.7.2.3, and the possible solutions to these problems are highlighted in Section 2.8 "Innovative Technological Options for Improving WWTPs". The problems highlighted below serve to provide important motivation for investigating the idea of urine separation as a method of improving WWTPs further on in this report.

2.7.2.1 Many WWTPs are Operating Over-Capacity

The following relevant findings from DWAF (2011a) are presented with respect to the Western Cape's WWTPs and their capacities:

"In the Western Cape in 2011, 13 of the 156 WWTWs (8%) operated at maximum hydraulic design capacity (>95% of design flow). 46 of the Western Cape's 156 WWTWs (29%) potentially operated beyond design capacity (in excess of 150% of design capacity). 6 out of 156 WWTWs (4%) are approaching their maximum capacity (close to 90%) and need to start planning for extension and upgrades over the next 1-5 years."

Clearly then, the capacity of many of the Western Cape's WWTPs are a problem when compared to the operational flows that they receive. Nationally, this problem is most certainly more dire than in the Western Cape. Increasing the capacity or decreasing the influent flows received by the WWTPs would assist in alleviating this problem.

2.7.2.2 Effluent Quality Needs to Be Improved

In the Western Cape in 2011, it was found that 30 of 156 WWTWs (19%) show non-compliant trends in 3-9 effluent quality parameters, and this does not include plants with "no information" (so the number of plants showing effluent non-compliance could actually be worse) (DWAF, 2011a). Interestingly, no direct link could be made between effluent non-compliance and plant flow capacity 'exceedence' (DWAF, 2011a). Clearly then, there could be other factors influencing the effluent quality. One of these possibilities is the operational complexity (and lack of necessary skills) of the WWTPs, as discussed in Section 2.7.2.3 below.

Currently, no (South African) effluent regulations exist for micro-pollutants such as environmental oestrogens (EOs) or endocrine disruptors (EDs). The release of hormones, medicine residues and pharmaceutical compounds (generically called EOs or EDs) have caused severe disruption in the environment affecting the gender of fish and reptiles in receiving water bodies and is believed to be detrimental to human health in the long term (Cadbury, 1997). The removal of these compounds is becoming increasingly important for environmental sustainability and human health (via drinking water), and will become even more important if wastewater reuse is adopted and WWTP effluent is reused in more applications.

Clearly then, there exists the need to improve the (traditional) effluent quality of South African WWTPs. Also, there is an opportunity to be forward-thinking and provide solutions to deal with the issue of removing/reducing EOs and EDs from WWTP effluent even before regulation is created that stipulates this (which may happen in the future).

2.7.2.3 WWTP Operational Complexity is a Major Problem

It was found by DWAF (2011a) that in the Western Cape, disinfection and nitrification remain the process areas with the highest evidence of non-compliance (as indicated by the E. coli and faecal coliform results and Ammonia in the effluent). The fact that nitrification is not being performed shows a distinct possibility that the plants are not being operated properly. As explained in Section 2.7.2.2, the problem of effluent quality is not always due to plants running over capacity, but may be due to other factors such as suboptimal operation, as explained by DWAF (2011b), "In many cases, WWTWs which are under hydraulic (overloaded) stress performed better than plants with sufficient capacity, whereas many of the WWTWs with sufficient plant capacity do not comply with effluent standards ... This leads to the conclusion that other factors are responsible for non-compliance, including the skills and experience, correct proportioning, and ongoing training of the operational staff and maintenance team."

While DWAF (2011b) found that technical skills at WWTPs are improving, they also found that there is a still a significant skills shortage – which is contributing to the problem of poor WWTP performance in South Africa. This skills shortage and the subsequent effects are clearly stated by DWAF (2011b), "A concerning factor is that high percentages of personnel employed in 'skilled' positions do not comply with the requirements for supervisors and process controllers. These numbers, combined with the number of vacancies in these positions, amount to a significant number of positions that are not filled by any form of skill or by inadequate/inappropriate skill." No matter how well engineered or designed a system is, it can only function effectively with good operation, maintenance and management.

Training programmes and skills development courses could be utilised to close the gap between the current available skills and the skills required to effectively operate the complex BNR plants. Another way to address this problem is to approach it from the design side, by designing WWTPs that are less intricate and therefore simpler to operate. Obviously every attempt to design uncomplicated WWTPs is being made, so a complete change in the approach to wastewater treatment would be needed to make the BNR WWTPs simpler than they currently are without compromising on effluent quality. Ideas that propose a complete change in traditional wastewater treatment approaches is presented Section 2.8 "Innovative Technological Options for Improving WWTPs".

Another pertinent problem facing WWTPs in South Africa is a lack of funding and administrative support. This is a more economic and politically based problem, but could also be addressed from a design perspective if WWTPs could be improved to make their operation cheaper. If improvements could be made to make WWTPs cheaper to build and operate, this would aid in greatly reducing the financial constraints facing WWTPs in South Africa today.

2.8 Innovative Technological Options for Improving WWTPs

As can be clearly seen from Section 2.7.2 "Motivation for Improving SA's WWTPs" above, there is motivation and a need to improve South Africa's WWTPs. While many of the problems could be solved with better operation, maintenance and management of existing WWTPs, there exists the opportunity to eliminate some of these problems entirely by changing the composition and delivery of the influent wastewater received by WWTPs.

While there are numerous ways to possibly alter and improve the operation of BNR WWTPs, one such 'outside the box' idea is to separate urine from the wastewater stream that ends up being treated at the wastewater treatment plants. Source separation of urine could reduce the nutrient load and hydraulic load on WWTPs. Another approach is to use seawater to flush domestic toilets, which will save freshwater and significantly change the characteristics of conventional BNR WWTPs. A further innovative approach is to combine these two technologies and implement them in a way that complements and enhances these two individual technologies. This section will look at these two technologies and the possibility of implementing them together.

2.8.1 Urine Separation

Urine is estimated to contain around 80% of the Nitrogen and 50% of the Phosphorous as well as 67% of the medical residues contained in the wastewater stream (Otterpohl 2002), while contributing only 1% of the volume (Wilsenach, 2006). Urine contains the bulk of the nutrients that are required by legislation to be removed from the wastewater at WWTPs, and also much of the micro-pollutants such as endocrine disruptors and medical residues. Most of the Nitrogen in urine is present as urea ($CO[NH_2]_2$) which rapidly hydrolyses to Ammonia (NH_4^+) and bicarbonate (HCO_3^-) in wastewater (Wilsenach 2006).

Table 2.3: Composition of urine, faeces and total wastewater load (including urine) per person (STOWA, 2002 as cited in Wilsenach, 2006)

	Nitrogen (gN/p.d)	Phosphorous (gP/p.d)	COD (gCOD/p.d)	Volume (ℓ/p.d)
Urine	12	1.0	12	36
Faeces	41	1.4	0.7	10
Total Wastewater	15	2.4	161	300

Table 2.3 above shows an estimate of the daily Nitrogen, Phosphate, COD and Volume contributions per person with regard to urine, faeces and the total wastewater produced per day in Holland. While European WW is generally more dilute as it contains all the stormwater as well, similar urine and faeces trends are expected in South African WW If urine can be partially or completely separated from the main wastewater stream, the removal of much of the nitrogen and phosphorous could result in significant changes to BNR WWTP systems. Particularly, separation of urine could potentially remove the need for WWTPs to perform excess nitrogen removal, that is nitrification and denitrification (ND). The selected sludge age

of the WWTP is the size-determining factor for WWTPs based on the UCT setup (as well as many others) (Ekama, 2011a). Where nitrification is required, it is the nitrification sludge age that is the determining factor of the system sludge age required at WWTPs (Ekama, 2011a). To guarantee nitrification for removal of Ammonia, long sludge ages of between 20 to 25 days are suggested (Ekama, 2011a). Due to the relationship between sludge age, reactor volume and sludge wasted per day, this means that in practice large biological reactors are required. If urine can be separated and the need to nitrify can be eliminated, WWTP sludge ages can be made shorter and hence reactor volumes can be reduced, or WWTPs can handle larger influent capacities. Aerobic nitrification requires mechanical aeration of the biological reactor. Aeration of the reactor requires large electricity inputs, and these electrical costs form a major portion of the operating costs of a WWTP. With either a capacity increase or a decrease in reactor size, there would also be a reduction in operating costs due to the reduced aeration costs (as a result of less nitrification) and power consumed.

There is also compelling evidence to indicate that reducing the influent nutrient loading on WWTPs by urine separation will reduce the nutrient concentrations in the WWTP effluent stream, as found by both Mbaya (2011) and Wilsenach (2006). This has positive impacts for the receiving water bodies, ensuring that the nutrients that cause eutrophication and deoxygenation are released in low concentrations. Also, endocrine disruptors and medical residues are concentrated in urine and could be treated in decentralised urine treatment facilities, preventing widespread release into the environment, which appears to have many dangers to nature as well as to humans (Cadbury, 1997).

2.8.2 Seawater Toilet Flushing

A system of using sea water to flush toilets could potentially have a significant effect on freshwater resources in urban environments. This system would use seawater to replace the freshwater in toilet cisterns, and would prevent potable water from being used to flush away faeces and urine, saving water on a widespread level. With flush-water making up 20-30% of the (South African) domestic wastewater total, there is potential to save up to 20-30% freshwater by implementing seawater flushing technology and infrastructure.

In Hong Kong, seawater has been used to flush toilets for four decades in an attempt to conserve freshwater (Chau, 1993). Sea water is distributed for toilet flushing (WSD, 2012), and around 80% of all water users in Hong Kong currently use sea water to flush toilets (WSD, 2012). Hong Kong has a dual distribution system for potable water and seawater and can provide the seawater for free by taxing freshwater consumption (WSD, 2012). According to WSD (2012), in 2011 an average of 740Mℓ per day of seawater was supplied for flushing purposes, conserving an equivalent amount of potable water. This is a large saving considering that all Hong Kong’s freshwater is imported and has to be transported long distances and at high cost (Chau, 1993).

From a WWTP point of view, there are potential benefits to using sea water to flush toilets as well. Sea water contains a high concentration of sulphate. There are wastewater treatment systems being developed (such as the SANI process briefly shown in Figure 13 below) that utilise biological sulphate reduction (using the sulphates in seawater) to break down the organics (COD) in the wastewater, eliminating oxygen demand requirements and producing low sludge quantities by utilising anaerobic bioprocesses (Ekama 2011b).

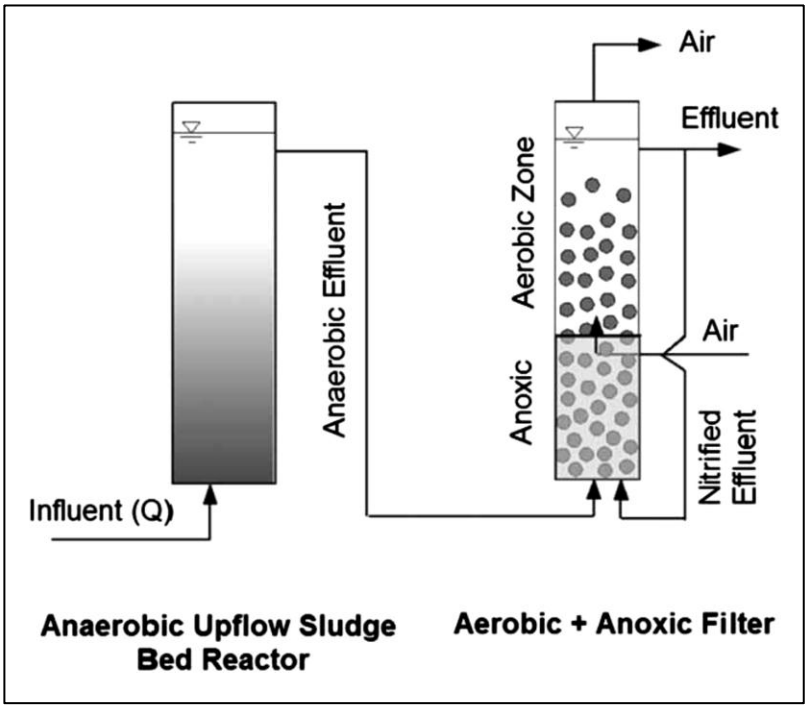

Figure 13: SANI process flow scheme [Ekama, 2011b]

There are, however, practical issues associated with this seawater flushing technology that need to be considered, such as the complexities of introducing a dual reticulation system to deliver the sea water and drinking water to consumers. Putting these practical implementation issues aside, there are also technical issues associated with using this technology. One such problem has to do with the high sulphate concentration in sea water. The sulphates contained in sea water would present problems in concrete sewers because they can be converted to sulphuric acid on the crown (top) of the inside of concrete sewers, exacerbating crown corrosion (Ekama, 2011b). Ekama (2011b) explains this process in detail: "In the presence of organics (electron donors), sulphate-reducing bacteria produce sulphide gas which escapes to the head space above the water in the sewer. Sulphide-oxidising bacteria on the upper walls of the sewer oxidise the sulphide to sulphuric acid in the presence of oxygen ($H_2S + 2O_2 \rightarrow H_2SO_4$), which corrodes the crown of the sewer." This corrosion of the concrete can cause pipes to cave in and fail well before their design life has been reached. Another disadvantage of sea water flushing and saline sewage treatment is that biological excess P removal cannot be included in the system (Ekama 2011b), and there is a problem with the effluent possibly being too saline for discharge into rivers – necessitating further treatment before discharge.

2.8.3 Combining Urine Separation and Sea Water Flushing

While the merits of urine separation as an isolated technology have been briefly discussed above, there are greater possibilities for combining it with other technologies. This is where the idea of dual implementation of seawater flushing and urine separation has potential success. Ekama (2011b) concludes that: "Combining source separation of urine and saline water toilet flushing can reduce sewer crown corrosion and reduce effluent P concentrations". If urine diversion is implemented in urine diversion toilets that flush faeces with seawater, then the urine can be collected and nitrified decentrally, during which excess P can be recovered from the concentrated urine stream. The nitrified liquor could then be discharged to the sewer with the rest of the wastewater stream, and the Nitrate in the sewer would decrease biological sulphate reduction (as it is a more readily used 'substitute' for sulphate in a sense), hence reducing sulphuric acid and crown corrosion (Ekama, 2011b).

In essence, the sewer network would then become part of the wastewater treatment system, as denitrification and removal of some organics would already begin to take place within the sewer pipes. Because urine is estimated to contain 50% of the P, 80% of the N and 67% of the medical residues, and because this urine could be processed decentrally (removing all or most of these constituents), the SANI WWTP systems would be able to release effluent containing half the P and only one third of the medical residues compared with not implementing source separation of urine (Ekama 2011b). So the dual-implementation of these two technologies would allow an anaerobic WWTP system that produces low sludge quantities, does not require costly aeration, has low effluent P, N and medical residue concentrations, prevents excess crown corrosion and preserves valuable freshwater resources.

Interestingly, this technology could possibly be implemented in some inland areas where access to sea water is restricted. In essence, there is evidence to suggest that acid mine drainage is a potential water source that with some treatment (and dilution with grey and brown municipal water), could exhibit the required sulphate concentration levels to be implemented in the SANI system illustrated earlier (Ekama, 2011b). In effect then, the acid mine drainage runoff could be used as a saline substitute for sea water in inland areas. Obviously this would require heavy investment into the mine drainage system to make use of this runoff, but it could help solve acid mine drainage pollution problems while also improving WWTP systems in certain inland areas.

The benefits of combining urine separation with seawater flushing have been explained above, but there is still an issue regarding the effluent discharge from a saline sewage plant. Because of its salinity, this effluent may not be suitable for direct discharge into river systems or for reuse in irrigation schemes, but would be appropriate for coastal locations where WWTPs discharge into the sea. If the salinity of the effluent needs to be reduced for discharge into river systems, desalination could be used. Desalinating saline sewage effluent via reverse osmosis would be cheaper than directly desalinating drinking water from the sea, as the salinity of treated saline sewage is only about one-third of that of sea water, and the life of the membranes in the desalinator units is inversely related to the salt concentration across the membranes (Ekama, 2011b). This gives the membranes a much longer life and gives the desalination units lower operating costs if used in this sort of post-treatment system compared to direct desalination treatment for drinking water.

Table 2.4 below briefly compares the aforementioned technologies individually and in combination with each other, with the Conventional BNR WWTP setup being the benchmark for comparison. This table is worth considering at length, as it summarises and highlights the potential benefits and drawbacks of urine separation, seawater flushing and a combination of the two.

Table 2.4: Comparison of the impacts on conventional, sea water toilet flushing and source separation of urine strategies on the urban water cycle (Ekama, 2011b)

Criterion	1. Conventional	2. Seawater Flushing	3. Urine Separation	4. Combination of (2) and (3)
Distribution	Single	Dual	Single	Dual
Collection	Single	Single	Dual	Dual
Sewer Corrosion	Normal	High	Normal	Normal
Energy Demand	High/V. High	Very Low	High	Low
Sludge Production	High	Very Low	High	Very Low
Sludge Age	Long	N/A	Low	N/A
Reactor Volume	Large/Small	Large	Small	Large
Sludge Treatment	High	None	High	None
Energy Recovery	Yes	No	Yes	No
Nutrient Recovery	Yes	No	Yes	Yes
Effluent Quality	Very Good	Fair	Good	Good
N & P Removal	Yes	No P Rem.	Not Required	No P removal
Effluent N and P concs	Low	High P	Low	Some P
Effl. Salinity	Low	High	Low	High
Effl. Susp Solids	Low/V. Low	High	Low	High
Effl. Pathogens	High/Low	Low	High	Low
Effl. ED & EO’s	High	High	Low	Low
Water Saving	No	Yes	No	Yes

Methane recovery and electricity generation at WWTPs is a common practice at BNR WWTPs, although this is often underused at WWTPs and should be promoted further. Interestingly, this energy recovery cannot be achieved at WWTPs where sea water flushing (only) has been implemented, or where sea water and urine separation have been combined. Energy can however still be recovered at WWTPs where only urine has been separated from the wastewater influent.

While there are quite clearly benefits to implementing these technologies in a dual way, this thesis is primarily focussed on the idea of urine separation as a stand-alone option. The background to this investigation is indeed contained within broader solutions to water conservation and an integrated move towards sustainability. However, this thesis investigates the merits of urine separation as a technology on its own. This system could practically be implemented before the seawater flushing system is implemented, or could be implemented country-wide as a base technology while being combined with seawater flushing in coastal (and other applicable) areas of South Africa. It is for this reason that this technology should have stand-alone benefits for implementation, and hence this thesis investigates the stand-alone impacts of urine separation on WWTPs.

2.9 Investigating the Impacts of Urine Separation as a Stand-Alone Option

Urine separation can have many intended benefits and implications. Urine separation from no-mix toilets (and hence faecal separation as well) can be implemented in low-income communities where offsite waterborne sanitation is not considered viable or possible (an example shown in Figure 14 below). This allows for separate storage, collection and treatment. Urine separation can be implemented in rural communities for the same reasons, or for the purpose of separating urine and faeces in order to effectively compost and dry faeces and recycle the nutrients to an agricultural environment.

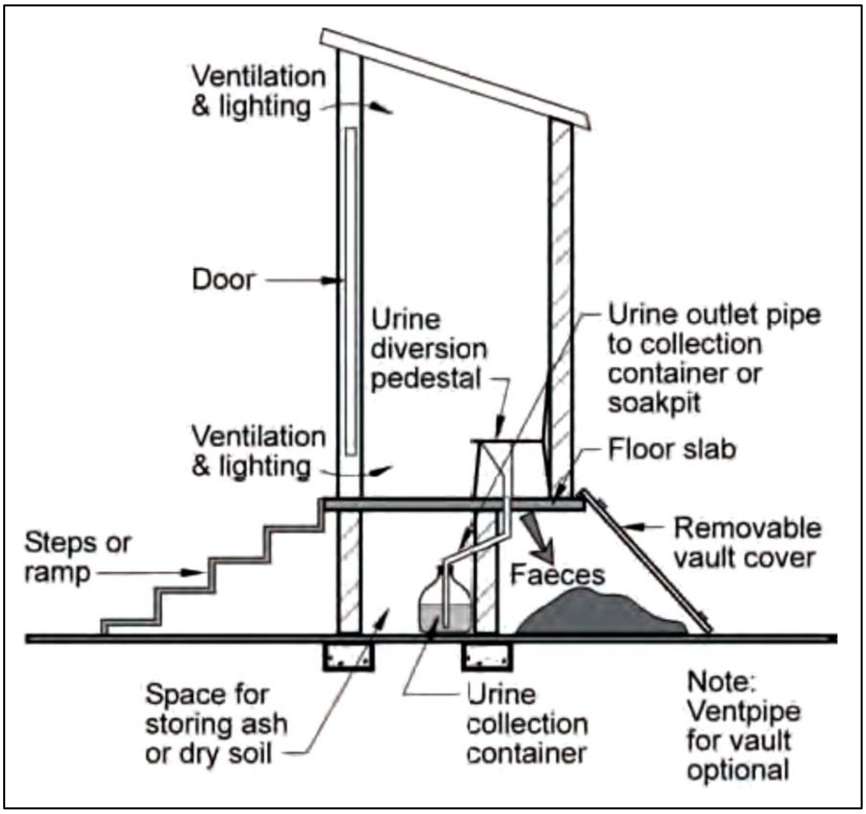

Figure 14: Urine separation in applications where wet sewerage is not available and composting of faeces is encouraged (i.e. rural locations) [Source: CSIR, 2000]

While acknowledging these other potential reasons for implementing urine separation and investigating them briefly, this research aims specifically at investigating the impact of urine separation on waterborne sanitation and the end of pipe WWTPs. This implies that this research is based on investigating urban environments like cities, where people are connected to a piped sewer network and where centralised WWTPs handle the wastewater flows.

When some of the key objectives of wastewater treatment are to remove the N and P nutrients from the wastewater, it seems counter-productive to dilute this concentrated nutrient stream (urine) into the large grey and brown wastewater flow, only to have to remove the nutrients again in a much more diluted form at the wastewater treatment plant. It is the historic development of waterborne sanitation that has led to the current situation, and now a considerable effort would be required to break away from this established convention. Therefore there needs to be considerable benefits to the idea of urine separation in order to make large-scale urine separation a viable idea.

2.9.1.1 Broader Impacts

This section will highlight the potential broader impacts of urine separation to society, excluding the primary impacts on the actual WWTPs, which will be discussed later.

2.9.1.2 Reduction in Water Consumption

Source separation of urine can be expected to result in many benefits. One of these is a reduction in water consumption. Around 20–30% of all domestic water is used to flush urine down the sewers (Ekama, 2011b: 1311). It is estimated that urine flushes are 5 to 10 times more frequent per person than faeces flushes. If undiluted urine can be separated at the source via waterless separation systems, this will lead to a reduction in water used per person. Overall then, this could lead to a significant freshwater savings.

The implementation of low-flow toilets and dual flush toilets (where lower quantities of water are used to flush away urine compared to faeces flushes) can help in reducing this portion of domestic water use. Source separation of urine will lead to a reduced hydraulic load at WWTPs (or at least a more gradual increase in hydraulic loading on WWTPs as cities expand and populations grow).

2.9.1.3 Possible Nutrient Recovery

Ammonia, which is a nitrogenous compound used in fertiliser, is produced efficiently using the Haber process ($N_2 + 3H_3 \rightarrow 2NH_3$). As a result of this efficient and relatively cost effective method of Ammonia production, the recovery of Nitrogen from urine for use as fertiliser does not presently seem to hold many economic benefits, especially in first world countries (like Sweden and the Netherlands) (Wilsenach, 2006). Nitrogen is a non-finite reserve in this sense, as it can be harvested indefinitely from the atmosphere. In terms of a sustainable "closing the loop" ideology however, Nitrogen could be recovered and used as fertiliser, especially in developing, agricultural countries. In this way, an attempt could be made to act more sustainably and recycle some portion of Nitrogen instead of adding more anthropogenic Nitrogen to the natural Nitrogen cycle through the Haber process.

In contrast to Nitrogen, rock-Phosphorous (also used in fertiliser) is a finite resource, which has a market value that will no doubt rise as reserves dwindle. An 8 year history of rock phosphate prices shown in Figure 15 confirms this theory. There is interest in harvesting Phosphorous from wastewater (particularly from urine) to supplement and replace some rock-phosporous. Ekama (2011), in citing Jiang et al. (2011), explains that “P can be recovered from urine by precipitating the phosphate as struvite with $Mg(OH)_2$ dosing”. Different forms of struvite can be precipitated from urine, and they hold potential as a slow-release fertiliser without the drawbacks of the current methods of precipitating phosphorous from wastewater with iron and aluminium salts, which makes this form unsuitable for agricultural use. At current extraction rates, reserves of rock phosphate that are economically recoverable with today’s technology will last less than 100 years, although there are reserves that need further technological advances to exploit (Driver, 1999 as cited in Wilsenach, 2006).

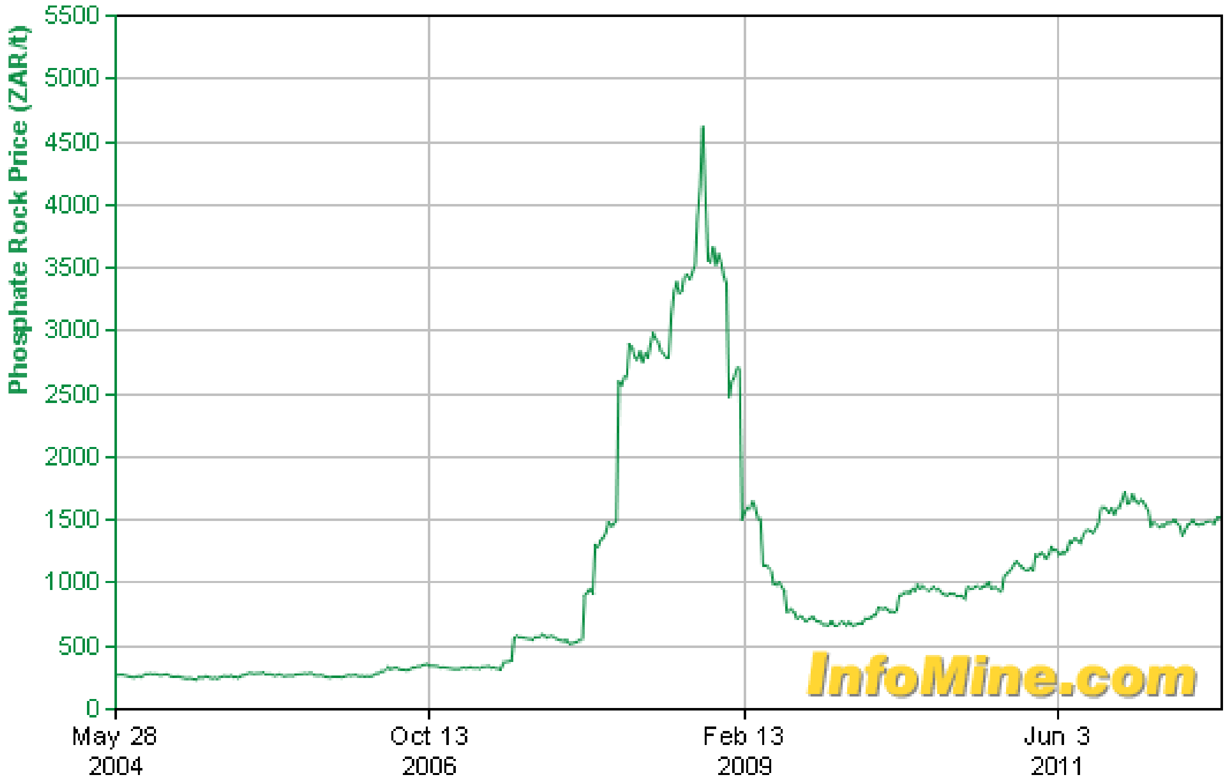

Figure 15: Eight year trend of Phosphate rock prices in Rands/tonne (Source: InfoMine.com)

Other finite resources that could be recovered are Potassium salts and Sulphates. Clearly then, there is some potential for nutrient recovery, particularly with regards to Phosphorous.

At present, inorganic fertilisers can be mass-produced industrially much more cheaply and efficiently when compared to production from urine treatment. Also, only a small fraction of the P input to fertilisers actually ends up in human urine, so this will never be a viable way to completely replace the rock phosphate input into fertilisers. Currently, the manufacture of fertiliser (struvite) from urine is generally viable in developing agricultural communities only because it is more economical to make on a small scale compared to importing industrially manufactured inorganic fertilisers (Ekama, 2011b). In time, nutrient recovery (N, P, Mg, K) from WWTP sludge liquors and separated urine may become a possibility as the quality and quantity of the available nutrient resources decline and the increasing costs of traditional extraction methods facilitate the need to search for alternatives. At present then, the drivers for adopting urine separation would be to save water, improve WWTP efficiency, capacity, and effluent quality, and to keep endocrine disruptors and environmental oestrogens out of the water cycle as much as possible.

2.9.2 Impacts on WWTPs

This section will focus on the specific impacts and expected results of urine separation on BNR WWTPs.

2.9.2.1 Sludge Age and Capacity

The requirement of WWTPs to nitrify Ammonia to Nitrate (as per effluent standards on Ammonia) means that sludge ages in BNR WWTPs have to be long enough to sustain the slow-growing autotrophic nitrifiers. To guarantee nitrification, the sludge age should be around 20 to 25 days (Mbaya, 2011). For a certain waste flow per day, a long sludge age requires a large volume of biological reactor, as explained by Ekama (2011a). If urine is separated from the rest of the wastewater at the source, the influent Ammonia levels could well be low enough to ensure that nitrification is not necessary to decrease the Ammonia effluent level to the regulatory limits. Therefore urine separation could remove the need for the WWTP size-defining nitrification bioprocess, and hence allow a large reduction in sludge age, down to about 8 to 10 days (Mbaya, 2011). Put boldly and succinctly, "The main bottlenecks in the biological processes of conventional wastewater treatment are related to the treatment of nitrogen and phosphorous" (Wilsenach, 2006). This reduction in sludge age will increase the influent hydraulic capacity of the WWTP by around 50%, or allow the volume of the biological reactors of new plants based on this principle to be around 2/3 of the volume that they would be with nitrification (i.e. without urine separation) (Mbaya, 2011). Agreement is found by Wilsenach (2006), where he concluded that if urine from an increasing part of the population were to be separated from the main wastewater stream, additional wastewater treatment plants or extensions to existing plants could be avoided.

Mbaya (2011), in an experimental setup, found much evidence to suggest that nitrification does not occur in a modified-UCT system of completely separated urine. This experiment aimed for 100% urine separation, although there was unexpected cross-contamination of urine with faeces in no-mix toilets. (Interestingly, there was in fact cross-contamination, which led to unexpected high TKN/COD ratios of the 'brown' (faecal) wastewater in this experiment.) This experiment was operated at a sludge age of 20 days, and showed significant nitrogen and phosphorous removal (although EBPR was unexpectedly not accomplished in this system due to a constant acclimatisation problem of the biomass to the new collected brown wastewater batches). The evidence of this experiment by Mbaya (2011) shows that the need to nitrify may be removed when dealing with urine-separated wastewater (and hence the WWTPs sludge age can be made shorter and reactor volumes can be made smaller), while still allowing adequate P and N removal (to meet effluent standards).

Some of the Nitrogen and Phosphorous present in the influent wastewater will always be taken up via the OHOs in their natural growth processes. In fact, Nitrogen and Phosphorous are important and necessary in the growth process and hence organic removal function of OHOs at Activated Sludge (AS) WWTPs. Generically, the molecular formula for a mole of these OHO organisms can be given as $C_5H_7O_2N_{0.8}P_{0.09}$ (Henze et al., 2008), which shows the N and P content of these OHOs (and hence their ability to capture the influent liquid nutrients in a solid form which can be collected and discharged). These AS systems would not function at their capacity, and the effluent organic content (given by the effluent COD value) would be high (as sludge growth would be retarded) if the influent TP/COD or the TKN/COD ratios were not high enough to sustain optimal (maximum) OHO growth. Accordingly, these systems would either be classified as Phosphorous-deficient or Nitrogen-deficient if these problems arose. Mbaya (2011) proposed that if the following TKN/COD ratios in Table 2.5 could be reached with urine separation, then N removal by ND would no longer be necessary. He also proposed the influent TP/COD ratios below which EBPR would no longer be necessary.

Table 2.5: Mbaya's (2011) estimates for influent TKN/COD and TP/COD ratios for complete removal of N and P without requiring ND and EBPR processes

Sludge Age (d)	Raw WW		Settled WW	
	TKN/COD	TP/COD	TKN/COD	TP/COD
5	0.031	0.0093	0.025	0.0076
8	0.028	0.0084	0.022	0.0066
10	0.026	0.0079	0.020	0.0061
20	0.023	0.0068	0.016	0.0049

2.9.2.2 Effluent Quality

With the lower influent loading of N and P into BNR WWTPs as a result of urine separation, the effluent levels of these nutrients can be expected to be lower as well (Wilsenach and van Loosdrecht, 2003). However, Wilsenach (2006) found in a model study that the level of Ammonia in the effluent remains roughly the same regardless of the influent concentration of Ammonia, due to the fact that the nitrifying biomass in the AS reactor decreases linearly as the influent Ammonia load decreases (with increasing levels of urine separation). In systems that already display full P removal, little change in the effluent P concentration is expected with increasing urine separation, as discovered by Wilsenach (2006) in a model study on urine separation. Wilsenach generally found a reduction in N in the effluent as urine was separated in great degrees, as shown Figure 16 on the following page.

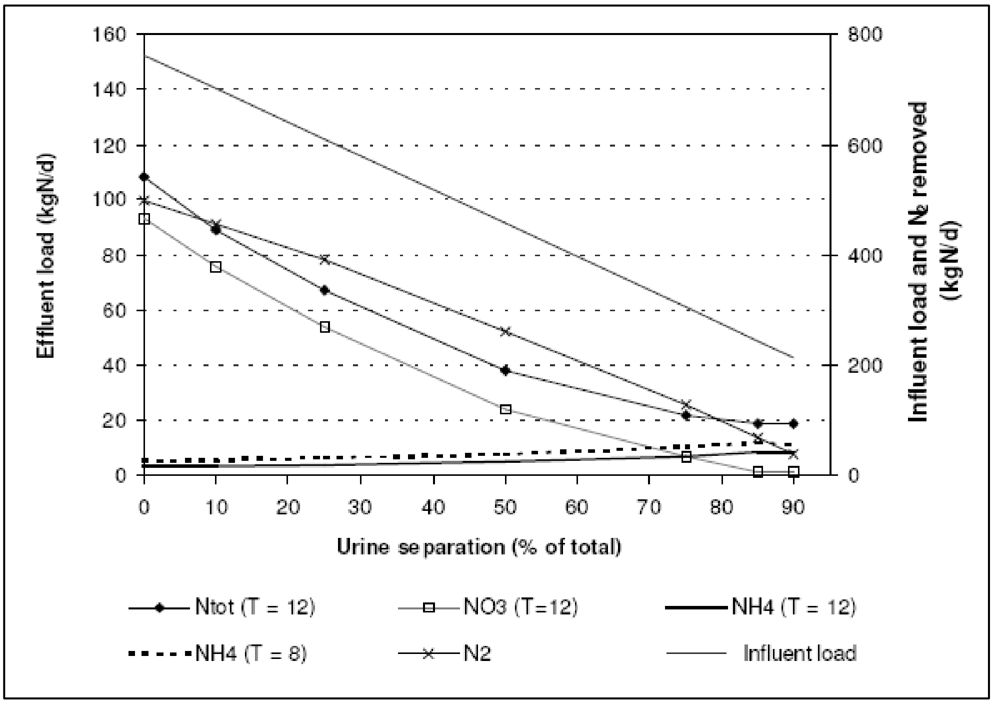

Figure 16: Expected effluent nitrogen concentrations with increasing urine separation as found by Wilsenach (2006)

The quality of the WWTP effluent in terms of micro-pollutants (EDs and EOs) would also increase. Endocrine disruptors and medical residues are largely excreted in human urine, and the dangers of EOs and EDs in WWTP effluent could largely be mitigated by source separation (and separate treatment and discharge) of urine.

2.9.2.3 Aeration Requirements

In terms of achieving goals such as reducing electrical operating costs (and hence also reducing CO_2 emissions from electricity produced from fossil-fuels), urine separation shows some good potential. Aeration costs (in terms of its electricity consumption) contribute a significant portion of the operational costs of a WWTP. To facilitate nitrification (in the aerobic zone of a biological reactor), part of the reactor must be aerated. This aeration uses electricity that, in South Africa particularly, predominantly comes from coal-fired power stations and is also susceptible to electricity price increases. While aeration of WWTPs will always be needed in the modified-UCT system to allow proper removal of organics in the biological reactor, the aeration requirements can be reduced if nitrification is not required and sludge age is reduced. This will reduce both the operating costs and carbon footprint of WWTPs and also increase the energy recovery via methane generation in anaerobic digestion (although realistically these benefits will have to be weighed up against the costs of implementing wide-spread urine separation, collection and treatment).

2.10 Practical Considerations for the Implementation of Urine Separation Technology

The technology exists to almost completely separate urine from the municipal wastewater stream. Urine separation toilets, as shown in Figure 17, can be implemented to separate both male and female urine from faeces without flushing. Simpler technology that is already in place and will require minor alterations (such as converting conventional male urinals to no-flush male urinals), could make the process of achieving some level of urine separation relatively simply. 100% urine separation would require retrofitting all household and other sewer systems. By implementing source separation of urine in new office blocks and other new buildings, and separately collecting urine from urinals at stadia, airports and shopping centres etc., this technology can begin to take root without needing major overhauls.

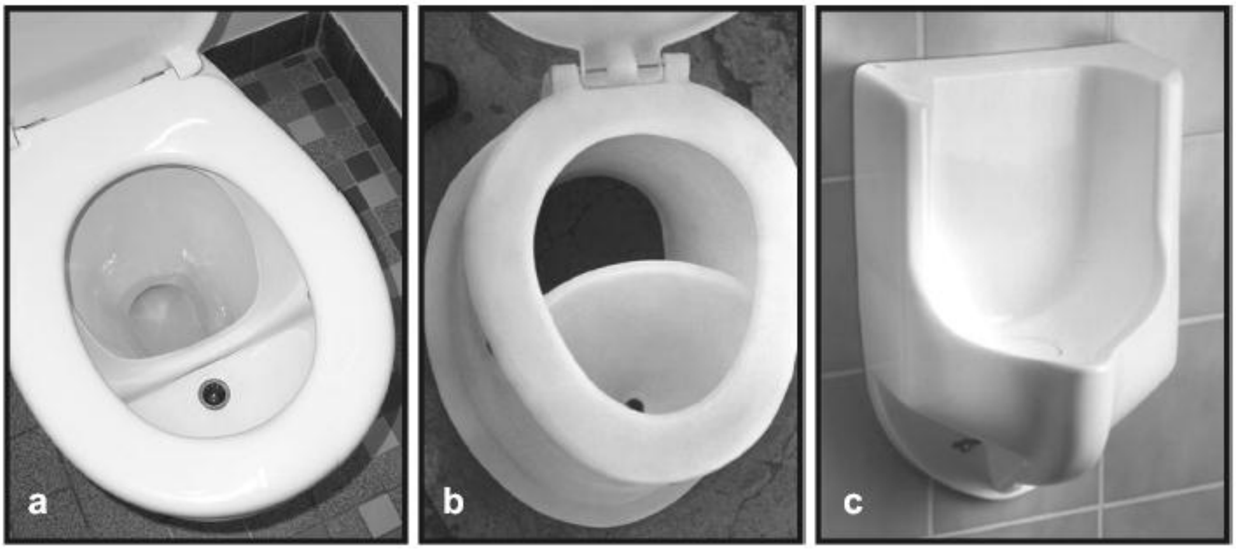

Figure 17: a) High-tech ceramic no-mix toilet, by Roëdiger, Germany b) Low-tech fibre resin dry no-mix toilet for improved pit latrine, by CSIR, South Africa and c) Waterless ceramic urinal, by the Waterless Company, USA [source: Wilsenach, 2006]

Urine can be collected separately via separate piping and passive collection (separate gravity sewers), or could be gathered in tanks at the source and actively collected via municipal trucks, in much the same way that solid waste is collected. However, urine is unstable and can quickly produce odours and precipitates, which have to be dealt with in urine collection systems. These practical details would need to be thought out at a planning level before this technology could be implemented.

Decentralised urine treatment plants would need to be established and the cost of building and operating these urine treatment facilities would need to be factored into the overall urine separation 'pros vs. cons' equation.

3. Chapter III – Simulation and Modelling

3.1 Specific Aims and Progression of this Experimentation

While the general aims of this thesis were outlined in Section 1.3 "Objectives of Thesis", there were some specific aims that had to be achieved in the experimentation (simulation) phase of this thesis investigation. These (somewhat intermediate) goals are given in the order that they were to be achieved, and show the outline and progression of the experimentation procedure:

- By deciding on the average daily wasteflow per person, the make-up of this wasteflow and the number of people to be serviced, develop influent data that reflects the properties of regular municipal wastewater.
- Using hand calculations and first-estimates, design a base-case WWTP based on the UCT system that services the (settled) influent wastewater with 0% urine separation.
- Check the validity of the UCTPHO software by reproducing this base WWTP in the software and comparing the hand-calculation base-case results with the base-case as modelled in UCTPHO software.
- By adjusting the percentage of urine separation in the serviced population, show the effect on the *influent* wastewater data with increasing urine separation.
- Preliminary Testing: Without changing the 'physical' setup (sludge age or reactor volumes) of the base WWTP, show the effect of the different levels of urine separation on the WWTP.
- Primary Testing I: By hand-calculations, develop a reference chart for optimising By changing and optimising the setup (sludge age and the anoxic and aerobic mass fractions) of the WWTP, show the performance of the optimised WWTP at incremental levels of urine separation.
- Primary Testing II: If a level of urine separation is found that facilitates sufficient N removal without requiring Nitrification and Denitrification, model a 2 reactor setup – with only aerobic and anaerobic reactors – and show the performance of this WWTP setup at and above the specified urine separation level.

The flow of the above steps is shown graphically in Figure 18 on the following page.

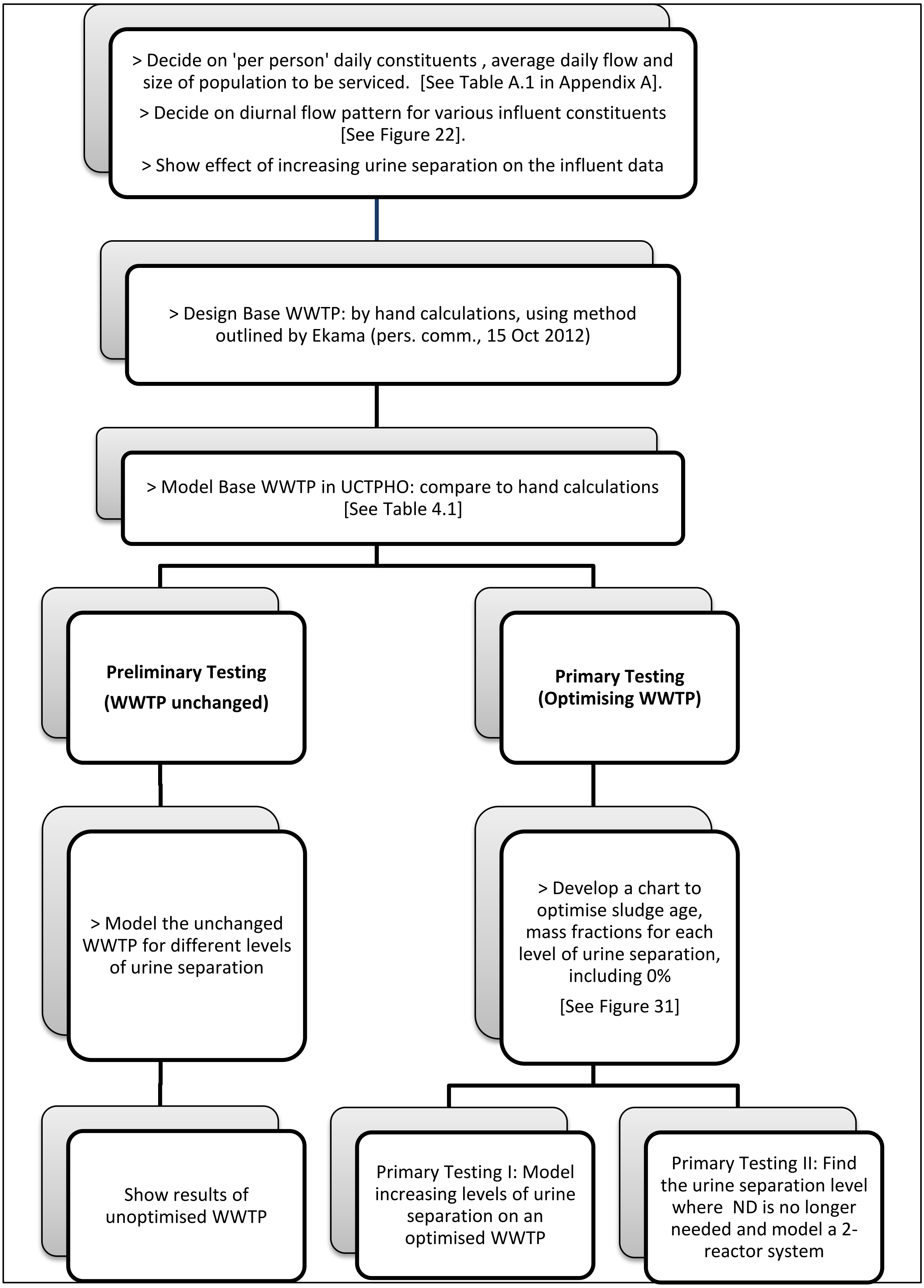

Figure 18: Schematic that graphically shows the order of steps followed to achieved the specific aims of this experimentation

3.2 Hypothesis

Steadily increasing the level of urine separation will reduce the effluent concentrations of TN and TP in direct relationship, while simultaneously allowing an increase in capacity (number of people serviced) and a lowering of the aeration requirements and complexity of the WWTP.

In the Preliminary Testing Phase it is expected to find the gain from urine separation weighing in on the side of improving effluent quality with limited addition in WWTP capacity. By contrast, in Primary Testing Phase I it is expected to find the same or somewhat lower effluent quality with the gains weighing in on the increased WWTP capacity side.

3.3 Developing the Experimental Setup

3.3.1 Average Influent WW Build-up and Characterisation

The diet of the community being serviced and the types of activities that the community engages in will greatly influence the properties of the wastewater being treated at the respective WWTP. High-protein diets (diets high in red meat) for example will result in high concentrations of Nitrogen in the influent wastewater, by resulting in relatively high concentrations of urea in the urine of the members of the community.

Table 3.1 below shows standard influent values for various strengths of raw municipal WW, which is mostly residential WW but includes some industrial WW. From the outset of this investigation, it was decided to aim for a raw influent WW strength that fell somewhere in the range between “medium” and “high” in Table 3.1 below. In Table 3.4 further on, the ‘chosen’ raw influent values are shown, and it is clear that these values fall within the desired range between “medium” and “high” strength raw WW.

Table 3.1: Standard influent concentration values for high, medium and low strength municipal WW (from Henze et al, 2008). All values are in mg(constituent)/ℓ.

Parameter	High	Medium	Low
COD total	1200	750	500
COD soluble	480	300	200
COD suspended	720	450	300
VFA	80	30	10
TKN	100	60	30
Ammonia	75	45	20
P total	25	15	6
Ortho-P	15	10	4
TSS	600	400	250
VSS	480	320	200

In an attempt to characterise the hypothetical WW in a realistic way, Table 3.2 and Table 3.3 below were consulted in addition to Table 3.1 above. The data in Table 3.2 shows historical influent data for Mariannridge WWTP in the eThekwini Municipality, South Africa. Table 3.3 shows the COD characterisation of average SA WW and compares this to data that was measured in a study at Mariannridge WWTP (Mhlanga, 2009). Mariannridge is a WWTP that treats 8Mℓ/d, with 70% of this being domestic and 30% industrial.

Table 3.2: Historical Data for Mariannridge WWTP in eThekwini Municipality (adapted from Mhlanga, 2009)

Component	Average	Units	No. of samples
COD total	774	mgO_2/ℓ	291
TKN	55	mgN/ℓ	15
Ammonia-N	25	mgN/ℓ	325
Nitrate	0.8	mgN/ℓ	15
P total	8	mgP/ℓ	113
Ortho-P	9	mgP/ℓ	15
TSetS	18	mgTSetS/ℓ	81
TSuspS	300	mgTSuspS/ℓ	62

Table 3.3 below shows the COD characterisation for average raw WW in SA, as well as measured COD characterisation at Mariannridge WWTP. A big difference, not shown in the table below, is that in Mariannridge WW, 14 % of the influent COD was present as OHOs in the raw influent WW. This is typically not considered in tradition WW characterisation (Henze et al., 2008).

Table 3.3: Showing the percentage make-up of "average" SA WW as well as measured WW

	SA: "Average" Wastewater	SA: Mariannridge Wastewater
COD_{TOTAL} (mgCOD/ℓ) (% of total)	- (100%)	774 (100%)*
COD_{RBSO} (VFA and FBSO) (% of total)	- (20%)	140.1 (18.1%)
COD_{USO} (mgCOD/ℓ) (% of total)	- (7%)	58 (7.5%)
COD_{BPO} (mgCOD/ℓ) (% of total)	- (60%)	342 (44.2%)
COD_{UPO} (mgCOD/ℓ) (% of total)	- (13%)	120.7 (15.6%)

(*Note that the rest of the total (not accounted for here) was made up by the COD of OHOs in the influent – which is considered to be zero in conventional WW characterisation.)

A hypothetical population had to be serviced and the size of this population had to be chosen. The population size was chosen as 136 500 people. CSIR (2000) suggests average daily flow values of between 70 ℓ/p.d (low income) to 125 ℓ/p.d (middle income) to 250 ℓ/p.d when designing sewers, and it was assumed that this could be used when designing for WWTPs as well. It was decided that the serviced population would fall between the "low" and "medium" income categories as per CSIR (2000). What is not taken into account in these values by CSIR (2000) are the infiltration of stormwater into the sewers as well as the equivalent contribution of industrial WW for each member of the community. Hence small adjustments of these values were made to account for these additions.

Although a settled WWTP was being designed for, it was apparent that the raw influent data was important because this represented the actual contributions from the community members at the source. The average daily contributions per person can be seen in Table A.1 in Appendix A, where the source of the WW and the characterisation of each type of WW (yellow, brown, grey, infiltration and industrial) is shown. It was important to perform an iterative process whereby the daily mass contributions per person, daily flow per person and number of people combine to produce influent with desirable (somewhat standard) properties. It was for this reason that some of the 'per person daily contribution' values were inferred from the total influent concentrations that were sought to be achieved. Basically, the following conditions were all sought to be met:

- Each person contributes around 100 ℓ/p.d of WW, of which yellowwater is 35l/p.d and is around 3 times more than brownwater. Also, total flush-water is 20-30% of the total wastewater generated per person per day. Grey water should be more than the total flush-water, and should be the major contributor to the daily average flow per person.
- The average COD contribution of each person is around 100 mgCOD/p.d
- The average TKN contribution of each person is around 8 mgTKN/p.d
- The average TP contribution of each person is around 2 mgTP/p.d
- The TKN in urine (almost all of which FSA) should make up 80% of the TKN in the influent wastewater.
- The TP in urine (all of which is OP) should make up 50% of the TP in the influent wastewater.

As can be seen from the above set of parameters, effectively a combination of simultaneous conditions had to be met by the influent data selected. With much trial and error and a few simultaneous equations, Table A.1 in Appendix A and Table 3.4 below were produced and used throughout the simulations as the base influent WW. Table A.1 in Appendix A is important as it shows the 'per person daily contributions' of the hypothetical population.

Table 3.4: Influent data 'chosen' for base WWTP with no urine separation

Component	Raw	Settled	Units
COD total	910	605.4	mgO_2/ℓ
TKN	75.4	63.9	mgN/ℓ
Ammonia-N	51.9	51.9	mgN/ℓ
P total	16.1	13.6	mgP/ℓ
Ortho-P	11.5	11.5	mgP/ℓ
ISuspS	79.0	27.1	mg/ℓ
TSuspS	508.4	252.7	mg/ℓ

By using the average daily contribution of roughly 110 ℓ/p.d (as shown in Table A.1 in Appendix A) and a catchment population of 136 500 people, an average daily (raw) WWTP influent flow of 15Mℓ/d was established.

3.3.2 Daily (Diurnal) Influent WW Fluctuations

In the absence of equalisation (balancing) tanks at the head of the WWTP facility, the fluctuations in flow over the course of a day vary from about twice or more down to about half or less the average dry weather flow (ADWF) (Ekama, 2012). The magnitude of the variation depends on the size of the community served, the layout of the sewerage system and the amount of infiltration. Gravity fed systems result in more gradual variations than pumped systems (Ekama, 2012).

As per CSIR (2000), when designing sewers for a large catchment population, one can expect a high attenuation ('levelling out') of peak flows, as per Figure 19 below. The same wisdom could be applied to the expected peaks in the sewers approaching the WWTP, and hence for the flow entering the WWTP itself. According to this logic and using Figure 19 below, the peak flow factor for a population of 136 500 people would be 1.8.

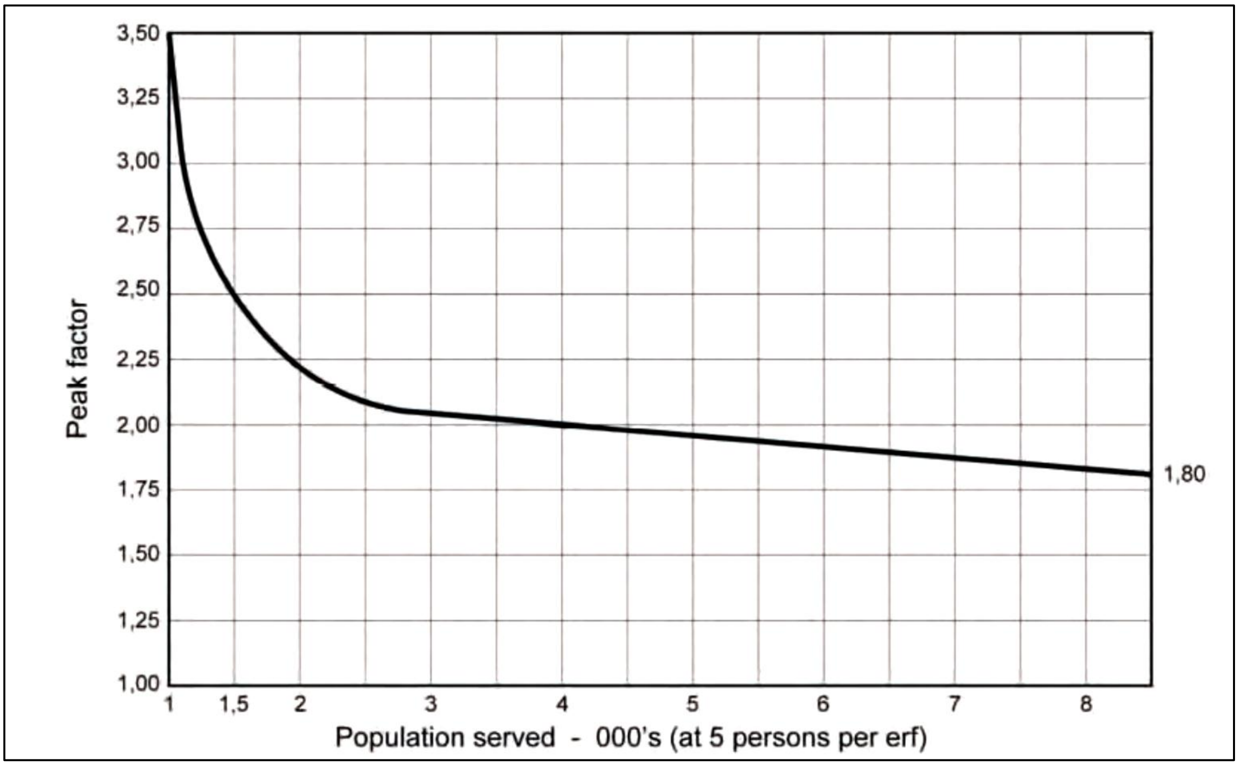

Figure 19: Graph showing how the peak design factor in sewage design is affected by the size of the population served [Source: CSIR, 2000]

Once the peak flow factor was decided, establishing an actual pattern for diurnal fluctuations in flow, COD, TKN, TP and other influent WW characteristics was an important next step. Various sources are available which cover the diurnal WW flow pattern, although sources showing how the rest of the influent characteristics vary diurnally are scarcer. Figure 20 on the next page shows three completely different sources showing basically the same trend with regard to the diurnal WW influent flow patterns: the main peak occurs around 10am to 12pm, while a second, smaller peak occurs around 8pm to 9pm. There is also a large lull in flow between 3am and 6am.

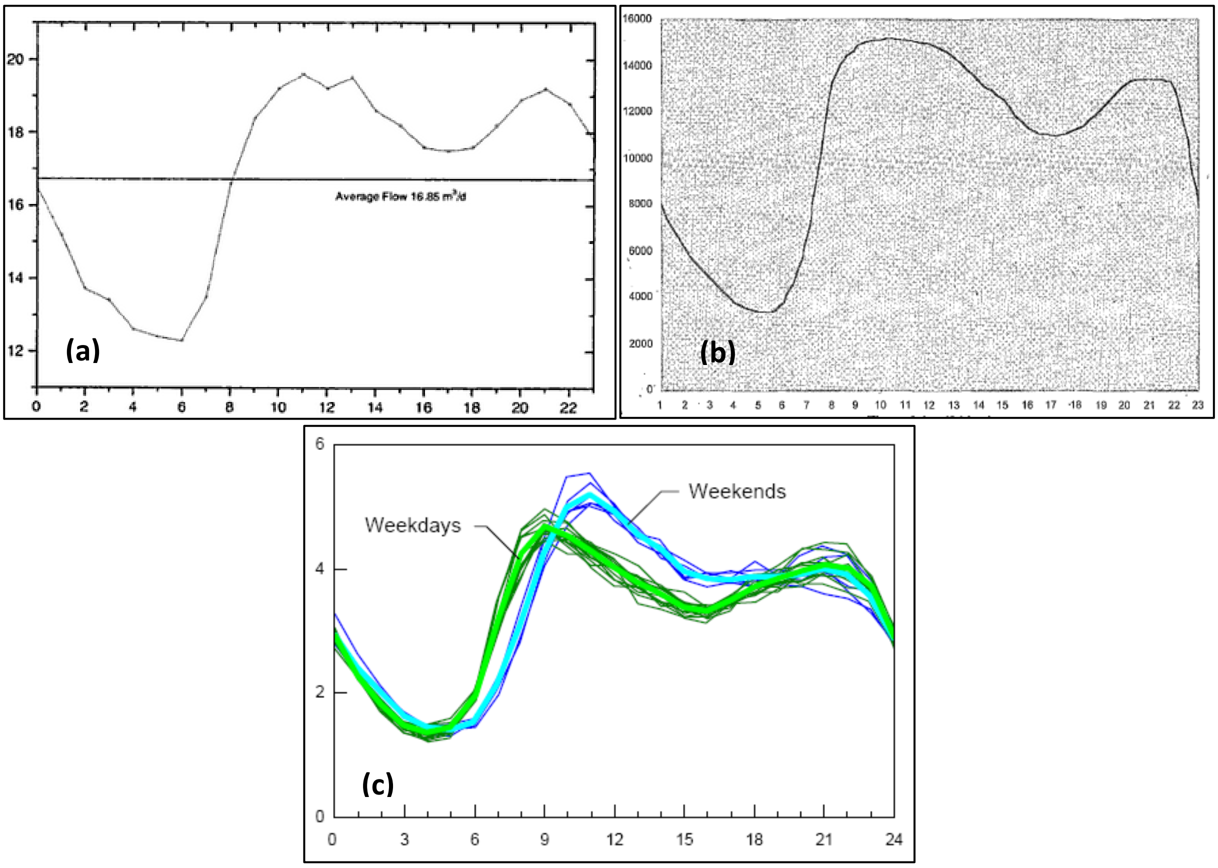

Figure 20: Various sources showing similar typical diurnal flow patterns (All show some form of Flow vs. Time of day) [Sources: (a) Karia et al. (2006), (b) EPA (1999) & (c) Enfinger (2006)]

In attempting to quantify how the rest of the influent WW characteristics varied diurnally, a study by Langergraber et al. (2007) proved useful. The influent data that Langergraber modelled is shown on the next page in Figure 21, and seems to suggest that TKN peaks earlier in the day than COD, which peaks at around the same time as TP. This would make sense, as FSA (which makes up the majority of influent WW TKN) is soluble and would travel almost simultaneously with the water flow (also, FSA is mostly contained in urine and it could be argued that urinating is one of the first WW-generating activities in the course of a day).

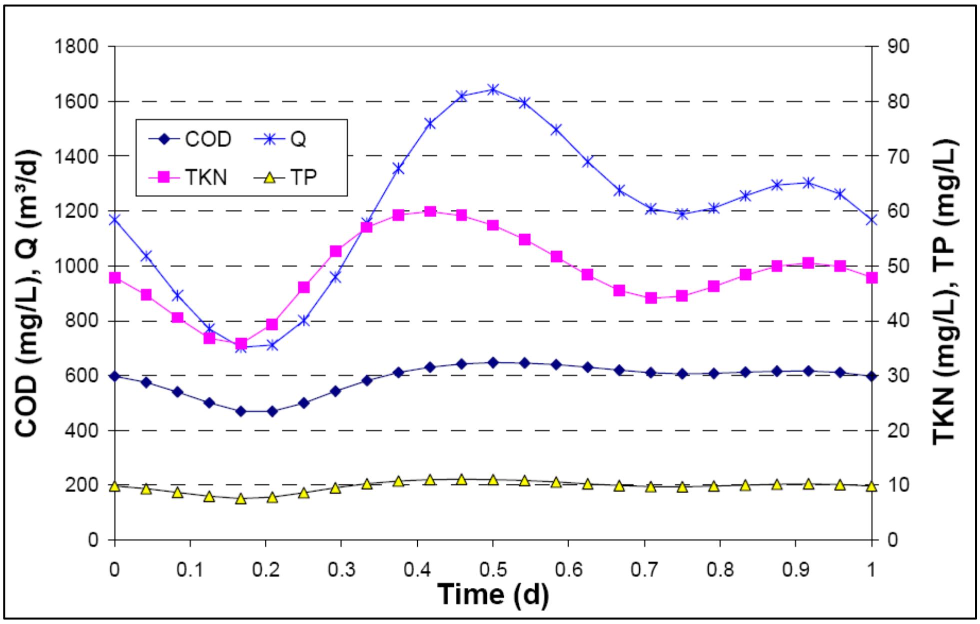

Figure 21: Modelled Flow, COD, TKN and TP diurnal fluctuations [Langergraber et al., 2007]

As shown in Figure 21 above, the COD and TP seem to peak later than FSA, and it is this thinking that was used to produce Figure 22 below, which shows the diurnal patterns that were constructed by this author and used throughout the dynamic simulations. An example of the diurnal inputs used in UCTPHO is illustrated in Figure 41 for 0% urine separation (in Appendix A).

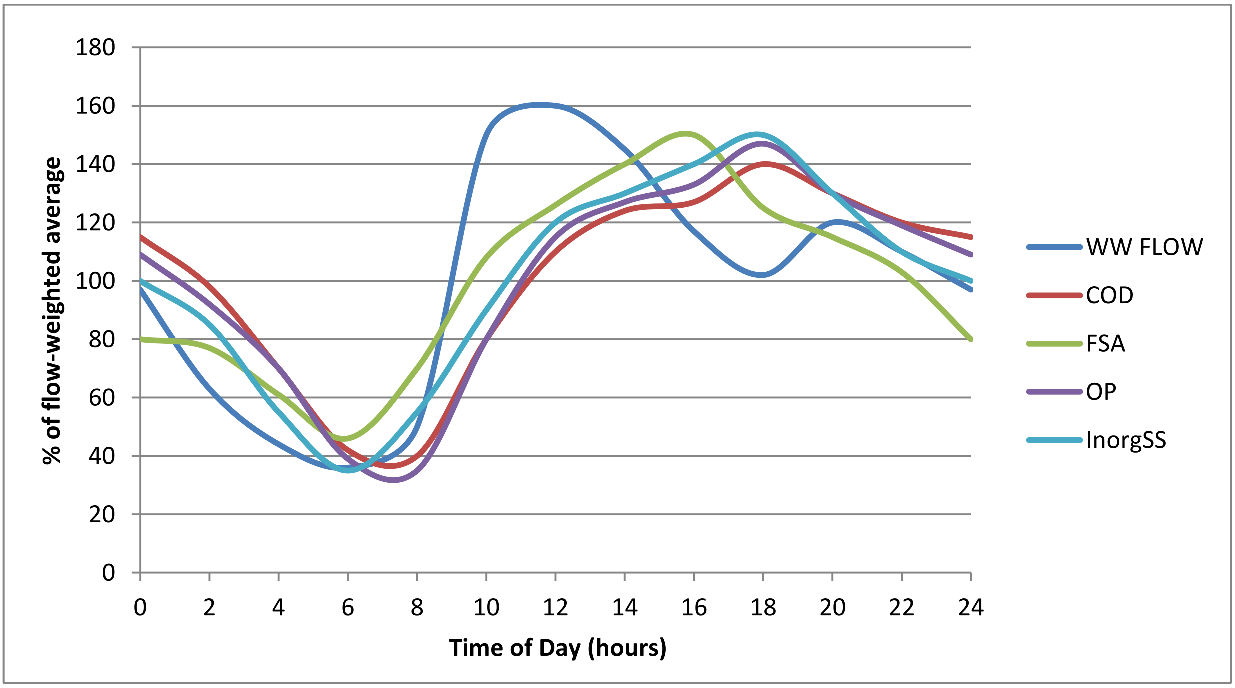

Figure 22: The diurnal pattern of different influent WW properties that was chosen for the all diurnal simulations in UCTPHO software

The diurnal fluctuations of the various nutrients were expected to have significant impacts on the peak system performance and the peak effluent concentrations. It is well known both experimentally and theoretically that under cyclic flow and load conditions the nitrification efficiency of the AS system is decreased compared with that under steady-state conditions (Henze et al., 2008). During the high load period, even though the nitrifiers are operating at their maximum rate, it is not possible to oxidise all the Ammonia available and an increased Ammonia concentration can be expected in the effluent (Henze et al., 2008). It is for this reason that a safety factor in the reactor calculations has been implemented. Still, these kinds of diurnal fluctuations in performance could still manifest in the diurnal simulations, and were expected to provide interesting results.

3.3.3 Effect of Urine Separation on Influent Data

Because yellowwater contains relatively little COD compared with faeces, what was not initially intuitive is why the influent COD concentration was increasing with increasing urine separation. It was then obviously understood that an increase in urine separation has a concentrating effect on the rest of the WW, as the majority of yellowwater is in fact simply flush-water (and hence also represents the freshwater saving potential). Removing this water from the rest of the WW stream allows an increasing concentration of some of the other influent characteristics, one of them being COD. The effect of urine separation on the settled influent COD is shown below in Figure 23 below. The same relationship would be found for raw WW, but with higher influent COD concentrations.

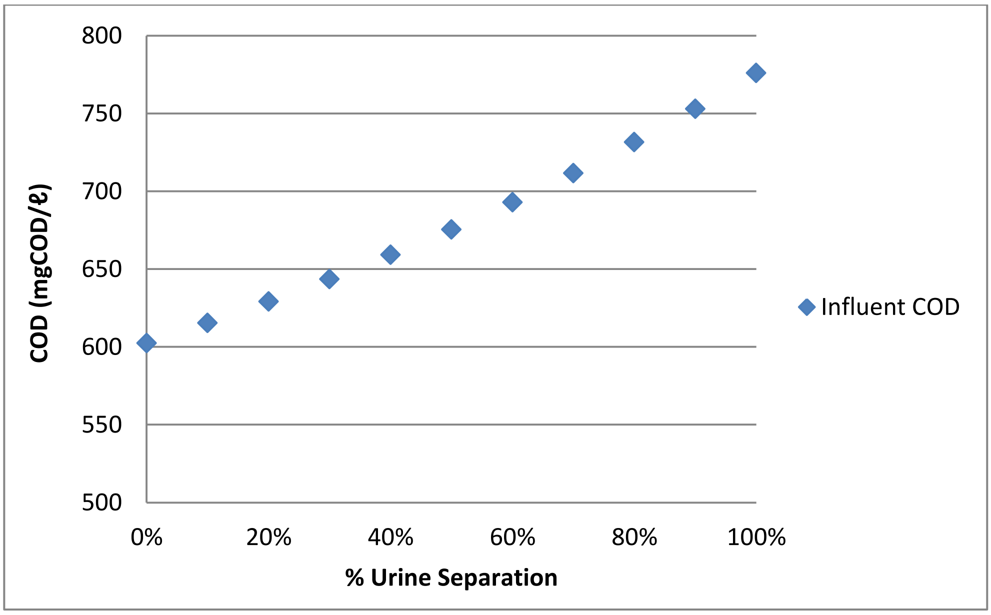

Figure 23: Graph showing the increasing (settled) influent COD concentration as urine is increasingly separated

It is also clear that as urine (containing around 80% of the total N and 50% of the total P) is separated in greater degrees, the influent TKN and TP concentrations should decrease. Hence, the influent TKN/COD and TP/COD ratios should also decrease. What is interesting to note is that this effect is two-fold, in that while the influent TKN and TP concentrations decrease with urine separation, the influent COD concentration also increases with increasing urine separation. This compound effect of urine separation on both the TKN/COD and TP/COD ratios is shown below Figure 24, with the TKN/COD axis on the left and the TP/COD axis on the right.

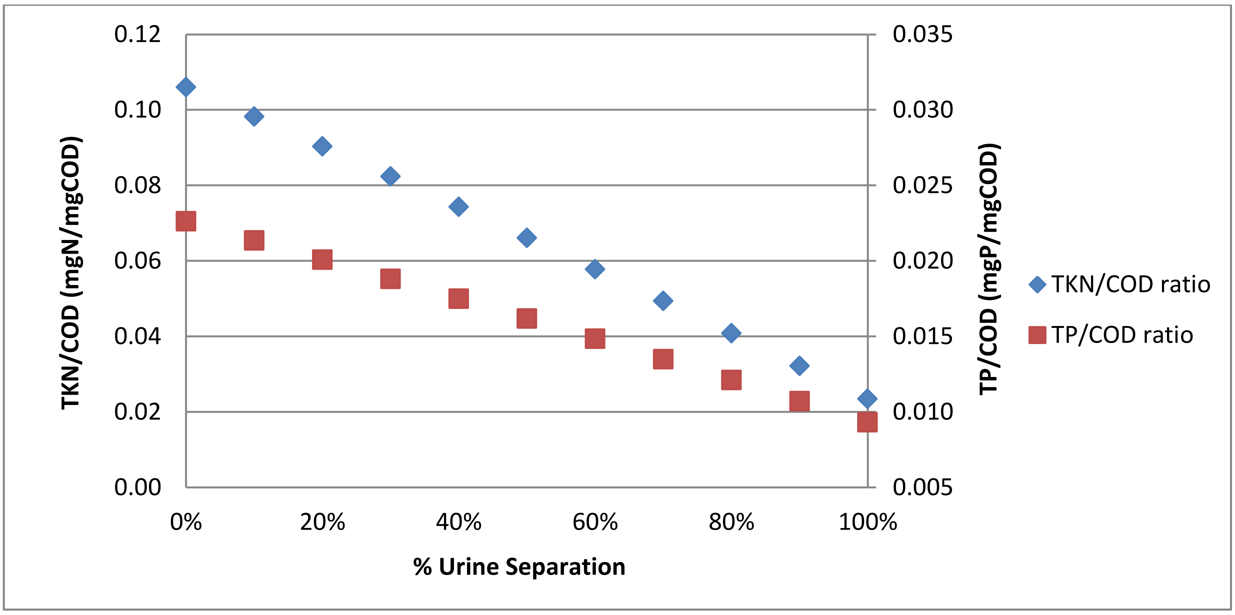

Figure 24: Effect of urine separation on the (settled) influent TKN/COD and TP/COD ratios (note the two different axes)

3.3.4 WWTP Model: Setup, Explanation and Assumptions made

3.3.4.1 General Assumptions

A list of the constants used in calculations can be found in Appendix B – "List of Constants".

- A UCT WWTP system was to be modelled (with an anaerobic, anoxic and aerobic reactor).
- No NO_3 or DO would be considered present in the influent flow. Also, no organisms such as OHOs, ANOs or PAOs would be considered present in the influent WW either.
- It was safe to assume that all the Nitrogen in urine would reach the WWTP as Ammonia (FSA) through the hydrolysis process.
- An operating temperature of 14°C was chosen, and this generally represents a low (conservative) average for winters in SA – representing a relative 'worst-case' scenario performance.
- The nitrification effect of ANOs are included in the hand-calculations, but in these hand calculations the mass (and growth) of ANOs is not included in the sludge mass, because they only make up a small percentage (<3% by mass) of the total sludge mass. In the simulation software however, the mass and growth of the ANOs is included in the VSS of the biological reactors.
- In UCTPHO, the Alkalinity of the influent WW was set at 6mg/ℓ as $CaCO_3$.

- Balancing tanks at the head of the WWTP were not considered, so as to show the effects of dynamic/diurnal fluctuations on WWTP performance. There is merit in considering a balancing tank to dampen out these fluctuations in the flow and pollution load of the wastewater flow, but generally it is uneconomical to achieve complete balancing, although the larger the plant the more attractive and economically viable a balancing tank becomes (Ekama, 2012).
- As urine is separated, the diurnal flow pattern was considered to stay the same i.e. The general daily 'spread' of wastewater flow, COD, FSA, TKN, TSS etc. will stay the same. This assumption is fairly appropriate, as people would generally urinate regularly throughout the day, as opposed to say greywater which would most likely be concentrated during the morning and evening peaks.
- In UCTPHO, a switching function for OHO and PAO growth on Ammonia (called the "K_s NH_3" switching function) was changed from 0.01 instead of 0.1 in attempt to avoid numerical instability in UCTPHO at high urine separation (but this was only partially successful and will be discussed in Section 4.1.2 "Difficulties with UCTPHO").
- The maximum specific growth rate of nitrifiers (μ_{am20}) was set at 0.6/d (which is within the range of 'common values' of 0.3 - 0.75/d). This value needs to be set in the WWTP modelling, and is an important choice that will have a significant effect on the magnitude of the minimum sludge age for nitrification as explained by Henze et al. (2008). A relatively high value was selected so that nitrification would not fail by 'wash-out' (where the sludge age is too short to support nitrifier growth) but rather by too little Ammonia in the influent. Low to zero Nitrate in the effluent should therefore mean the point of not requiring ND for N removal in the WWTP has been reached. It is acknowledged that had a lower μ_{am20} value (say 0.5/d) been selected, the gain in WWTP capacity would have been lower than with 0.6/d because the system sludge ages would have been longer for each level of urine separation.
- A Safety Factor of 1.25 was used to account for nitrifier growth during diurnal fluctuations.
- The Dissolved Oxygen (DO) level in the aerobic reactor was set at 2mgO/ℓ in the UCTPHO software and in the hand calculations. This value affects the ANOs significantly. Generally, the higher the DO value, the better the OHO and ANO performance. However, it becomes exponentially more difficult to force more oxygen into solution at high DO values, which consequently becomes expensive. Generally, Nitrifiers require a minimum concentration of 1-2mgO/ℓ (Henze et al., 2008), so setting the DO value at 2mgO/ℓ ensures that it falls within the acceptable range where optimal organism behaviour is facilitated, but at a reasonable economic cost as well.
- For economic reasons, as based on Figure 25 on the next page (from Henze et al., 2008), the TSS concentration in the last reactor before the SST (generally the aerobic

reactor in a UCT WWTP system setup) should be chosen between around 3.5kg/ℓ and 5.5kg/ℓ. Generally 5kg/ℓ was chosen.

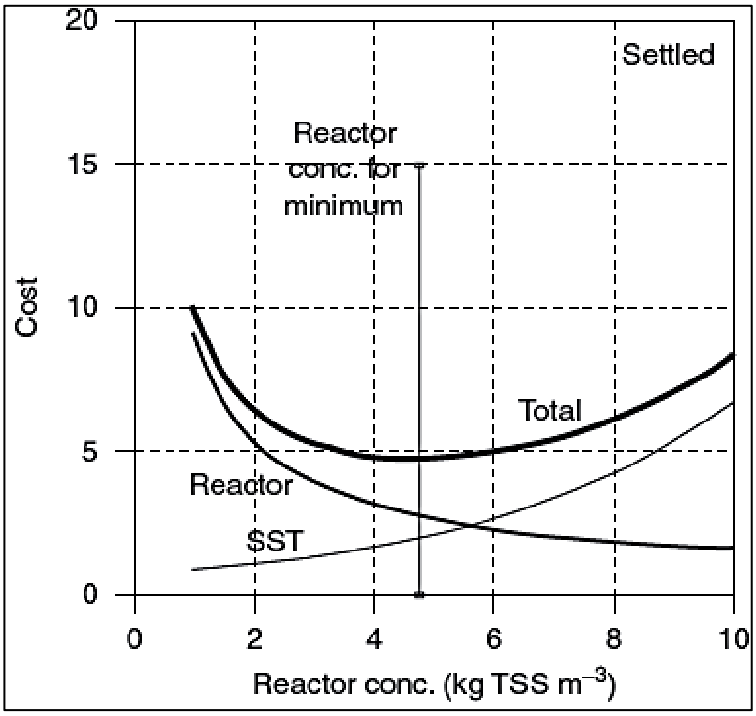

Figure 25: Showing how the choice of reactor TSS concentration affects the total WWTP cost [Source: Henze et al., 2008]

3.3.4.2 Notes on Preliminary Testing

The point of this testing was to see the impact of an unchanged WWTP under increasing degrees of urine separation. This situation could represent a real-world WWTP that is not managed efficiently and would not get optimised with changing influent WW characteristics, or a poorly funded WWTP where such optimisation changes are not financially possible.

The mass fractions and volumes of reactors were set throughout this preliminary testing phase. The anaerobic mass fraction was set as 0.12 of the total, and the anaerobic reactor was set as 2.24Mℓ. The anoxic mass fraction was set as 0.38, and the anoxic reactor was set as 3.54Mℓ. The aerobic mass fraction was 0.5 and the aerobic reactor was set as 4.66Mℓ.

Throughout this testing phase, only the influent WW data and the ‘a recycle’ (between the aerobic and anoxic reactors) were changed. The sludge age was chosen as 15 days and operated at this sludge age throughout this testing phase. The reactor mass fractions and volumes were unaltered throughout. The ‘a recycle’ was calculated to optimise the recycle of Nitrate to the anoxic reactor, and hence maximise N removal via denitrification. There is a limit to this recycle that can practically be achieved (a ratio of 6:1 in terms of ‘a recycle’ to influent flow), and this maximum practical recycle ratio was most often used as the optimum ‘a recycle’ ratio for Nitrogen removal.

Throughout this testing phase, for the different levels of urine separation, the serviced population was increased to maintain the raw organic load (flux of COD per day) at the same value as the base-case organic load. The organic load (COD Flux = Influent COD x Influent Flowrate) was kept the same regardless of the urine separation level in order to see what the gain in WWTP capacity is when the WWTP was not changed in any way (except for maybe the 'a recycle'). So new hypothetical people were added (with the same degree of urine separation technology as those already in catchment population) to the WWTP catchment population in such a way so as to keep the raw organic load constant, which also kept the aerobic reactor TSS concentration at roughly the target of 5 kg/ℓ.

Although the modelled WWTP was a plant with primary settlement (i.e. treating settled wastewater), it was not possible to keep the settled WW organic flux exactly the same at every level of urine separation. Instead, the raw organic load (COD raw flux) was kept constant. This was due to the fact that the simulation modelled the effect of the primary settler and removed the settleable solids from the influent WW before biological treatment in the modelled WWTP. This is worth noting, but was not deemed significant, as the maximum difference in the settled organic load (flux of settled influent COD = settled influent flowrate x settled influent COD) between 0 and 100% urine separation was only 2%.

3.3.4.3 Notes on Primary Testing I

During this testing phase the aim was to apply urine-separated wastewater to the WWTP but to optimise the operation at each urine separation level. Here the mass fractions of the anoxic and aerobic reactors were changed (practically this is possible – where different portions of the reactors can be turned into aerated or unaerated zones) to optimise N removal. The anaerobic mass fraction was maintained as 0.1 throughout this testing phase.

As the influent TKN/COD ratio decreased with increasing urine separation, it was found that the sludge age of the system could be lowered while still providing sufficient ND capabilities. This will be explained in detail in Section 4.3 "Primary Testing".

The plant was optimised for Nitrogen removal, not P removal. The reasons for this are that it is extremely difficult to simultaneously optimise for both N and P removal, and that P removal is more dependent on the influent readily biodegradable organics and less on the setup of the WWTP (when compared to N removal processes).

Depending on the urine separation level and the subsequent TKN/COD ratio of the settled influent WW, the WWTP was optimised for sludge age and reactor mass fractions, which will be explained in Section 4.3. The unaerated mass fraction was set at the maximum unaerated mass fraction, which was calculated to ensure nitrification in the aerated (aerobic) reactor. Equation 5 below was used to calculate this maximum unaerated mass fraction.

$$f_{xmax} = 1 - \frac{S_f\left(b_{AT} + \frac{1}{R_s}\right)}{\mu_{AmT}}$$ **Equation 5**

It is suggested by Henze et al. (2008) that the maximum unaerated mass fraction as calculated in Equation 5 should be capped at 0.6 (i.e. 60% of the total mass fraction). The anaerobic mass fraction was set at 0.1 and the anoxic mass fraction was therefore calculated as the difference between the maximum unaerated mass fraction and the anaerobic mass fraction.

The approach followed by Wilsenach (2006) to determine the capacity increase of a WWTP with increasing levels of urine separation was based on the reduced hydraulic load due to increasing levels of urine separation. Wilsenach (2006) therefore increased the catchment population to restore the original hydraulic load on the WWTP. However, this approach was questioned by Ekama (pers. comm., 15 Oct 2012) and instead an approach was suggested where the capacity increase is based on keeping the TSS concentration of the aerobic reactor the same at all levels of urine separation. This TSS concentration is what is discharged to the SST and effectively determines the capacity of a WWTP with existing SSTs (G. Ekama, pers. comm., 15 Oct 2012). The expected capacity increase would come from the decrease in the TSS concentration as the optimal sludge age was dropped with increasing urine separation. The total reactor volume and the volume of the anaerobic reactor, as well as the anaerobic mass fraction (0.1) were kept constant throughout the simulations. The anoxic TSS concentration was set at 4kgTSS/ℓ throughout and the calculated maximum anoxic mass fraction (maximum unaerated minus anaerobic mass fraction) was used to determine the volume of the anoxic reactor required to keep this reactor’s TSS concentration at 4kgTSS/ℓ. Thus the aerobic reactor volume was calculated as the difference between the (constant) total volume and the unaerated reactor volumes (anoxic plus anaerobic), and the aerobic mass fraction was the difference between 1 and the max unaerated mass fraction. This left the TSS concentration of the aerobic reactor as the only (capacity-related) variable, which would inevitably decrease if the catchment population was left unchanged with increasing levels of urine separation (and hence lower optimum sludge ages for increasing levels of urine separation). For each level of urine separation, with its own optimised sludge age and maximum unaerated mass fraction, the population was increased to ‘restore’ the TSS concentration back to the TSS concentration of the optimised base-case WWTP of 5kgTSS/ℓ In this way, the capacity increase was expressed and reported in Section 4.3.1 “Capacity Change”. Therefore as the organic load (COD flux) decreased due to increasing urine separation (the flow decreases while COD concentration increases slightly), people were added (with the same level of urine separation) to keep the TSS concentration at 5kgTSS/ℓ.

3.3.4.4 Notes on Primary Testing Phase II

Once the point was found where nitrification and denitrification were no longer necessary for optimum nitrogen removal, a 2-reactor system (only an anaerobic and aerobic reactor) was used to simulate the simplified WWTP setup where just PAOs and OHOs perform the COD, N and P biological removal functions.

4. Chapter IV - Results and Discussion

4.1 Discussion of performance of UCTPHO simulation software

4.1.1 Comparison between hand-calculations and UCTPHO simulations for base-case WWTP with no urine separation

The *influent* WW characteristics were slightly different between the SS hand calculations and the UCTPHO calculations. This difference arose from a slight variation in the way that the flow-weighted averages of different influent characteristics (COD_i, TKN_i and TP_i for example) were calculated from the daily (diurnal) flow patterns in the UCTPHO program, compared to how these were calculated using Simpson's Rule in the hand-calculations. However, these differences were less than 0.5%, so were not worth giving any more thought to.

Using the exact inputs from the hand-calculations in UCTPHO and modelling what was thought to be the exact same systems produced effluent qualities that varied between the hand calculations and the UCTPHO simulation results. The effluent quality of the hand calculations and UCTPHO steady state were all fairly close except for the P_S in the effluent. It was found that NO_3 was being recycled to the anaerobic reactor via the b recycle, and this was not predicted by the hand-calculations.

To elaborate on this discrepancy, when using the exact inputs from the hand-calculations there appeared to be denitrification occurring in the anaerobic reactor in the UCTPHO steady state simulation (for the base-case of 0% urine separation). The presence of Nitrate in the anaerobic reactor greatly impedes the EBPR function of the PAOs in this reactor as explained by Henze et al. (2008), so it was imperative to find the root of this problem and fix it. After consultation with Ekama (pers. comm., 15 Oct 2012), it was postulated that perhaps the K_{2T} rate in the UCTPHO software slowed down more rapidly than the value used in the hand-calculations and hence perhaps the hand-calculations over-estimated the denitrification potential of the anoxic reactor. Assuming that the UCTPHO software was more accurate than the hand-calculations, it was suggested by Ekama (pers. comm., 15 Oct 2012) that in order to overcome this problem, the 'a recycle' (between the anoxic and anaerobic reactors) be lowered from the a-opt value (calculated by hand) to ensure that the anoxic reactor was not overloaded with Nitrate (which is the root of the problem of Nitrate entering the anaerobic reactor and inhibiting P removal by PAOs). It was decided that because the anaerobic reactor was under-capacity (at steady state), it could be made to perform a minor denitrification function, if the NO_3 in the b recycle was be kept below a value of $1mgNO_3$-N/ℓ (to prevent interference with EBPR processes) (G. Ekama, pers. comm., 15 Oct 2012).

There were, however, some important implications of lowering this 'a recycle'. It was found that a lower 'a recycle' would reduce the Nitrate load on the anoxic reactor and hence reduce the Nitrate load on the anaerobic reactor (if the denitrification capacity of the anoxic reactor is not high enough to deal with the Nitrate load from the aerobic reactor) leading to better P removal via PAOs, and hence better (lower) effluent P concentrations. However, a lower ‘a recycle’ would also mean that the Nitrate was not recycled from the aerobic reactor to the anoxic reactor (where denitrification would take place – releasing the nitrogen in the form of N_2 gas), meaning that more Nitrate would escape with the effluent, ultimately raising the NO_3 and TN concentrations in the effluent. Effectively then, when deciding to change the ‘a recycle’ ratio, one is trading off between the TP and the TN in the effluent.

Despite the best efforts of the author, the effluent quality of the base case WWTP developed by hand calculations could not be matched by the steady state effluent quality of the UCTPHO simulations, even with the reduction of the ‘a recycle’ ratio. The UCTPHO steady state effluent values were significantly higher, especially with respect to Total N and Total P, as shown in Table 4.1 below. Table 4.1 below shows the comparison between the hand calculations and the UCTPHO steady state simulations.

After due consideration and in consultation with Ekama (pers. comm., 15 Oct 2012), it was decided that lowering the ‘a recycle’ (below the optimum ‘a recycle’ calculated) was the only possible method of altering the WWTP setup (without changing the sludge age). For the base-case 0% urine separation situation, the ‘a recycle’ was set at 1½:1 in UCTPHO instead of the calculated value of 2.71:1. It was concluded that the best-possible ‘a recycle’ ratio was chosen, and a fair trade-off was reached between lowering the P effluent concentration and increasing the NO_3 effluent concentration, and the results of this change and the subsequent comparison is shown in Table 4.1 below. Some of the effluent qualities (NO_3 in particular) fall outside the range of acceptable effluent standards in South Africa.

Table 4.1: Showing the comparison between the UCTPHO steady-state and Hand Calculation NDEBPR UCT models for various characteristics of the base-case influent data.

Parameters	**UCTPHO steady-state with adjusted a recycle (set at 1½:1)**	**Hand Calculations (a recycle set at a-opt of 2.71:1)**	**% Difference**
Total OHO concentration (mgCOD of OHOs/ℓ)	1360.3	1277.4	6%
Total Endogenous Residue concentration (mgCOD of ER/ℓ)	1016.8	978.1	4%
Total VSS concentration (mgVSS/ℓ)	2974.3	2866.3	4%
OUR_t $mgO_2/(ℓ.h)$	59.6	61.7	-3%
$N_{a,effluent}$ (mgFSA-N/ℓ)	1	1.02	-2%
$NO_{3,effluent}$ ($mgNO_3$-N/ℓ)))	11.5	9.95	13%
$P_{s,\ effluent}$ (mgP/l)	0.6	0	-
$S_{us,\ effluent}$ (mgCOD/l)	61.3	54.6	11%

While the resulting conditions of this base WWTP under 0% urine separation load (especially in the UCTPHO simulations) conditions do present some difficulty with regards to its justification, it could be argued that having a base model where the effluent qualities exceed the effluent regulations could actually be beneficial to this research. There are many WWTPs in South Africa that do not conform to the national effluent quality guidelines, as shown in the Green Drop Report and highlighted in Section 2.7 “Status of WWTPs in South Africa”, and many more are not operating to their maximum potential capacities due to sub-optimal setup or operation. While setting up a base WWTP model that does not meet the effluent guidelines was certainly not desirable at the beginning of this research, it has become apparent that perhaps this may in fact give great insight after all. The pertinent question then becomes, “What affect can urine separation have on WWTPs that currently do not meet effluent quality guidelines, as many in South Africa do not?” It could even be argued that setting up a base WWTP in UCTPHO software which does not meet the current effluent requirements is perhaps more appropriate for the South African context. However, it must be reiterated that this is simply an optimistic way of looking at an unforeseen and unplanned situation that arose.

4.1.2 Difficulties with UCTPHO

Modelling the desired WWTP at different levels of urine separation on WWTP computer software was not as easy as first anticipated. UCTPHO software is not updated for Windows 7 and as such, old software or an emulator had to be used in order to run this software. A DOS emulator (called “DOSBOX”) was used to run this programme in Windows 7. While enabling the programme to run, this emulator software slowed the programme down significantly and lead to diurnal simulations taking upwards of 2 hours each to run. This, combined with the fact that the software would often encounter “runtime errors” or “freeze” and exit when an error occurred, put a significant limitation on the quality and number of (diurnal) simulations that could be run.

The programme showed a tendency to get “stuck” at high urine separation levels. For this reason the K_s NH_3 ‘switching function’ was changed from 0.01 to 0.1 under suggestion from Ekama (pers. comm., 15 Oct 2012). This was partially successful, and enabled previously-unrunnable simulations of up to 90% urine separation. This programme was extremely sensitive to the various inputs, and had a habit of exiting unexpectedly. As explained, this happened especially at high levels of urine separation when the sludge age was lowered. It is predicted that perhaps the optimum sludge ages at high urine separations chosen were too low, and combined with low growth-rate constants or ‘switching function’ values, caused the inconsistencies that were experienced.

This resulted in tests often having to be run more than once. In a way, this situation was analogous to ‘experimental error’ or ‘experimental limitations’ that can result during physical scientific testing, and the errors and omissions resulting from software restrictions were treated in the same way as experimental limitations would be treated in physical testing methods. Consequently, diurnal data was not available for each level of urine separation.

Regardless of which UCTPHO properties were tinkered with, successful steady state simulations could not be run above 90% urine separation, and full diurnal simulations could not be run above 70% urine separation.

4.2 Preliminary Testing

With the point of these simulations being to show the effects of urine separation on the quality of effluent from WWTPs, the actual effluent values should not be read into too much – especially those from the base WWTP setup with 0% urine separation. While every effort was made in the hand calculation steady-state setup to keep the effluent quality below acceptable limits, as set out in Section 2.3: “Current Objectives of Wastewater”, some of the effluent quality values from the steady state simulations in UCTPHO were above the effluent quality regulations. For convenience and efficiency, the base WWTP system was designed (via hand-calculations) under steady state conditions to meet the effluent standards, but with the increased complexity of the UCTPHO software simulations, meeting these effluent quality guidelines could not be guaranteed in these UCTPHO software simulations. However, the objective of these simulations is to show the *trends* in the effluent quality as urine separation is implemented to varying degrees, so the actual values (if they exceed the regulatory guidelines) should not be read into too much. It is very possible that a sub-optimum sludge age of 15 days (and also sub-optimum anoxic and anaerobic mass fractions) may have been selected initially (this was proven in the Primary Testing where the base case was optimised), but this might be reflective of the situation at current WWTPs in South Africa, so was not thought to be critical at this stage.

4.2.1 Capacity Increase

The relationship between urine separation and capacity increase appeared to be linear, as shown by Figure 26 below. At 100% urine separation, it was found that 123% of the original capacity could be accommodated. This equated to 167 568 people in total, an addition of 31 068 people from the 136 500 people in the original population.

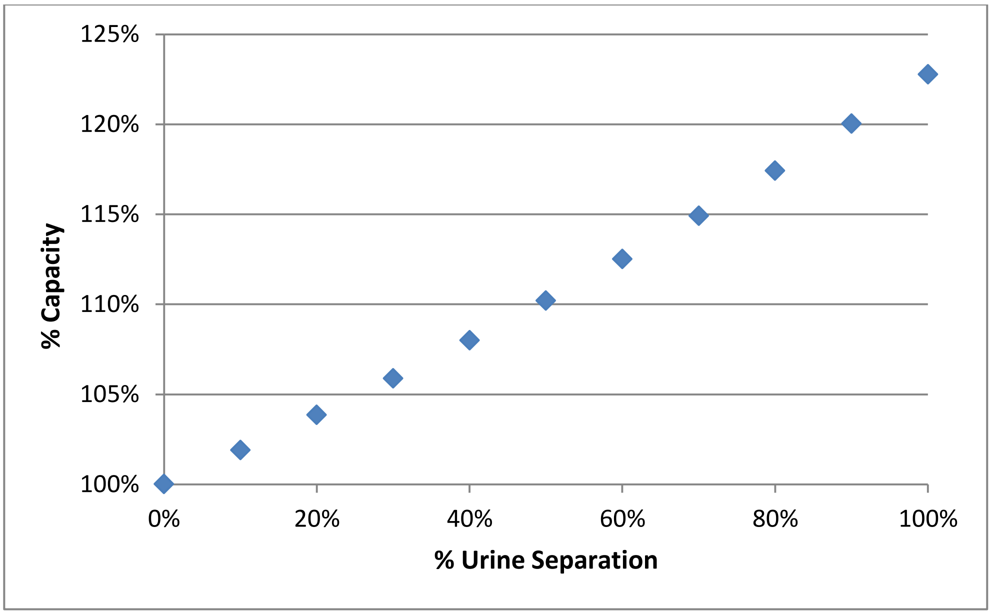

Figure 26: Graph showing percentage capacity increase (% of base population of 136 500) for an unchanged WWTP at each level of urine separation

The capacity increase could be calculated up to 100% urine separation because this was done by hand and as such didn't require simulation in UCTPHO software.

4.2.2 Aeration Requirements

The Oxygen Utilisation Rate for nitrification (OURn) dropped off from 28.6mgO/(ℓ.h) at 0% urine separation to 6.5mgO/(ℓ.h) at 80% urine separation for steady state testing, showing that in this unchanged WWTP system, nitrification has not been eliminated at 80% urine separation. According to Figure 27 below, nitrification can only be expected to be eliminated at almost 100% (extending the trend line of the OURn data points in blue below until the x-axis is reached). The steady carbonaceous oxygen utilisation rate (OURc below) was actually shown to increase slightly with increasing urine separation, although this wasn't expected, as the organic load (COD flux) was kept the same throughout this testing phase (by adding people – showing the capacity increase in this regard). However, the overall steady state oxygen utilisation rate (OURt below) was shown to decrease with increasing urine separation, and this was expected because OURt represents the combination of OURn and OURc.

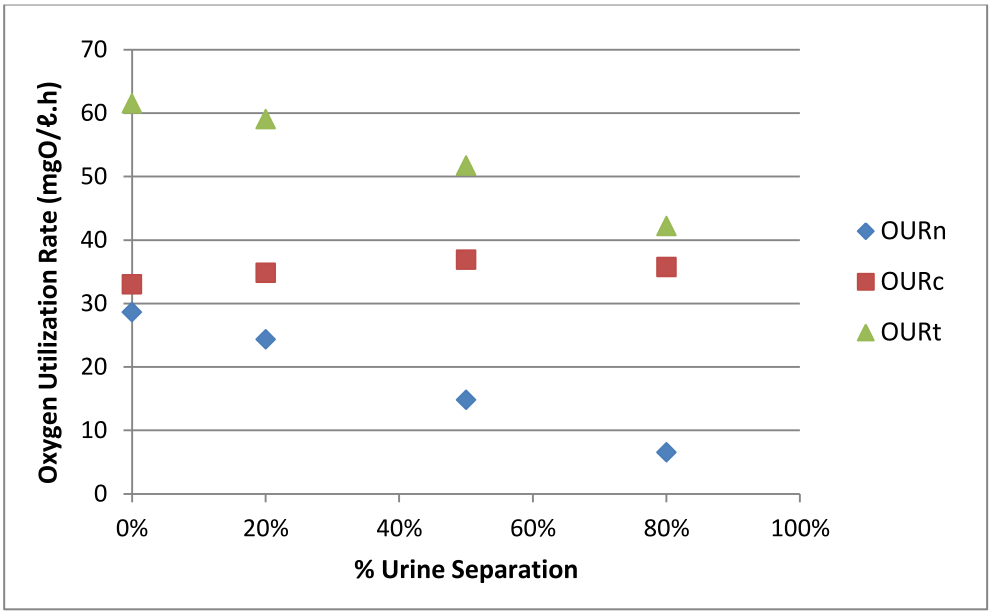

Figure 27: Graph showing the steady-state nitrification (OURn), carbonaceous (OURc) and total (OURt) oxygen utilization rates within the aerobic reactor of the optimised WWTP setup at each degree of urine separation.

This drop in OURt represents the direct saving potential with regards to reducing the aeration costs at an unchanged WWTP while still gaining increases in capacity.

4.2.3 Effluent Quality

In general, with this unchanged WWTP, it was found that increasing levels of urine separation exhibited steadily improving effluent quality (in terms of both N and P).

4.2.3.1 Effluent Ammonia

As shown by Equation 6 in Section 4.3.3.1 further on, the effluent Ammonia concentration is solely dependent on the system (sludge age and reactor mass fractions), and not on the influent TKN or Ammonia. Therefore for a fixed sludge age and reactor sludge mass distribution, the steady state Ammonia concentration in the effluent theoretically stays the exact same regardless of the level of urine separation. This situation is shown in Figure 28 below, where the steady state Ammonia concentration of the system was 1mgFSA-N/ℓ, as was determined in the steady-state hand-calculations as well. The diurnal maximum shows a significant decrease as urine is separated in greater degrees, dropping from almost 3.5mgFSA-N/ℓ at 0% urine separation down to 1.5mgFSA-N/ℓ at 80% urine separation. This would be expected to drop even further as 100% urine separation is approached, following the trend set before (up to 80% urine separation).

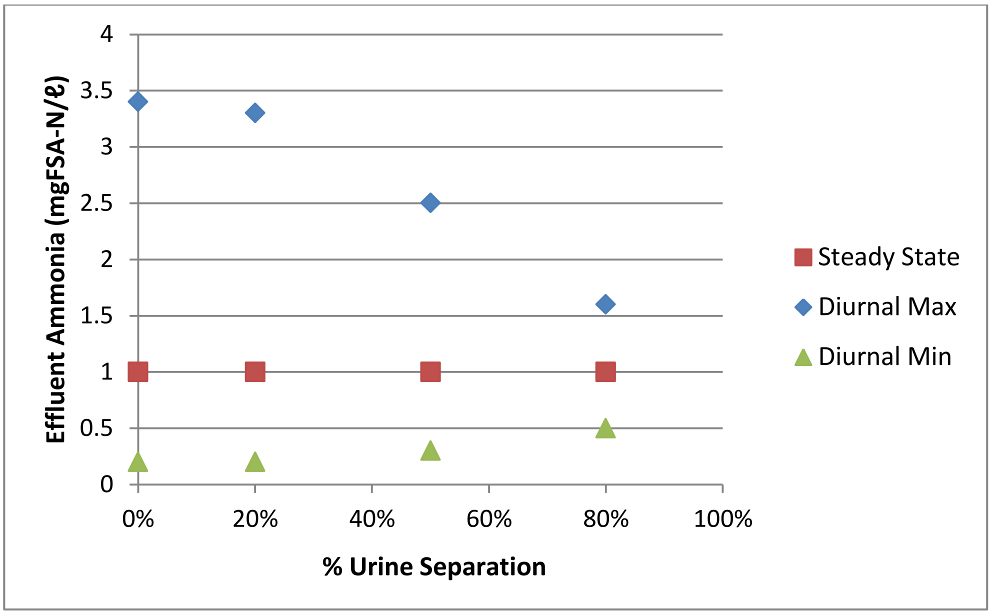

Figure 28: Graph showing the change in Ammonia concentration in the effluent of the unchanged WWTP at each level urine separation

Interestingly, the Diurnal minimum effluent Ammonia concentration increases as urine is separated in greater degrees. The reason for this is suspected to be that as urine is separated in greater degrees, so the maximum and minimum effluent FSA tend towards the steady state value.

4.2.3.2 Effluent Nitrate

The effect of keeping the setup of the WWTP the same throughout this testing phase was to effectively have a system that was increasingly over-designed (for N and P removal) with increasing urine separation. With the anoxic reactor volume and mass fraction staying the same throughout, there exhibited increasing surplus denitrification capacity with increasing urine separation. This manifested as lower steady state, diurnal maximums and diurnal minimums for Nitrate with increasing urine separation, and this is shown in Figure 29 on the next page. The steady-state effluent Nitrate concentrations dropped from almost 12mgNO_3-

N/ℓ at 0% urine separation down to less than 1mgNO_3-N/ℓ at 80% urine separation. If tests could have been run at higher urine separation levels, this trend would have been expected to continue.

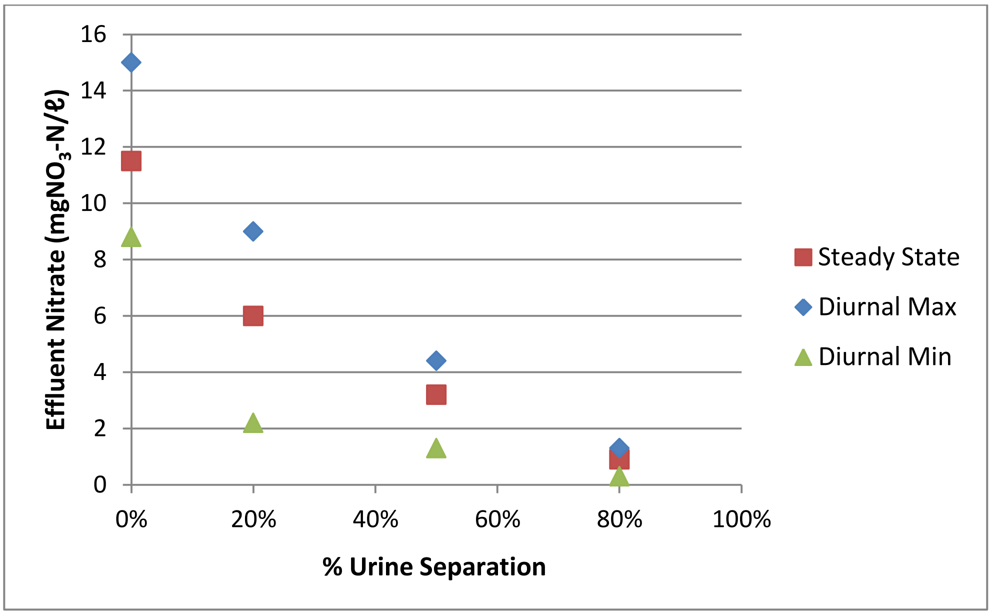

Figure 29: Graph showing the change in the effluent Nitrate concentration of the unchanged WWTP at each level of urine separation

In Figure 29 above, the steady state, diurnal max and diurnal min effluent Nitrate at 0% urine separation appear to be higher than the general (linear) relationship would suggest. This is because the 'a recycle' in the 0% urine separation base-case WWTP was adjusted (lowered) to achieve better effluent P concentrations as explained in Section 4.1.1 earlier. The result of this adjustment of the 'a recycle' in the 0% urine separation base-case WWTP was to increase the effluent Nitrate levels at this urine separation level because it caused the anoxic reactor to be under-loaded with Nitrate.

The combined effect of the effluent steady state Nitrate concentration decreasing and the effluent Ammonia remaining constant throughout was to represent a decrease in total effluent N (the sum of TKN and Nitrate) as urine was separated to greater degrees.

4.2.3.3 Effluent Phosphorous

As explained earlier, there was the unexpected and unavoidable situation where denitrification was occurring in the anaerobic reactor due to incomplete denitrification in the anoxic reactor at low levels of urine separation. The effect of denitrification occurring in the anaerobic reactor seemed to have the effect of causing high diurnal maximum effluent P concentrations. This denitrification thus seemed to occur during peak loading of influent N, and this was ratified by the base-case WWTP diurnal UCTPHO outputs shown in Appendix C.

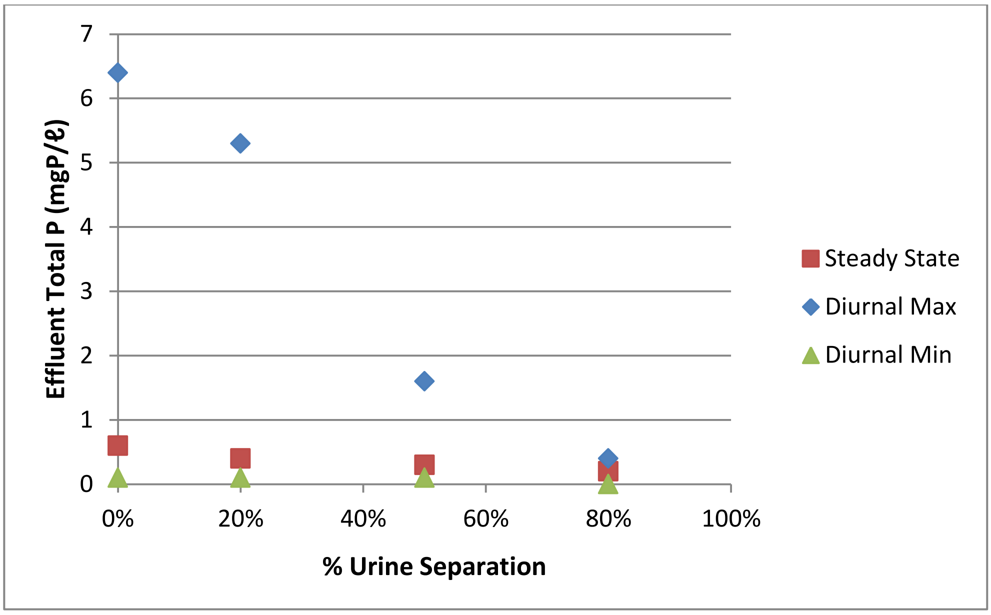

Figure 30: Graph showing the change in Total P in the effluent of the unchanged WWTP with increasing urine separation

The sharp decrease in the diurnal maximum effluent P values as shown in Figure 30 above was due to the denitrification rate dropping consistently in the anaerobic reactor as urine was separated in greater degrees. At around 50% urine separation, it appeared that denitrification no longer occurred in the anaerobic reactor, and the diurnal maximum effluent P concentrations beyond 50% urine separation were purely down to the diurnal fluctuations of influent P, and not due to Nitrate being recycled to the anaerobic reactor during peak N loading times.

The steady-state effluent P concentrations in this unchanged WWTP setup seemed to drop off slightly between 0% urine separation and 80% urine separation, with the steady state effluent P being 0.6mgP/ℓ at 0% urine separation and 0.2mgP/ℓ. At 80% urine separation, the steady state, diurnal max and diurnal min values have effectively converged, and gains in effluent P quality beyond 80% urine separation are not expected to any significant degree.

4.3 Primary Testing

In this primary testing phase, as explained earlier, the WWTP would be optimised (for N removal) at each degree of urine separation, changing the reactor mass fractions and sludge age. Note that even the base-case WWTP here was optimised, so that the results shown are compared to the optimised base-case WWTP with 0% urine separation, as opposed to the unoptimised base-case as used in the Preliminary Testing previously. The optimised base-case WWTP serves the same population as the unoptimised base-case WWTP previously modelled, but has an optimised sludge age and optimised distribution of reactor mass fractions (optimised for Nitrogen removal).

Figure 31 on the next page was produced by incrementally lowering the sludge age and determining the maximum influent TKN/COD ratio that can be handled by the system at that sludge age. According to Equation 5, a lower sludge age lowers the maximum necessary unaerated sludge mass fraction (and hence also lowers the maximum anoxic mass fraction). Decreasing the sludge age also increases the Nitrogen used for sludge production (by a small but not insignificant amount). In the Primary Testing Phase II it was attempted to find a sludge age and corresponding TKN/COD ratio that could be accommodated by system sludge removal of N only, and to show the corresponding urine separation level that could achieve this influent TKN/COD ratio. It was found that this situation occurred above 80% urine separation, i.e. at around 85% urine separation and above.

At every urine separation percentage (and corresponding influent TKN/COD ratio), there exists a sludge age (and corresponding anoxic mass fraction) that effectively gives a balanced UCT system (with a fixed anaerobic mass fraction and volume), where the anoxic reactor is exactly loaded to its denitrification potential by the maximum practical a-recycle ratio of 6:1. This situation is given by Figure 31 on the next page. The point of this testing was to drastically increase the capacity of the plant by increasing the aerobic reactor (as the required unaerated mass fraction decreases) with increasing urine separation. An increased aerobic mass fraction would thus have a lowered TSS concentration if the population was left unchanged, and increasing this population to restore the TSS concentration of the aerobic reactor back to its original value was what truly unlocked the capacity increase potential of this WWTP.

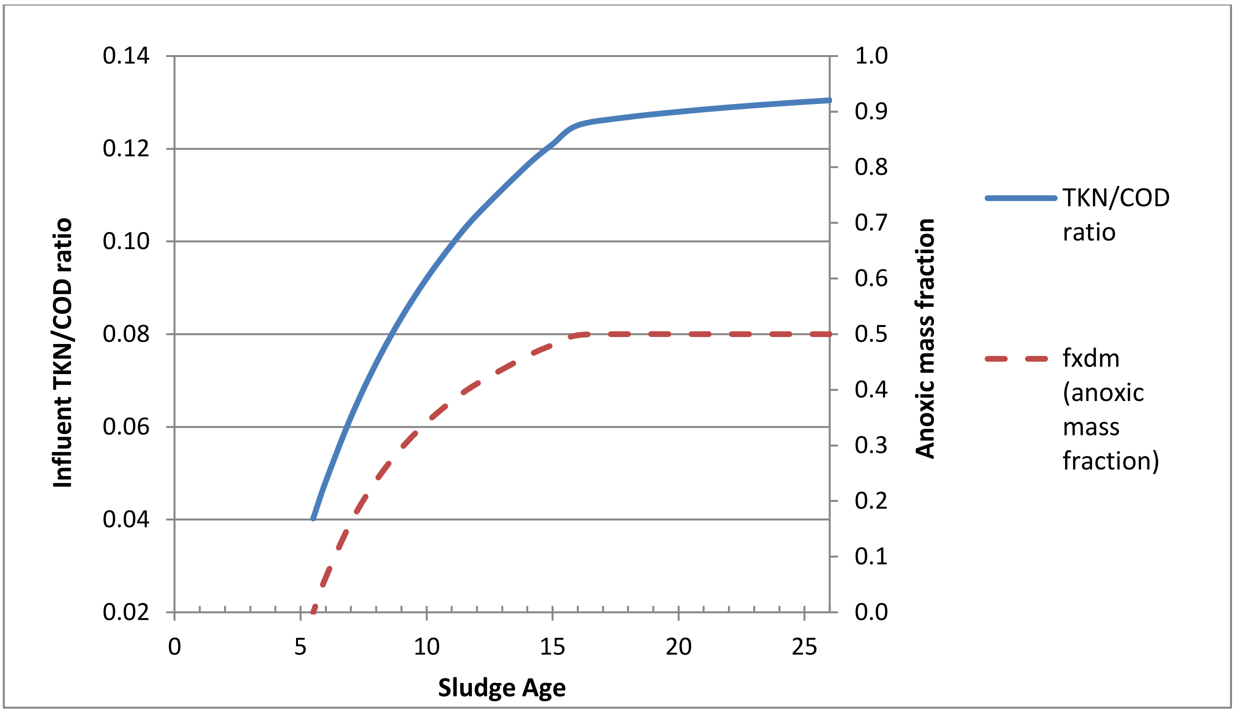

Figure 31: Influent TKN/COD ratio and maximum anoxic mass fraction (fxdm) for balanced UCT system with an anaerobic mass fraction of 0.1 and a 6:1 practical upper limit to the a recycle ratio for settled WW at 14°C

Henze et al. (2008) produced a similar graph as Figure 31 for a balanced Modified Ludzack Ettinger (MLE) system with ND only, i.e. without EBPR (and hence without an anaerobic reactor). The figure above applies specifically to the system parameters chosen by this author. The above graph is used by entering the graph (on the left vertical axis – "Influent TKN/COD ratio") with the influent TKN/COD ratio of the WW in question. Then, moving horizontally until the (solid blue) TKN/COD ratio line is reached, a vertical line is drawn down to the sludge age. Where the line passes through the (dashed red) fxdm line, this gives the anoxic mass fraction that corresponds with the sludge age that gives the optimal Nitrogen removal in a ND UCT system WWTP setup. The anaerobic mass fraction for the above graph is set at 0.1 and the aerobic mass fraction thus makes up the difference between the total mass fraction (1.0) and the maximum unaerated mass fraction (sum of anoxic and anaerobic mass fractions).

Table 4.2 on the next page shows the results of using Figure 31 to determine the optimum sludge age and anoxic mass fraction for each level of urine separation.

Table 4.2: WWTP setup parameters for optimum nitrogen removal in an NDEBPR AS system for different levels of urine separation

Influent		System		
Urine Separation Level	Influent TKN/COD ratio	Balanced UCT sludge age for optimal nitrogen removal (from Figure 31 above)	Maximum unaerated mass fraction	Optimum Fxdm – the anoxic mass fraction (with anaerobic mass fraction set at 0.1)
0%	0.106	12	0.51	0.41
10%	0.098	10.7	0.47	0.37
20%	0.090	9.7	0.43	0.33
30%	0.082	8.8	0.39	0.29
40%	0.074	8	0.34	0.24
50%	0.066	7.3	0.29	0.19
60%	0.058	6.7	0.23	0.13
70%	0.049	6	0.17	0.07
80%	0.041	5.55	0.11	0.01
90%	0.032	N/A	N/A	N/A*
100%	0.023	N/A	N/A	N/A*

*Note that for influent TKN/COD ratios of less than 0.04 (from above 80% urine separation), where the sludge age would have to be below 5.5 days, it was found that the anoxic mass fraction could no longer physically be provided (details of this are given below). Rather than changing the sludge age to some arbitrarily defined value, it was decided to set the sludge age at 5.5 days for simulations above 80% urine separation.

With the anaerobic mass fraction set at 0.1, and the maximum unaerated mass fraction calculated (using Equation 5) to be less than 0.1 for urine separation above 80%, there was physically no mass fraction remaining to be assigned for the anoxic reactor. However, the optimisation parameter results shown in Figure 31 and Table 4.2 are for systems which assume Nitrification and Denitrification, and it remained to be seen if it was possible to achieve the goals of Nitrogen removal without ND above 80% urine separation.

For urine separation levels above 80% (i.e. from 85% and up), a setup was modelled in which there were only 2 reactors, namely the anaerobic and aerobic reactors. This was done to determine whether the effluent quality goals could be achieved without an anoxic reactor (while adding people to keep the aerobic reactor concentration the same), and at what level of urine separation (and corresponding sludge age) this would be possible. This (two reactor system) was *attempted* with 80% urine separation, but it was found that nitrification was still taking place in the aerobic reactor, showing that the influent TKN/COD ratio was still too high for Nitrogen removal to be completed through sludge uptake of Ammonia alone.

The diurnal performance of a few chosen effluent parameters for 3 different setups is shown in Appendix C. The figures in Appendix C compare the UCTPHO outputs of the 0% urine separation unoptimised and optimised WWTPs and the optimised WWTP at 90% urine separation (the 2 reactor system). These graphics make for interesting comparisons and should be viewed at leisure, but the contents of the results contained in Appendix C are contained and summarised in the following sections.

4.3.1 Capacity Change

It was calculated that for 100% urine separation, 190 500 new people (i.e. from the original population of 136 500 to an estimated population of 327 000 people) could be connected to the WWTP due to the increased capacity. At 100% urine separation, this resulted in a population capacity of 234% of the original. With plant optimisation as explained in Section 4.3 (changing the sludge age and reactor mass fractions), there seems to be a linear relationship between the capacity of the plant and the % urine separation (up until the point where an anoxic reactor is no longer required), and this relationship is shown in Figure 32 below.

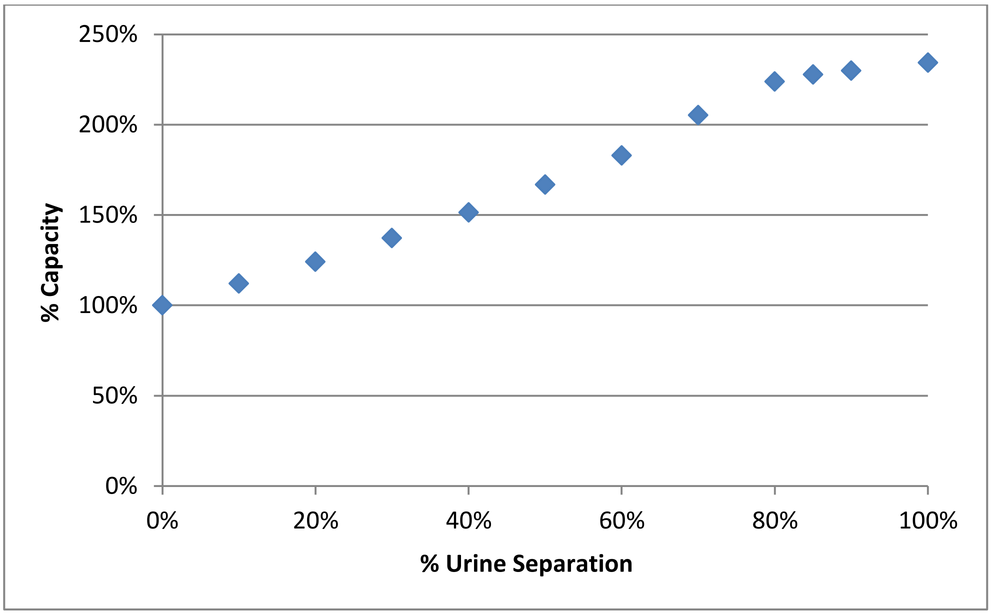

Figure 32: Graph showing percentage capacity increase (as a % of base population of 136 500) for an optimised WWTP at each level of urine separation

Admittedly, the capacity increase comes solely from the lower sludge age. However, the lower allowable sludge age comes from the lower TKN in the influent, which in turn comes from the increasing levels of urine separation. Where the sludge age was set at 5.5 days for 85% urine separation and above, the capacity-increase curve seems to level off a bit.

4.3.2 Aeration Requirements

A similar graph as Figure 27 of the aeration requirements in the preliminary testing phase was produced in Figure 33 below. This seemed to suggest similar trends as in the preliminary testing, where the OURn decreased as nitrification was eliminated to greater degrees (and then completely from 85% urine separation and upwards), while the OURc increased slightly and the OURt decreased as a combination of the OURn and OURc. However, this does not tell the full story of the aeration requirements at a WWTP that is optimised at every level of urine separation. The difference between this testing phase and the preliminary testing phase is that here, the volume of the aerobic reactor is steadily increased with increasing urine separation. The results in Figure 33 are for the hourly Oxygen Utilisation Rate (which is the total daily oxygen demand divided by the volume of the aerobic reactor times 24 hours/day) and therefore show a skewed perspective of the real situation.

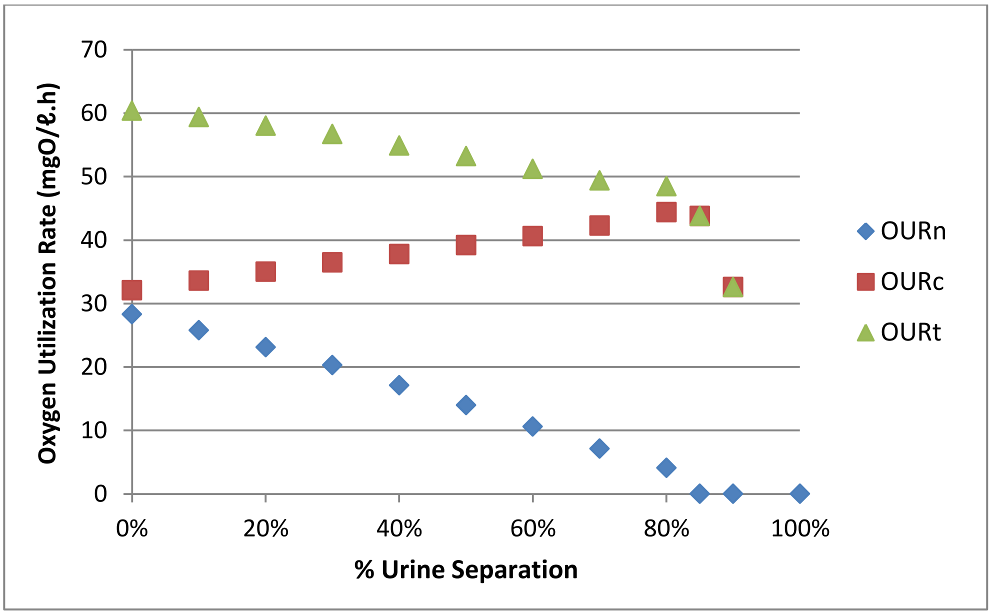

Figure 33: Graph showing the steady-state nitrification (OURn), carbonaceous (OURc) and total (OURt) oxygen utilization rates within the aerobic reactor of the optimised WWTP setup at each degree of urine separation.

While the results in Figure 33 above seem to suggest a drop in total aeration demands with increasing urine separation, Figure 34 on the next page highlights the real situation. Because the aerobic reactor is constantly increasing with increasing urine separation, the total oxygen demand (in kg/d) is actually increasing. This is expected, and is due to the vastly increased population being serviced as urine separation increases in this testing phase. The reason for the drop in the total Oxygen Utilisation Rate in Figure 33 (on the previous page) is because the aerobic reactor is being increased with increasing urine separation, masking the fact the total oxygen demand is actually increasing with increasing urine separation. This could easily have been an oversight, and would have falsely shown that urine separation has a positive downward effect on the total aeration requirements (and therefore aeration costs). There seems to be evidence from Figure 34 on the next page to suggest that after nitrification has been eliminated, the total oxygen demand could drop below the original oxygen demand. The peak oxygen demand was 9640kgO/d which occurred at 80% urine separation, and from then on (with nitrification eliminated) the total oxygen demand seemed to drop off, reaching 6530kgO/d at 90% urine separation. This was almost the same as the total oxygen demand of the base-case optimised WWTP at 0% urine separation, which is remarkable when it is acknowledged that the catchment population was increased to 234% of the original at 90% urine separation. The trend of urine separation beyond 80% seems to suggest that the total oxygen demand beyond 90% would actually drop below the original total oxygen demand, giving real aeration savings while treating higher populations. However, these simulated oxygen requirements should be viewed with caution above 85% urine separation, as the instability of UCTPHO at 85% urine separation (and above) could have influenced the oxygen requirement results shown below.

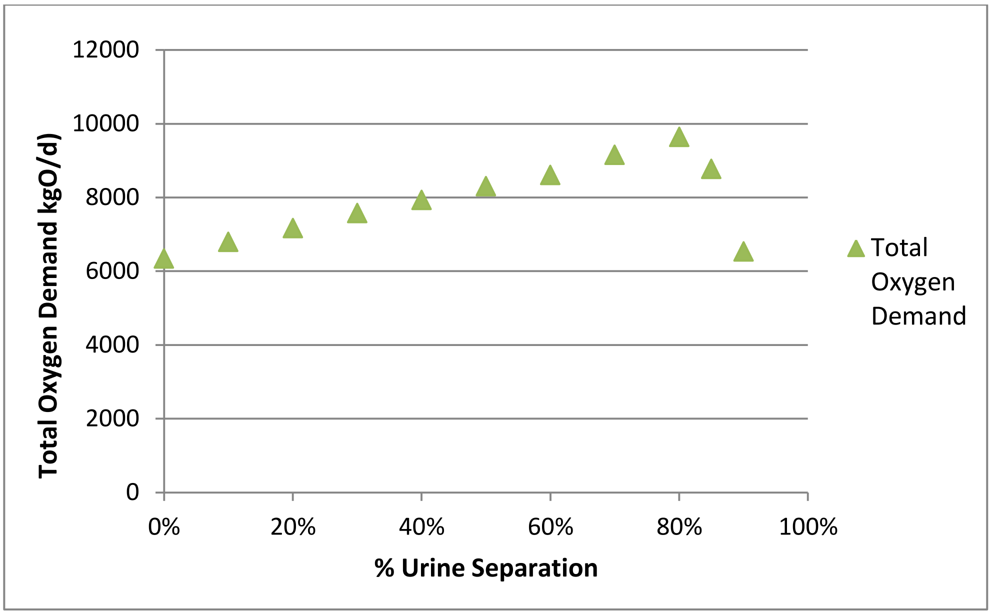

Figure 34: Graph showing the steady-state total oxygen demand in kgO/d, giving a truer reflection of the changing aeration requirements of the optimised WWTP at each level of urine separation.

4.3.3 Effluent Quality

4.3.3.1 Effluent Ammonia

In this testing phase, the steady state Ammonia effluent concentration stays virtually unchanged (at 2.2mgFSA-N/ℓ) for all levels of urine separation.

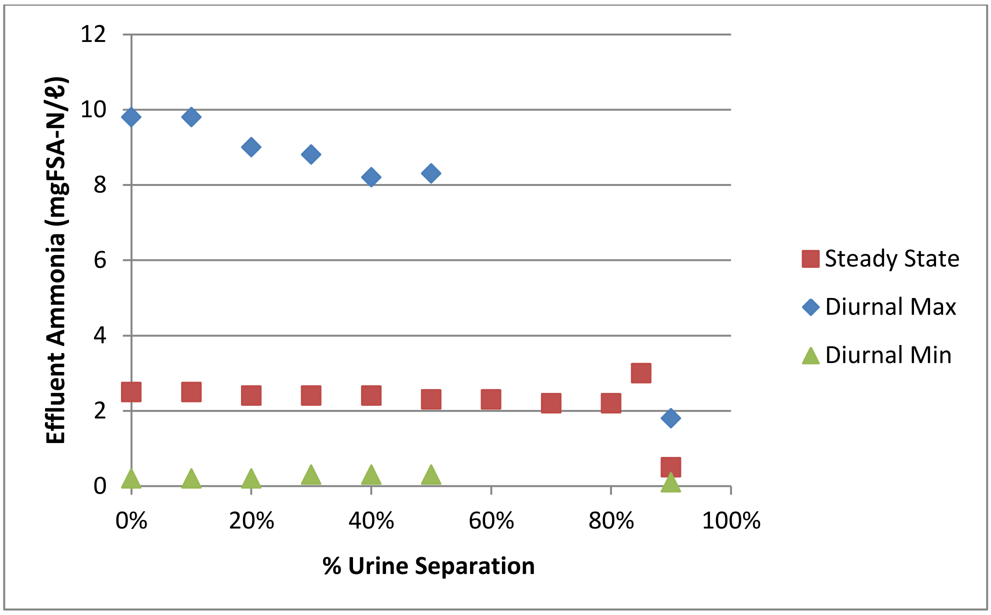

Figure 35: Graph showing the change in the effluent Ammonia concentration of the optimised WWTP at each level of urine separation

The reason the steady state effluent Ammonia concentration remains virtually unchanged was no coincidence and was in fact expected. The way that Figure 31 (shown earlier) was produced was to design on the limit of Nitrate and Ammonia removal. While the steady-state effluent Nitrate unexpectedly decreased with increasing urine separation (as shown in the next section) no anomalies existed that altered the steady-state effluent Ammonia concentration. The effluent (steady state) Ammonia concentration is given by Equation 6.

$$N_{ae} = \frac{K_{nT}\left(b_{AT} + \frac{1}{R_s}\right)}{\mu_{AmT}(1 - f_{xt}) - (b_{AT} + \frac{1}{R_s})} \qquad \textbf{Equation 6}$$

(Symbols are explained in the auxiliary Section “List of Symbols”.)

The effluent Ammonia is a function of both Sludge Age (R_s) and unaerated mass fraction (f_{xt}) if all other variables are kept constant in Equation 6 above. With the way the WWTP was optimised in this phase of testing, a reduction in sludge age brought about a corresponding increase in the unaerated mass fraction, such that the effluent Ammonia stayed constant at every level of urine separation.

Only where there was a disparity between the optimum sludge age and the chosen unaerated mass fraction (at 85% urine separation and above) was there a change in steady state effluent. From explanations before, for urine separation at 85% and above, a two-reactor system was modelled with a constant sludge age (5.5 days) and a constant reactor mass fraction distribution (anaerobic at 0.1 and aerobic at 0.9 of the total), such that nitrification (and hence no excess removal of Ammonia) was not facilitated.

As shown in Figure 35 before, there is a small but noticeable increase in the steady-state effluent Ammonia concentration at 85% urine separation. This consequence of the optimised system at 85% urine separation (where the two-reactor system was used for the first time) seems to suggest that nitrification was still necessary to remove Ammonia from the system, although this increase (of around 0.8mgFSA-N/ℓ) was not deemed catastrophic or significant enough to justify a complete rethink of the situation. The sharp

The effluent diurnal maximum Ammonia concentrations (as shown in Figure 35 on the previous page) are high relative to the steady-state effluent concentrations. The diurnal maximum effluent Ammonia concentration trend shows a slight decrease with increasing urine separation. It is probable that with this optimised WWTP setup (where it is designed 'on the edge' of Nitrogen removal) the combined rate of Nitrogen uptake via Nitrification and sludge growth is lower than the rate of Ammonification during peak loading periods. This would lead to the observed situation where the peak effluent Ammonia concentrations are so high. Also, Ammonification "switching function" of the OHOs was increased in UCTPHO, backing up the evidence to support this claim.

4.3.3.2 Effluent Nitrate

Theoretically, with the way the optimum sludge age and anoxic mass fraction for each level of urine separation were calculated, it was expected that the effluent Nitrate would stay roughly the same regardless of the level of urine separation. The reason for this initial thinking was that the 'optimised' WWTP for each degree of urine separation was constantly being configured to be right 'on the edge' with regard to denitrification. With the anoxic mass fraction being steadily reduced as the urine separation increased, the denitrification capacity decreased in line with the decreasing anoxic mass fraction. However, it was found that as with the Preliminary Testing phase, there appeared to be denitrification occurring in the anaerobic reactor, resulting in more Nitrate being removed than initially thought.

It was suspected that there was a similar discrepancy as in the Primary Testing phase, where it was postulated that the K_{2T} rate in the UCTPHO software had the effect of underestimating the denitrification potential of the anoxic reactor (when compared to the hand-calculations that produced the reference Figure 31) – leading to denitrification in the anaerobic reactor in UCTPHO.

Therefore it was interesting to note that in Figure 36 (on the next page), the trend of effluent Nitrate is downwards even though the anoxic mass fraction was being decreased for increasing urine separation levels. This was not an ideal set of results as it thus failed to isolate the effects of decreasing the anoxic mass fraction on the effluent Nitrate concentrations.

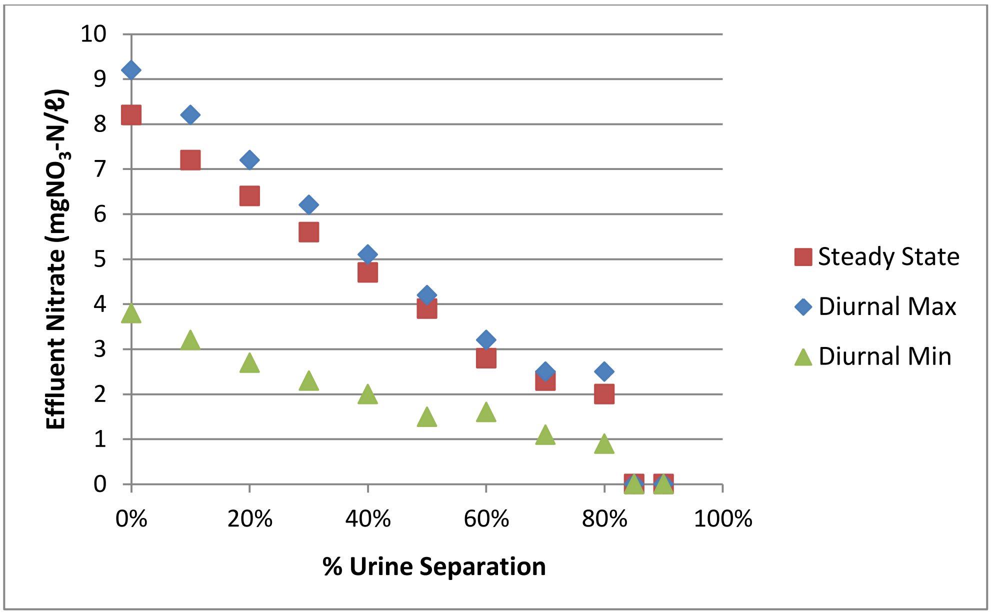

Figure 36: Graph showing the change in the effluent Nitrate concentration of the optimised WWTP at each level of urine separation

It is also unclear from the UCTPHO simulation software why the steady state effluent Nitrate concentrations are so close to the diurnal maximum effluent Nitrate concentrations. Clearly though, the need for nitrification was eliminated at upwards of 80% urine separation, where Nitrogen uptake by sludge production was sufficient to achieve full influent Nitrogen removal.

4.3.3.3 Effluent Phosphorous

As explained earlier, there was the unexpected and unavoidable situation where denitrification was occurring in the anaerobic reactor due to incomplete denitrification in the anoxic reactor (and hence recycling of Nitrate into the anaerobic reactor). Again, this was potentially down to the hand-calculations (that produced Figure 31 to give the optimised sludge age and unaerated mass fraction) overestimating the denitrification potential of the anoxic reactor when compared to the UCTPHO simulation software. Therefore the inputs from Figure 31 that were used in UCTPHO were possibly based on an overestimation which led to the discrepancy in UCTPHO and the subsequent occurrence of denitrification in the anaerobic reactor.

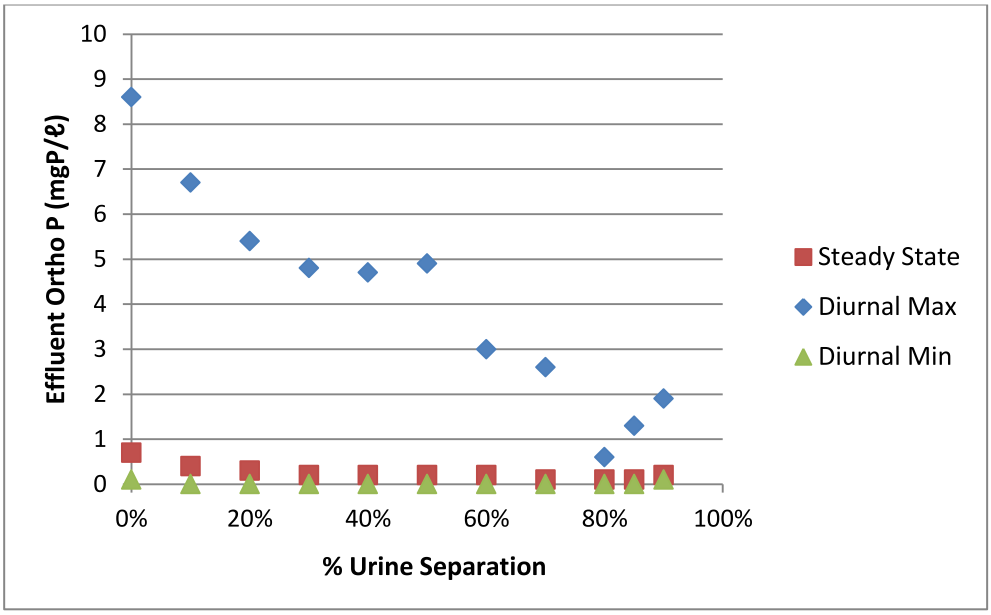

Figure 37: Graph showing the change in the effluent Ortho P* concentration of the optimised WWTP at each level of urine separation

(*The effluent Ortho P, as shown in Figure 37 above, roughly equals the effluent Total P as, according to Ekama (pers. comm., 15 Oct 2012), the P content of the unbiodegradable soluble organics (USOs) is zero. Therefore the term 'ortho P' here can be taken to mean Total P.)

The unexpected denitrification in the anaerobic reactor did not seem to have an effect on the effluent steady state Phosphorous concentration, as shown by the (red) data points in Figure 37 above. The peak effluent P concentrations (Diurnal Max data points as shown in Figure 37 above) were generally found to occur during the periods of peak nutrient loading. As urine was separated to greater degrees, the peak influent P load dropped considerably, and hence led to the drop in the diurnal maximum P effluent concentrations with increasing urine separation. This may have been assisted by the fact that the denitrification rate in the anaerobic reactor dropped consistently with increasing urine separation and thus had reduced interference on the EBPR processes.

In addition, as can be seen in Figure 38 on the next page, generally there is no change in total N and P percentage removal (percentage difference between influent concentration and effluent concentration) with increasing urine separation. An optimised UCT system WWTP can generally achieve the same percentage of N and P removal regardless of the level of urine separation. However, at 90% urine separation, there appeared to be a sharp change in the total percentage N removed, improving the overall N removed by almost 10%.

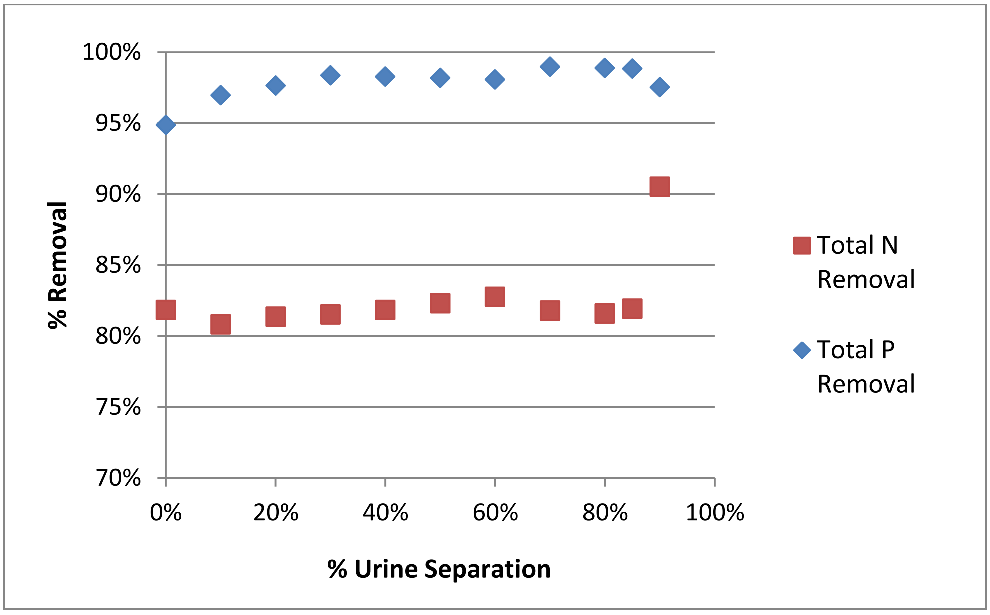

Figure 38: Graph showing the total percentages of N and P removal (at steady state) for each level of urine separation

4.3.3.4 Effluent COD

Increasing urine separation also had the effect of increasing the effluent COD concentration. This was simply due to the fact that increasing urine separation resulted in higher influent COD concentrations, meaning higher influent USOs. These USOs are untreatable by conventional WWTPs, and so increased urine separation inevitably resulted in higher effluent COD concentrations (COD as USOs). The steady state effluent COD concentration increased linearly from 60mgCOD/ℓ to 75.5mgCOD/ℓ between 0% and 90% urine separation (Figure 39).

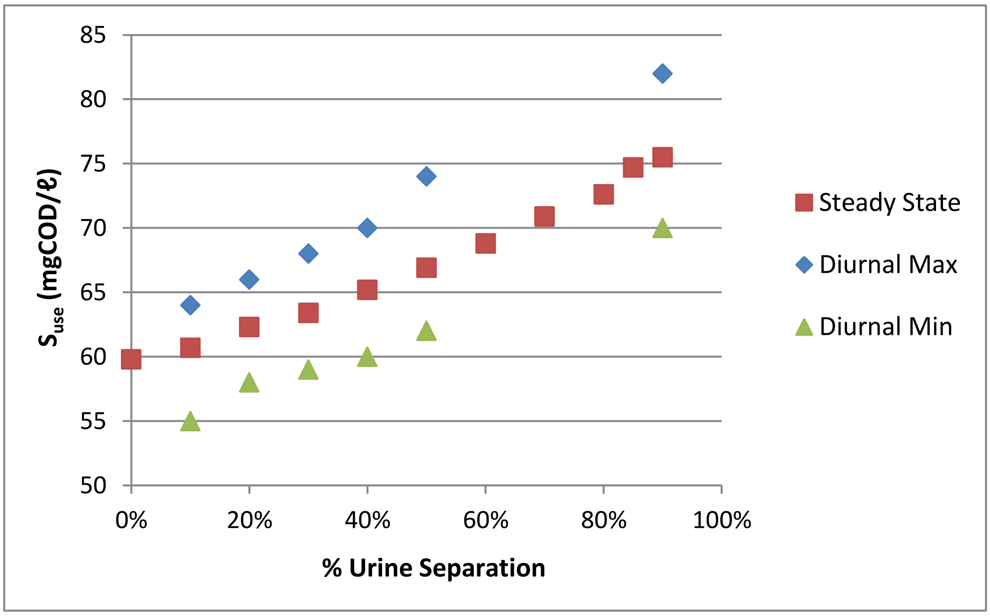

Figure 39: Graph showing increasing effluent COD with increasing urine separation

5. Chapter V - Conclusions

5.1 Reduced Hydraulic Load/Increased Capacity

Urine separation, (only) when combined with low-flush or dry separation toilets, will significantly reduce water consumption in South Africa by up to 20%. With the implementation of urine separation, there can be a lower expected WW flow at WWTP if the catchment population remains unchanged while the urine separation is implemented. This would either constitute a reduced hydraulic load (both on the WWTP and on the receiving water body) if the population is not increased (in a fully developed suburb or area for example), or this could constitute the potential for increased capacity (if people are added to the WWTP catchment area to keep the operating organic load or aerobic reactor concentrations the same at every level of urine separation).

For an unoptimised WWTP servicing a base load of 136 500 people at 15Mℓ/day, one can expect a linear capacity increase in the extra capacity to a maximum of 31 070 extra people (to a total of 167 570 people) and 13.4Mℓ/day at 100% urine separation.

For an optimised WWTP servicing a base load of 136 500 people at 15Mℓ/day, one can expect a semi-linear increase in the extra capacity to a maximum of 183 270 extra people (to a total of 319 770 people) and 25.4Mℓ/day at 100% urine separation.

It was thus found that increasing urine separation had the effect of profoundly increasing the capacity of a WWTP, and that only by optimising the WWTP can the real capacity increase potential be unlocked.

5.2 Changes in Effluent Quality

5.2.1 Ammonia

For both the unoptimised and optimised WWTP systems, a constant (for all intents and purposes) concentration of effluent Ammonia at steady state can be expected. For the modelled unchanged WWTP, this effluent Ammonia steady state concentration remained at roughly 1mgFSA-N/ℓ, while for the optimised WWTP, this effluent Ammonia remained at roughly 2.1mgFSA-N/ℓ for all levels of urine separation.

For the unoptimised WWTP, the diurnal maximum effluent Ammonia concentration was however highly sensitive to the different urine separation levels, showing a sharp decrease from 3.5 to 1.5mgFSA-N/ℓ between 0% urine separation and 80% urine separation.

The effect of the increasing urine separation on the diurnal maximum effluent Ammonia concentration in the optimised WWTP system was inconclusive, showing either a minor change or no change at all. Generally in this optimised WWTP system, the diurnal maximum effluent Ammonia was up to 4 times greater than the steady-state effluent Ammonia concentrations, and represented unacceptably-high peak effluent FSA concentrations in excess of 8mgFSA-N/ℓ.

It was thus found that increasing urine separation had little to no effect on the steady-state effluent Ammonia concentrations, with a decreasing effect on the diurnal maximum effluent Ammonia concentrations of an unchanged WWTP. The effect of increasing urine separation on the maximum diurnal effluent Ammonia concentration of an optimised WWTP was slight to negligible, although low effluent peak Ammonia concentrations were observed for the cases of urine separation levels above 85% (where the 2-reactor system was used).

5.2.2 Nitrate

For an unoptimised WWTP, the effluent Nitrate decreased as expected with increasing urine separation. The steady state effluent Nitrate concentration decreased from 11.5mgNO_3-N/ℓ to 0.9mgNO_3-N/ℓ between 0% and 80% urine separation, while the diurnal maximum effluent Nitrate concentration dropped from 15mgNO_3-N/ℓ to 1.3mgNO_3-N/ℓ between 0% and 80% urine separation.

What was less expected was the change in the effluent Nitrate concentrations for an optimised WWTP at increasing degrees of urine separation. The decrease in effluent Nitrate as urine separation was increased was deemed to be due to continued (and unavoidable) denitrification in the anaerobic reactor, due to a discrepancy between an unknown UCTPHO parameter or a combination of UCTPHO inputs.

In any case, when the need for N removal by ND was eliminated from the optimised WWTP setup at 85% urine separation and above (with a sludge age of 5.5 days), the effluent Nitrate concentration became zero for both the steady state and diurnal simulations.

It was thus found that increasing urine separation has a stark effect of decreasing the effluent Nitrate for an unoptimised WWTP. For an optimised WWTP, urine separation also had an unexpected decreasing effect on effluent Nitrate concentrations (which was due to denitrification occurring in the anaerobic reactor), while having the expected effect of decreasing the effluent Nitrate effluent concentrations to zero when the influent TKN/COD ratio was low enough to facilitate N removal without ND.

5.2.3 Phosphorous

The simulated WWTP systems were effectively optimised for N removal and not P removal, but it was thought that for the optimised WWTP, P removal would remain constant because the anaerobic mass fraction was kept constant. Low steady-state and diurnal maximum effluent P concentrations were expected, but this was not found. The high maximum diurnal effluent P concentrations were attributed to the interference of Nitrate with EBPR in the anaerobic reactor during peak nutrient (N and P) loading periods.

In reality, if the denitrification could be eliminated in the anaerobic reactor, it would be expected to see steady (and low) effluent Nitrate concentrations and greatly reduced effluent P diurnal maximum concentrations for all levels of urine separation.

In the two-reactor system, with urine separation above 80%, sharp increases in effluent P quality were observed – obviously showing the removal of denitrification interference.

It was thus found that urine separation had the effect of producing high diurnal maximum effluent P concentrations in the optimised WWTP when this was not initially expected.

5.2.4 Effluent COD

With increasing urine separation and the subsequent increase in influent COD as discussed in Section 3.3.3 "Effect of Urine Separation on Influent Data", inevitably the influent concentration of USOs would increase as well. These USOs are untreatable by conventional WWTPs and pass straight out with the effluent. The highest diurnal maximum effluent COD concentration was found at 90% urine separation (for the unoptimised plant) as 82mgCOD/ℓ. If this concentration is too high for discharge into the environment then post-treatment techniques may have to be used, such as maturation ponds for example.

Urine separation thus has a marked effect of increasing the effluent COD concentration in both unoptimised and optimised WWTPs.

5.3 Simpler WWTP Setup

The WWTP system could only be made simpler when the need for N removal by ND was eliminated and the anoxic reactor was eliminated. For an optimised settled WWTP, this was achieved at a urine separation level of 85%, with a sludge age of 5.5 days and influent TKN/COD ratio of 0.037mgTKN/mgCOD. In comparison, Mbaya (2011) found that for a settled WWTP system operating at a 5 day sludge age, a TKN/COD ratio of 0.025mgTKN/COD would need to be achieved in order to facilitate N removal without ND processes.

The removal of ND facilitated the simulation of a much simpler two-reactor WWTP system, with just an anaerobic and aerobic reactor and no inter-reactor recycles. This simplified setup is shown in Figure 40 below.

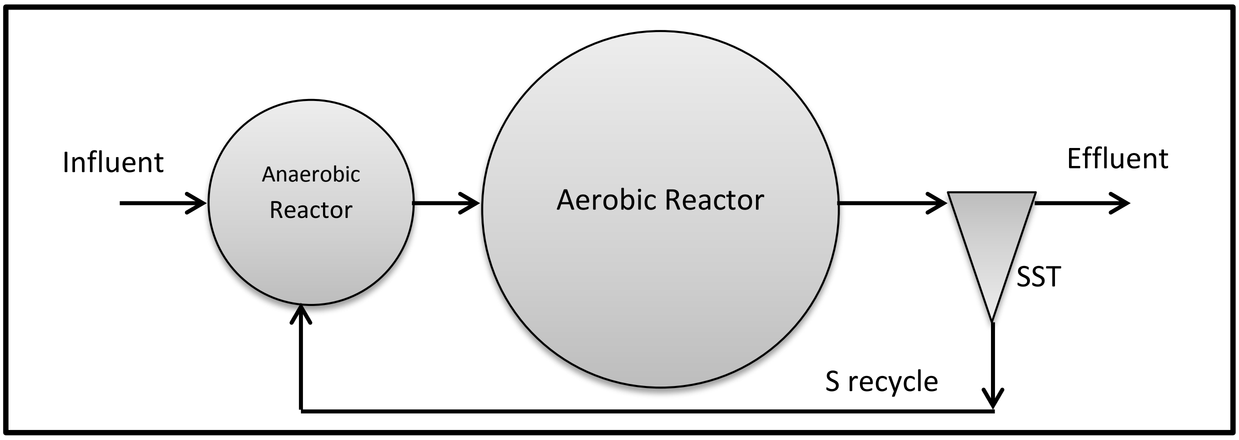

Figure 40: Simplified two-reactor WWTP system with the anoxic reactor eliminated

This two-reactor system would be much simpler to operate efficiently, as there are a lot fewer variables (inter-reactor recycles etc.) to consider. This would make these WWTPs easier to manage and could assist in closing the skills gap in the WW industry. The lack of skills necessary to operate complex BNR WWTP systems efficiently is a problem and is highlighted in Section 2.7 "Status of WWTPs in South Africa and Motivation for Improving WWTPs".

Through urine separation, simpler WWTP designs may be able to improve the correlation between the design and the operation of WWTPs by eliminating or reducing operator error.

5.4 Aeration Requirements

For the unoptimised WWTP, even when the catchment population is increased, there is a decrease in the overall daily oxygen demand (in kgO/d) and in the total oxygen utilisation rate (in mgO/ℓ.h). While the carbonaceous oxygen demand increased, the decreasing nitrification oxygen demand was sufficient to offset the carbonaceous oxygen increases and show a combined total oxygen demand that decreased with increasing urine separation. There was roughly a 50% decrease in the oxygen requirements of the unoptimised plant, between 0% urine separation and 80% urine separation, even while increasing the catchment population to 123% of the original population. The reductions in oxygen demand would have been greater had the population not been increased.

For the optimised WWTP, it was found that while the total daily oxygen demand actually increased with increasing urine separation and increasing population size. The peak total daily oxygen demand was found at 80% urine separation, and was 9640kgO/d, compared to 6330kgO/d required to treat the base population with 0% urine separation. However, these higher oxygen requirements were as a result of the increasing population with increasing urine separation.

5.5 Final Conclusion

Save for a few problems encountered with the denitrification problems in the optimised WWTP setup, the original hypothesis was largely proved correct. When not optimising a WWTP, the effluent quality improved in a direct relationship to the lower influent nutrient concentrations with increasing urine separation. The aeration requirements showed real decreases with increasing urine separation. However, the gains in capacity were not as significant when not optimising the WWTP. When simulating urine separation on an optimised WWTP, the gains in capacity are significantly higher than when not optimising the plant. The gains in effluent quality were not as significant in the optimised WWTP as in the unoptimised WWTP, as the optimised WWTP was configured to be ‘on the edge’ with respect to nutrient removal. However, some denitrification in the anaerobic reactor resulted in unexpected improvements in the effluent nitrate of the optimised WWTP and high peak P effluent concentrations. The aeration requirements showed decreases in terms of the oxygen utilization rate, but showed increases in terms of the real mass of oxygen required per day. However, these aeration demand increases were a direct result of the massive gains in capacity and increase of catchment population size with the optimised WWTP. Above 80% urine separation, a two-reactor system could be implemented, making the WWTPs significantly simpler, making them easier to build and operate efficiently.

6. Chapter VI - Recommendations

6.1 General Recommendations

It would perhaps be egotistic to suggest that urine separation should or shouldn't be implemented based on the simulation experiments and results of this undergraduate thesis. However, the potential that urine separation has as a method for furthering sustainability, saving water and preventing environmental damage has been shown through the literature review and experimentation of this research. The simulations performed in this thesis apply to a specific hypothetical population, influent data and WWTP setup, and as such the results and conclusions gleaned from these simulations should not be blanketed over all communities and all WWTPs. For example, this research showed that at above 80% urine separation, there appears to be no need for nitrification, but this figure may not apply in all cases.

Any degree of urine separation will present the benefits to WWTPs shown in Section 5 "Conclusions" above. However, the 'sweet spot' of these benefits, where it will no longer be necessary to accommodate ND bio-processes, happens at a relatively high degree of urine separation (around 80-85%). This 'sweet spot' where ND is no longer necessary brings exponentially-better benefits. The operational complexity, for example, is reduced massively, as a simple 2-reactor system can be used instead of a 3-reactor UCT system. However, for the most part, the risks involved with not accommodating the nitrification process could outweigh the potential benefits. With diurnal influent patterns, where the max TKN in the influent can be 1.5-1.8 times as much as the average, even if there is no nitrification at steady state, there is a good chance that there will still be nitrification with dynamic operation and unexpectedly high random peak loading times. If nitrification is not facilitated, then high effluent Ammonia concentrations can be expected (which is what happened in the two-reactor setup that was modelled at 85% urine separation), and this would be toxic for receiving water bodies.

Urine separation could either be implemented at unoptimised WWTPs to significantly improve the effluent quality while increasing the capacity to a lesser extent, or implemented at WWTPs that will be optimised at each urine separation level to give small benefits to the effluent quality and larger capacity increases. The main benefit of urine separation in South Africa would be the increased treatment capacity, which would help alleviate some of the capacity-related problems outlined in Section 2.7 "Status of WWTPs in South Africa and Motivation for Improving WWTPs" and prevent the need for the costly construction of new WWTPs or upgrades to existing WWTPs. If the WWTPs are to be optimised to increase the capacity of WWTPs and this capacity was to be filled by accommodating a larger catchment population, then better effluent quality cannot be expected in any great quantity – especially if denitrification is not facilitated in the anaerobic reactor (which was unfortunately observed in the simulations performed). If urine separation were to be implemented, it would certainly be worth investing in reconfiguring existing WWTPs. Depending on the level of urine separation achieved, the layout of existing BNR WWTPs would need to be altered to reduce the sludge age (maintain the same total volume and waste more sludge per day) and reduce the anoxic mass fractions (by aerating parts of the anoxic reactors to turn them into aerobic zones instead). At new WWTPs, the design volume of the reactors could be significantly reduced (in order to lower the sludge age) if based on this principle. However, the trade-off with designing lower reactor volumes on this principle would thus be that the serviceable population size would not be maximised, so this thinking could only be used in developed suburbs/areas where the population would not be expected to increase significantly.

The capacity-related benefits of urine separation would need to be weighed up against the cost of implementing urine separation to society. A system of urine collection and decentralised treatment would be expensive to implement, manage and maintain. With many of the existing WWTPs in South Africa currently not performing to desired standards due to a lack of skills, it is questioned whether constructing even more (urine) treatment facilities is a good idea. The labour and skills required to operate these decentralised urine treatment facilities would need to be secured (without comprising the skilled labour already operating in the WW industry) in order for urine separation to be a viable idea. Significant buy-in from the government and investment from municipalities would be required, and as such this idea would need to be sold on the ideas of social and environmental improvement, as well as reduced long term costs to government.

Also, as a side note, freshwater could only be saved with the implementation of urine separation technology if this technology is accompanied by the implementation of low-flow or flush-less toilets. Urinals would need to be fitted with flush-less technology and urine diversion toilets would need to ensure that the urine diversion compartment uses little to no water (obviously faecal matter would always need to be flushed). The implementation of urine separation technology provides the perfect opportunity to retrofit many toilet facilities with low-flow flush systems and flush-less water saving devices. If this type of low-flow sanitation system is installed along with urine separation, a freshwater saving of up to 20% could be realised. This would represent a significant relief for the strained freshwater resources and infrastructure in this country.

6.2 Further research, testing and improvements

It would be interesting to see the effect of balancing tanks on the peak effluent quality, and whether this would be economical to implement. As explained above, one of the problems with restricting denitrification and optimising WWTPs 'on the edge' is that during peak nutrient loading, it was found that denitrification was simply taking place in the anaerobic reactor. This interfered with EBPR and caused high diurnal peak P effluent concentrations. Also during peak nutrient loading times high effluent Ammonia concentrations were found. It would be interesting to see whether it would be economical to construct balancing tanks to regulate the flow to prevent these peak nutrient loading periods on the basis of peak effluent P and Ammonia concentrations.

It would also be interesting (although not perfectly practical) to perform physical batch tests of different urine separation levels and optimised physical WWTP models and compare these to the results of the simulations conducted in this thesis.

Another recommendation would be that for further research in this field, different simulation software should be used, as it would be remiss of this author not to point out the potential frustration that future researchers will go through if they use UCTPHO on Windows 7 or later operating systems.

In light of the motivation for this research and the topic of combining urine separation with seawater flushing as discussed in the Literature Review, it would be interesting to do further research on nutrient recovery from WWTPs and decentralised urine treatment plants. Also, the technology required to treat urine at decentralised urine treatment plants would make for intriguing research. Another interesting topic for further research could involve combining urine separation with seawater toilet flushing.

If the opportunity presented itself, this author would certainly be interested in pursuing further research in these fields.

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Appendix A – WW Contributions of Population

This data, particularly column 3 “Yellow (water)”, was manipulated to show changes in influent data with increasing urine separation.

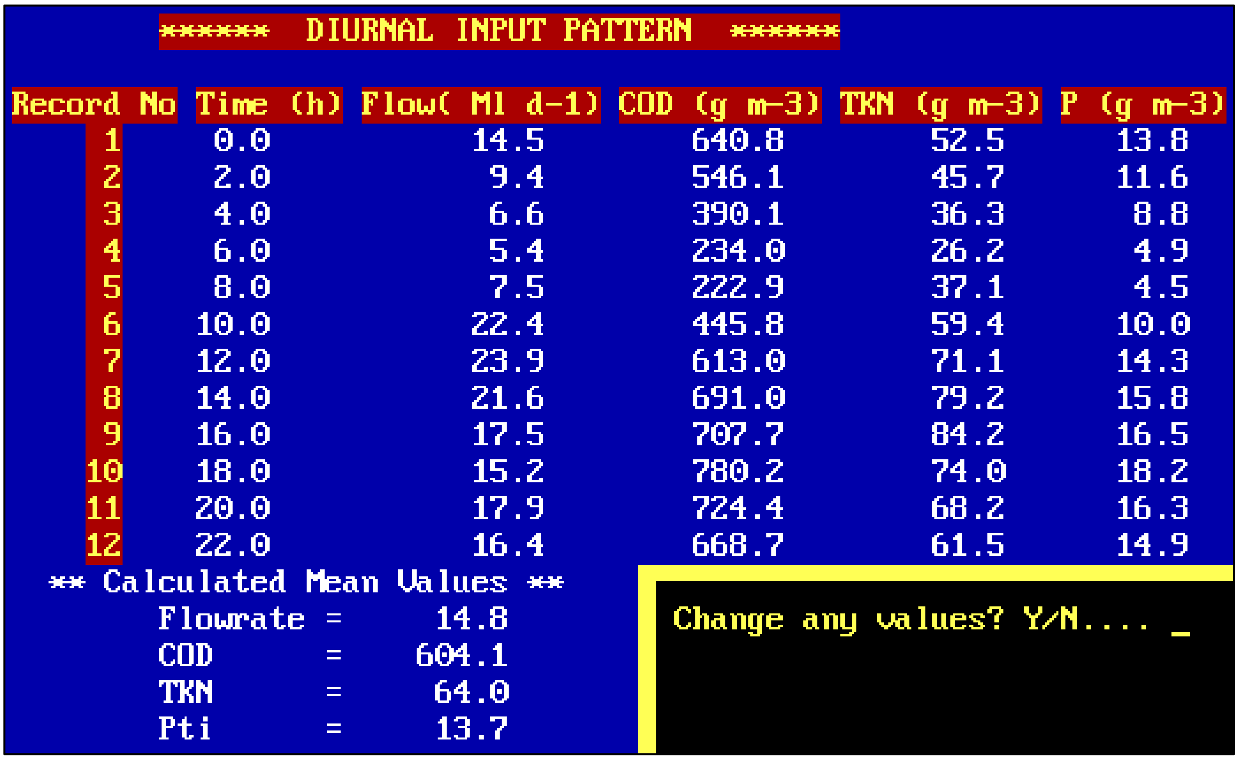
Table A.1: Daily ‘per person’ contributions for 0% urine separation

Constituent/person	Unit	Yellow	Brown	Grey	Infiltration	Industry	Total	
Water	ℓ/d	30	8	62	2	7.89	109.9	(These values were all chosen as inputs)
VFA	gCOD/d	0	2	2	0	2.35	6.35	
FBSO	gCOD/d	4.2	5	5	0	1.95	16.15	
USO	gCOD/d	0	1	1	2	2	6.00	
SetBPO	gCOD/d	0	13.3	5.5	0	3.95	22.75	
NonSetBPO	gCOD/d	0	28	4	0	3.75	35.75	
SetUPO	gCOD/d	0	5	5	0	1.05	11.05	
NonSetUPO	gCOD/d	0	0.5	0	0.4	1.05	1.95	
TOTAL COD	gCOD/d	4.2	54.8	22.5	2.4	16.1	100.00	(Calculated)
FSA	gFSA-N/d	5.5	0	0.2	0	0	5.70	(Chosen)
OrgN	gOrgN-N/d	0.07	1.42	0.65	0.06	0.39	2.59	(Calculated)
TKN	gTKN-N/d	5.57	1.42	0.85	0.06	0.39	8.29	(Calculated)
OP	gOP-P/d	0.9	0	0.3	0	0.06	1.26	(Chosen)
OrgP	gOrgP-P/d	0.02	0.26	0.14	0.01	0.07	0.50	(Calculated)
TP	gTP-P/d	0.92	0.26	0.44	0.01	0.13	1.76	(Calculated
SetVSS	gVSS/d	0.00	12.11	6.99	0.00	3.30	22.40	(Calculated)
NonSetVSS	gVSS/d	0.00	18.72	2.63	0.27	3.17	24.79	(Calculated)
TotVSS	gVSS/d	0.00	30.83	9.61	0.27	6.47	47.19	(Calculated)
SetISS	gISS/d	0.00	0.00	1.97	3.09	0.64	5.70	(Calculated)
NonSetISS	gISS/d	0.00	0.00	1.03	1.61	0.34	2.98	(Calculated)
TotISS	gISS/d	0	0.00	3.00	4.70	0.98	8.68	(Calculated)
TSS	gTSS/d	0.0	30.8	12.6	5.0	7.5	55.9	(Calculated)

Table A.2 Diurnal Flow and Loading Patterns for Raw Wastewater

Time (hrs)	Flow %	COD %	FSA %	OP %	InorgSS %
	%ADWG	%Ave	%Ave	%Ave	%Ave
0	97	115	80	109	100
2	63	98	77	92	85
4	44	70	61	70	55
6	36	42	46	39	35
8	50	40	70	35	55
10	150	80	108	80	90
12	160	110	126	115	120
14	145	124	140	127	130
16	117	127	150	133	140
18	102	140	125	147	150
20	120	130	115	130	130
22	110	120	103	119	110
24	97	115	80	109	100
Flow-weighted Average	100	100	100	100	100

Note that in the above table, the values are as percentages of the average, not discrete values.



******* DIURNAL INPUT PATTERN *******

Record No	Time (h)	Flow(Ml d-1)	COD (g m-3)	TKN (g m-3)	P (g m-3)
1	0.0	14.5	640.8	52.5	13.8
2	2.0	9.4	546.1	45.7	11.6
3	4.0	6.6	390.1	36.3	8.8
4	6.0	5.4	234.0	26.2	4.9
5	8.0	7.5	222.9	37.1	4.5
6	10.0	22.4	445.8	59.4	10.0
7	12.0	23.9	613.0	71.1	14.3
8	14.0	21.6	691.0	79.2	15.8
9	16.0	17.5	707.7	84.2	16.5
10	18.0	15.2	780.2	74.0	18.2
11	20.0	17.9	724.4	68.2	16.3
12	22.0	16.4	668.7	61.5	14.9

** Calculated Mean Values **
Flowrate = 14.8
COD = 604.1
TKN = 64.0
Pti = 13.7

Change any values? Y/N.... _

Figure 41: An example of the diurnal UCTPHO input for 0% urine separation

Appendix B – List of Constants

Table B.1: Table used to determine influent WW characteristics

Fraction	VFA	FBSO	USO	BPO	BPO	UPO	UPO
	Soluble acetic	Soluble biodeg	Soluble unbio	Set bio	NonSet bio	Set unbio	NonSet unbio
f_{cv}	1.0667	1.42	1.493	1.523	1.523	1.481	1.481
f_c	0.4	0.471	0.498	0.498	0.498	0.518	0.518
f_n	0	0.0231	0.0258	0.035	0.035	0.1	0.1
f_p	0	0.0068	0	0.0054	0.0054	0.025	0.025

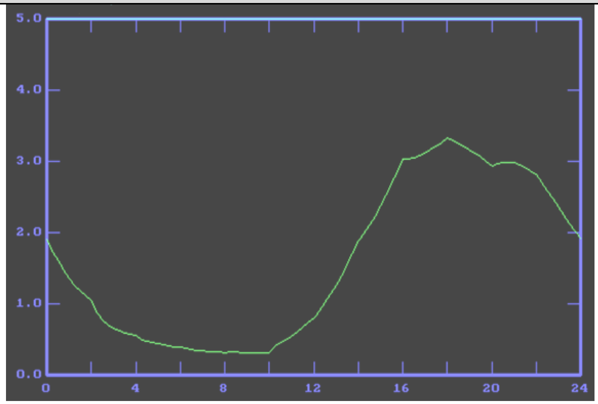
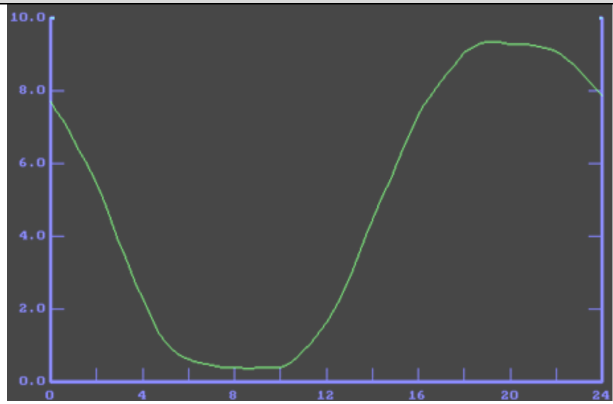
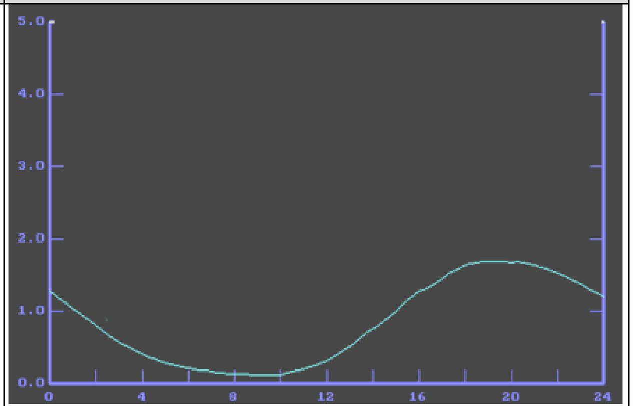
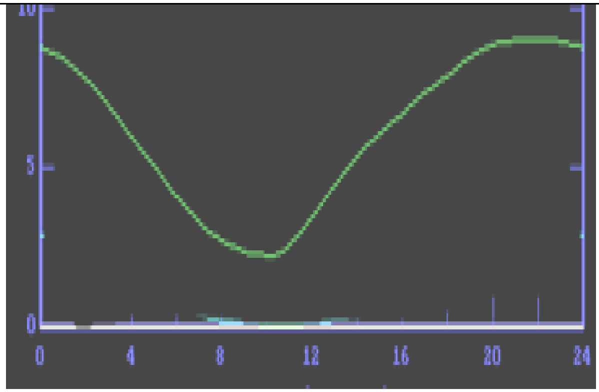
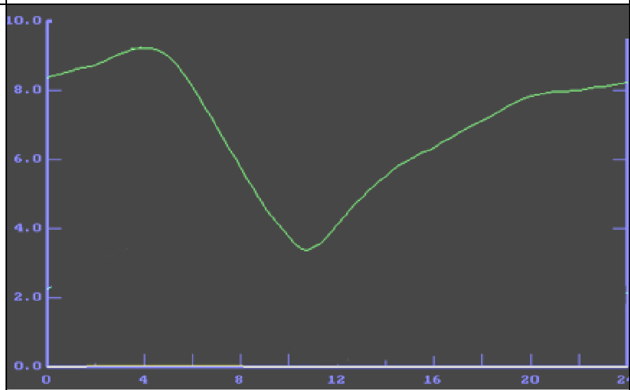
Table B.2: List of various constants used for UCTPHO and hand calculations

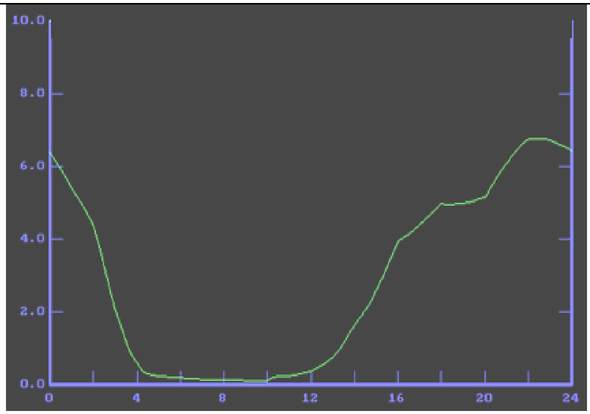
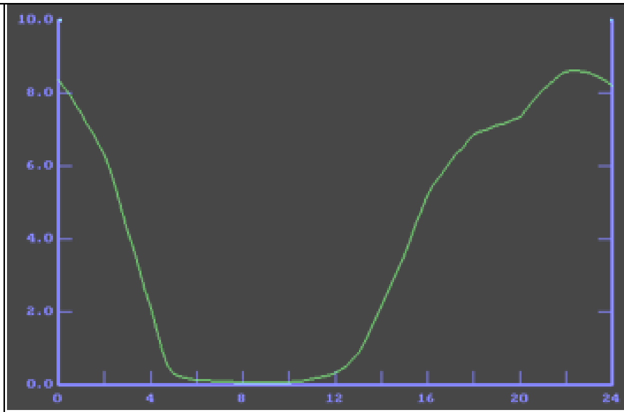
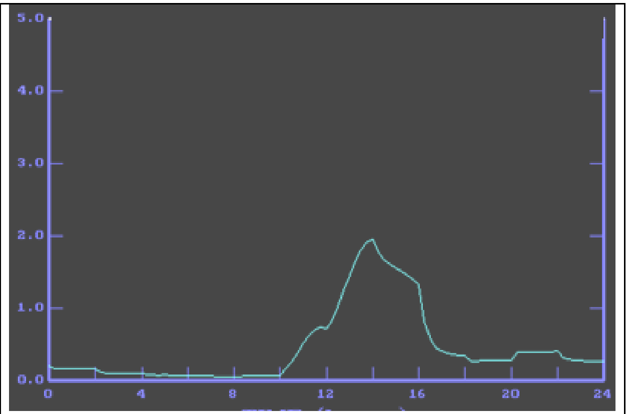
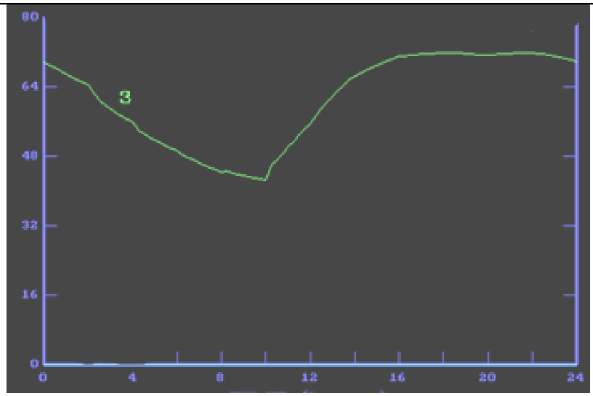
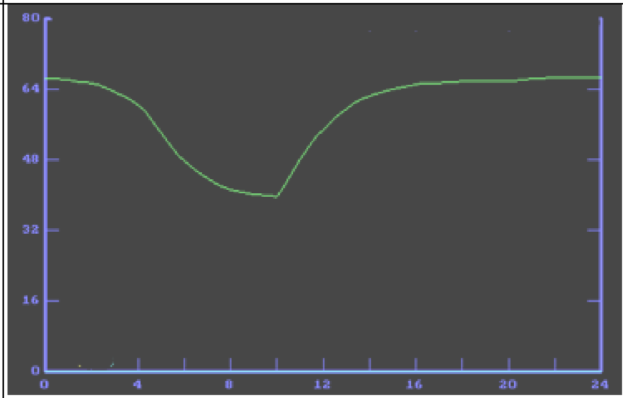
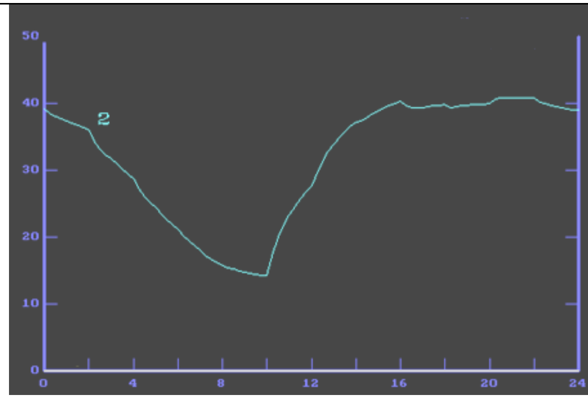
Constant	Value	Unit
Various:		
S_f (safety factor)	1.25	-
T (temperature)	14	°C
s (sludge underflow recycle)	1	Ratio x:influent flowrate
No. of anaerobic *compartments* (not reactors)	3	no.
O_a (DO in a recycle)	2	mgO/ℓ
O_s (DO in s recycle)	1	mgO/ℓ
b recycle (between anoxic and anaerobic)	1	Ratio x:influent flowrate
OHOs:		
Y_h (yield coefficient)	0.45	mgVSS/mgCOD
f_{iOHO} (ISS content)	0.15	fraction
f_h (endogenous residue fraction)	0.2	fraction
b_{h20}	0.24	/d
b_{h14}	0.202	/d
ANOs:		
Y_h	0.1	mgVSS/mgFSA
U_{am20} (maximum specific growth rate)	0.6	/d
U_{am14}	0.299	/d
b_{a20}	0.04	/d
b_{a14}	0.034	/d
K_{n20}	1	mgFSA/ℓ
K_{n14}	0.499	mgFSA/ℓ

Constant	Value	Unit
PAOs:		
Y_g	0.45	
f_g	0.25	
F_{xbgpp} (polyP content)	0.355	
Denitrification Rates		
$K_{1,20}$	0.072	$mgNO_3$-N/mgOHOVSS.d
$K_{1,14}$	0.241	$mgNO_3$-N/mgOHOVSS.d
$K_{2,20}$	0.101	$mgNO_3$-N/mgOHOVSS.d
$K_{2,14}$	0.064	$mgNO_3$-N/mgOHOVSS.d
Fermentation Rate at 20°C	0.0505	ℓ/mgOHOvss.d

Appendix C – Performance of 2-Reactor System at 90% urine separation compared to optimised and unoptimised base-case WWTPs

The performance of different effluent quality variables (UCTPHO outputs) is compared for 3 WWTP configurations and 2 urine separation levels (0% and 90%) below (note the different scales for the different WWTP configurations – some inconsistencies were unavoidable):

	0% urine separation WWTP unoptimised	0% urine separation WWTP optimised	90% urine separation 2-reactor system
Na mgN/ℓ			
NO_3 mgN/ℓ			There was no Nitrate in this setup as there was no nitrification in this 2-reactor system.

P_s mgP/ℓ			
OURt mgO/ℓ.h			

The contrast and brightness of the above images have been altered to ensure maximum visibility of the curves and axes.