Post-treatment technologies for Integrated Algal Pond Systems

THESIS

Submitted in fulfilment of the

requirements for the degree of

MASTER OF SCIENCE

(Environmental Biotechnology)

Rhodes University

Grahamstown

by

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November 2014

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Abstract

Integrated Algae Pond Systems (IAPS) are a derivation of the Oswald designed Algal Integrated Wastewater Pond Systems (AIWPS®) and combine the use of anaerobic and aerobic bioprocesses to effect wastewater treatment. IAPS technology was introduced to South Africa in 1996 and a pilot plant designed and commissioned at the Belmont Valley WWTW in Grahamstown. The system has been in continual use since implementation and affords a secondarily treated water for reclamation according to its design specifications which most closely resemble those of the AIWPS® Advanced Secondary Process developed by Oswald. As a consequence, and as might be expected, while the technology performed well and delivered a final effluent superior to most pond systems deployed in South Africa it was unable to meet The Department of Water Affairs General Standard for nutrient removal and effluent discharge. The work described in this thesis involved the design, construction, and evaluation of several tertiary treatment units (TTU's) for incorporation into the IAPS process design. Included were; Maturation Ponds (MP), Slow Sand Filter (SSF) and Rock Filters (RF). Three MP's were constructed in series with a 12 day retention time and operated in parallel with a two-layered SSF and a three-stage RF. Water quality of the effluent emerging from each of these TTU's was monitored over a 10 month period. Significant decreases in the chemical oxygen demand (COD), ammonium-N, phosphate-P, nitrate-N, faecal coliforms (FC) and total coliforms (TC) were achieved by these TTU's. On average, throughout the testing period, water quality was within the statutory limit for discharge to a water course that is not a listed water course, with the exception of the total suspended solids (TSS). The RF was determined as the most suitable TTU for commercial use due to production of a better quality water, smaller footprint, lower construction costs and less maintenance required. From the results of this investigation it is concluded that commercial deployment of IAPS for the treatment of municipal sewage requires the inclusion of a suitable TTU. Furthermore, and based on the findings presented, RF appears most appropriate to ensure that quality of the final effluent meets the standard for discharge.

Acknowledgements

Firstly I would like to thank Andile Magaba and Olwethu Baba for giving me the support when building my structures.

Thank you to the Water Research Commission (WRC) for the financial support. Your help was greatly appreciated.

I would also like to thank Gila Eustace, Michelle Isaacs, Richard Laubscher and Bonga Zuma for their small roles they played in helping me overcome some hard times during the project.

Dave Render and Prudence Mambo, without your unbelievable help, guidance and perseverance, none of this would have been possible. No words can describe how much I appreciated your support.

Prof. Ashton Keith Cowan, I would like to thank you for giving me the opportunity and belief to tackle a project of this calibre. Your knowledge, support and supervision towards me and this project was no doubt of the highest quality and therefore can't thank you enough.

Finally to my parents, thank you for believing in me and giving me the comfort when times were tough. You guys are an inspiration to me and hopefully I have become an inspiration to you. You are truly the best.

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List of Abbreviations

ABP	**Algae- Based Ponds**
ADB	**Algae Drying Beds**
AFP	**Advanced Facultative Ponds**
AIWPS	**Advanced Integrated Wastewater Pond System**
ARP	**Algae Rock-filter Ponds**
ASP	**Algae Settling Pond**
BOD	**Biochemical Oxygen Demand**
BOD_{ult}	**Ultimate Biochemical Oxygen Demand**
BOD_5	**Five Day Biochemical Oxygen Demand**
C/F	**Coagulation/ Flocculation**
COD	**Chemical Oxygen Demand**
COD_t	**Total Chemical Oxygen Demand**
cfu	**Colony Forming Unit**
DBP	**Disinfection By-Products**
DO	**Dissolved Oxygen**
DWA	**Department of Water Affairs**
DWAF	**Department of Water Affairs and Forestry**
DWP	**Duckweed Ponds**
EBRU	**Environmental Biotechnology Research Unit (Rhodes University)**
EC	**Electrical Conductivity**
FC	**Faecal Coliforms**
HLR	**Hydraulic Loading Rate**
HRAOP	**High Rate Algal Oxygenated Pond**
HRT	**Hydraulic Retention Time**
IAPS	**Integrated Algal Pond System**
IPD	**In- Pond Digester**
MGD	**Mega Gallons Per Day**
ML	**Mega Litre**

MLSS	**Mixed Liquor Suspended Solids**
MP	**Maturation Pond**
MPN	**Mean Probable Number**
mS.m^{-1}	**milliSiemens per meter**
P.E.	**Persons Equivalent**
PVC	**Polyvinyl Chloride**
QTS	**Quaternary Treatment System**
RF	**Rock Filters**
RSF	**Rapid Sand Filter**
SB	**Splitter Box**
SSF	**Slow Sand Filter**
TC	**Total Coliforms**
TDS	**Total Dissolved Solids**
TSS	**Total Suspended Solids**
TTU	**Tertiary Treatment Units**
UV	**Ultraviolet**
WHO	**World Health Organization**
WL	**Wetlands**
WRC	**Water Research Commission**
WSP	**Waste Stabilization Ponds**
WWTS	**Waste Water Treatment Syatems**
WWTW	**Waste Water Treatment Works**

CHAPTER 1

Introduction and Literature review

1.1 Introduction

A continuous supply of fresh water is one of the most essential commodities, for both the environment and for human necessity. Vast improvement is needed in order to decrease pollution in water supplies and the degradation of finite resources (Rose *et al.,* 2002b). There is a need to prioritize cost-effective ways to improve the social welfare, in terms of enhancing water quality and sanitation in South Africa.

South Africa is a semi-arid country and population growth is increasing at an exponential rate (Erasmus *et al.,* 2005; Vetter, 2009). According to Mwenge Kahinda *et al.* (2007), 3.7 million people in South Africa do not have infrastructure for their daily water supply and a further 5.4 million people are left with only a 'basic level of service'. Due to the scarcity of potable water in South Africa, water prices in the future will rise. Government will have to increase tariffs for local water supply in order to uplift the national water resource infrastructure, water services and conservation (van Rooyen *et al.,* 2011).

In South Africa, the distribution of wealth is seen to be unequal when compared to the rest of the world and many households especially in the poverty stricken areas are without adequate health care, education, energy and clean water (May and Govender, 1998). According to the PROVIDE Project (2005), it is estimated that 65% of the people in the Eastern Cape live in rural areas. Of the 55 wastewater treatment plants in the rural parts of the Eastern Cape Province, only 18% were operating to the correct microbiological recommendations (Momba *et al.,* 2006). It is estimated that only 34% of rural households in the Eastern Cape Province have access to sewage treatment facilities. Insufficient sanitation is one of the major problems with regard to water pollution causing water-borne illnesses to humans e.g. cholera, which has become endemic to the country (Wells, 2005; Lee and Kamradt-Scott, 2010). There is an increase in the discharge of wastewater into the rivers on a national level in South Africa which has become problematic due to poor administration and management by the municipalities (Eales, 2011). This is a major obstacle to socio-economic development in South Africa because poor quality water can cause devastating effects on the health of people, especially in poverty stricken areas (Rose *et al.,* 2002b).

Clean water is a limiting resource and as the human population continues to grow, this problem will become more pressing and serious around the world (Gleick, 1998). Although conventional technologies (activated sludge and biological nutrient removal systems) are important for wastewater treatment, they are expensive processes and require high energy input. Management of these sewage plants in rural areas is also challenging due to the lack of local technical expertise (Letinga, 1995; Horan *et al.,* 2006). Therefore, scientists have been strategizing in an effort to derive new, cheap and innovative ways of sustaining water in all forms. The supply of clean water is considered a major driver for sustainable development and the main challenge for this development is the recycling of wastewater nutrients, irrigation and urban agriculture (Horan *et al.,* 2006). Therefore, the development of alternative more appropriate wastewater treatment is seen to be very beneficial. The Integrated Algal Pond System (IAPS) seems to be one possible solution to the problem.

The IAPS technology adheres to these prioritizations with regard to South Africa's infrastructure, services and conservation in the water sector. The IAPS has the ability to treat and recycle domestic wastewater. With the IAPS being a low-cost domestic wastewater treatment technology, the system has the potential of being implemented by small municipalities as a rural treatment works (Rose *et al.,* 2002a). Operating and control skills are limited; require low maintenance and upkeep, and also energy efficient, therefore requiring less electricity for operational use compared to other wastewater treatment systems (Wells, 2005).

The first IAPS was designed and developed by Prof. William Oswald from the University of California, Berkley, USA. The IAPS system was specifically designed as an alternative wastewater treatment plant to the conventional wastewater waste stabilization pond systems. This system is ideally an enhancement of the conventional waste stabilization pond (WSP) systems because it was able to remove the same amount of nutrient and organic material compared to the conventional WSP at a lower cost and lower energy efficiency (Wells, 2005).

1.2 History of IAPS

The international oil market during the 1970's came to disruptive halt due to the concerning fact of fossil fuel depletion which is used as an energy resource. A global interest then came into energy conservation as well as renewable energy resource development. Scientists came up with ways of trying to avoid disastrous environmental impacts as well as finding ways to reduce costs of energy resources. These factors also played an influential role in the development of environmentally friendly wastewater treatment plants (Green *et al.,* 1995).

Oswald was one of the pioneers in the development of the IAPS (Mambo *et al.,* 2014). It was in the 1950's when he became interested in the design of natural, affordable and sustainable wastewater treatment systems. Most of his research was done at the Lawrence Berkeley National Laboratory in California. His early research began with the role of microalgae in sewage ponds in 1949. A high rate algal oxygenated pond (HRAOP) system was then developed by Oswald in 1957 which was used for wastewater treatment and nutrient recovery as algal biomass (Craggs, 2005).

From there Oswald expanded these systems to other research sites such as Concord and Richmond in California, with the Richmond HRAOP the largest outdoor algae cultivation pond on the globe (Oswald *et al.,* 1994; Craggs, 2005).

Methane production in algal pond systems was one of the main focuses during the early 1960's. Craggs (2005) revealed that better treatment was provided by deeper anaerobic pits within the ponds. In 1967, the first full scale Advanced Integrated Wastewater Pond System (AIWPS®) (initial name), with a deep fermentation pit integrated into the facultative pond was built in St Helena, California (Oswald, 1990).

During the 1970's, HRAOP systems continued to play a big role with regard to algal growth and productivity. Paddlewheels seemed to be the most effective and efficient mixer and replaced the propeller pumps in the Richmond HRAOP in 1978 (Green *et al.,* 1995; Craggs, 2005). Hollister, California became the second area where a full scale IAPS was built in 1980 (Oswald, 1990).

As global research continued for more efficient wastewater treatment systems, it was during the late 1970's and early 1980's when a researcher by the name of Gedaliah Shelef and his co-workers expanded the HRAOP technology worldwide and decided to use these systems as a treatment for domestic wastewater in Israel (Craggs, 2005). Similar work was initiated in Kuwait by Ismail Esen, Kazmer Puskas, Ibrahim Banat, Reyad Al-Daher and Yousif Al-Shayji. These researchers had set up pilot plant HRAOP systems to treat municipal wastewater for irrigational use and often with the use of a post- treatment sand filtration unit for algae removal (Puskas, Esen, 1989; Puskas *et al.,* 1991; Esen *et al.*, 1987; Esen *et al.*, 1991; Al-Shayji *et al.,* 1994).

Nutrient removal, energy efficiency and resource recovery continued throughout the 1980's and 1990's using these IAPS systems. A large amount of research was done on algae harvesting, methane recovery and utilization from HRAOP systems (Craggs, 2005). The harvesting of algal/bacterial biomass (or mixed liquor suspended solids: MLSS) from the HRAOP system can be beneficial for the use in biofuel (methane), fertiliser and feed production (Craggs *et al.,* 2012).

Currently extensive research has been done on using sand filtration, dissolved air flotation and reverse osmosis on enhancing IAPS effluents. The reason being is that the IAPS does not produce a final effluent that meets the general standards set by various authorities for environmental discharge. Also the removal of selenium from agricultural drainage waters using the IAPS system has been a fascination amongst some researchers (Craggs, 2005). Selenium is a toxic pollutant that contaminates drainage waters. It causes a devastating effect on aquatic life e.g. birds, which causes embryonic deformity and mortality (Kharaka *et al.,* 1996).

1.3 IAPS as a technology for municipal sewage treatment

The IAPS can be regarded as an innovative wastewater treatment system; however, without the correct design and configuration, it may give misleading results. Multiple pond systems like the IAPS perform well in terms of organic solids removal but the removal of algal solids and nutrients as well as the disinfection of wastewater are very inconsistent (Craggs, 2012). The configuration and design of the system does not allow for the final effluent to meet the recommended standards for environmental discharge in South Africa (Wells, 2005). Therefore one way to avoid effluent quality issues is to introduce polishing components and to include tertiary treatment units (TTU) to ensure that the final effluent meets the discharge standards.

1.3.1 General Information on IAPS

The integrated algal pond system (IAPS) as a wastewater technology is a derivation of the AIWPS® developed by Oswald who is credited as the pioneer of algae pond technology which he began studying in 1949 (Ludwig and Oswald, 1952). Initially, Oswald focussed on the symbiosis of algae and bacteria in wastewater treatment (Oswald *et al.*, 1955). This later became known as photosynthetic oxygenation (Oswald *et al.*, 1957). Photosynthetic oxygenation is the aeration effect caused by algae by release of photosynthetically generated oxygen on treated wastewater (Ludwig *et al.*, 1951; Ludwig and Oswald, 1952; Oswald *et al.*, 1953, 1955). By 1957, Oswald had established the High Rate Algae Oxidation Pond (HRAOP). This algae-containing raceway incorporated wastewater remediation via biological oxygenation and nutrient removal (Oswald *et al.*, 1955, 1957) and eventually led to the fully developed IAPS (Oswald *et al.*, 1957) (Fig 1.1).

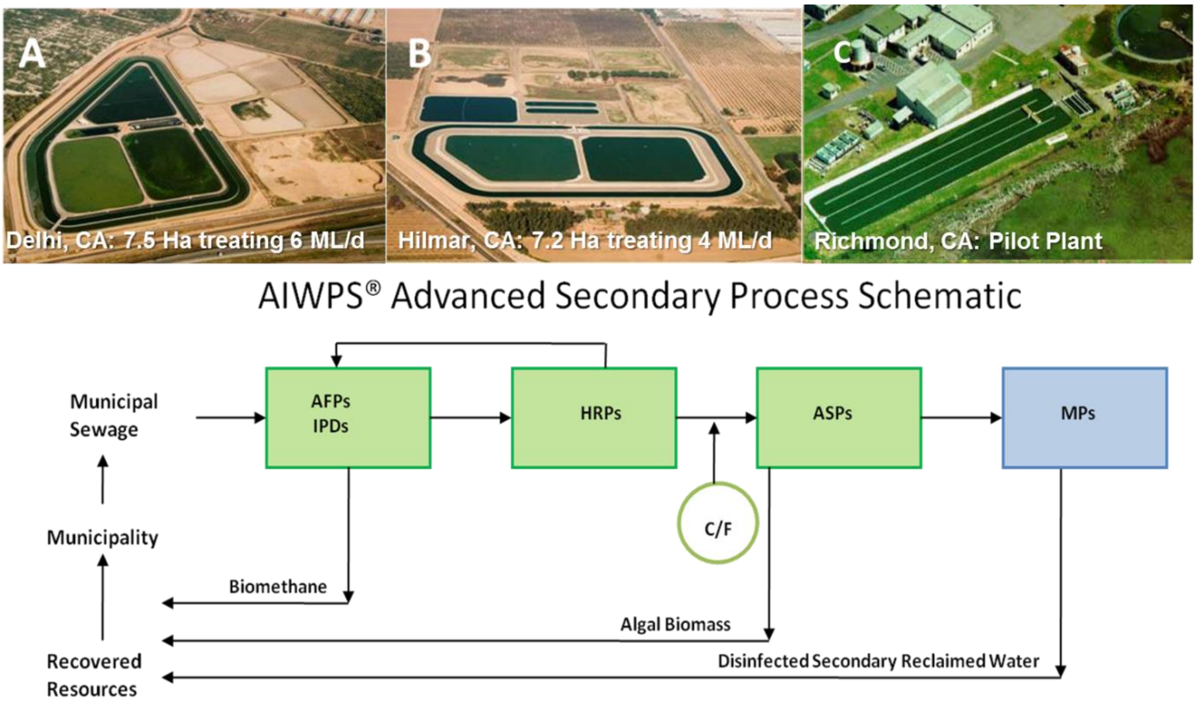

Figure 1.1: IAPS operating configuration design deployed in California, USA. A) Delhi, California plant with design flow= 6 ML/d, wet surface area= 7.5 hectares, total area= 16 hectares, annual energy cost=$20,000, wastewater rate=$21/month/household; B) Hilmar, California plant with design flow= 4 ML/d; wet surface area= 7.2 hectares; total treatment area= 15 hectares; annual energy cost= $13,000; wastewater rate=$21.85/month/household C) St Helena, California plant established in 1965 (ML/d = mega litres per day).

1.3.2 IAPS Process Flow

There are 5 phases with regard to wastewater treatment. Primary treatment (AFP) includes the removal of suspended solids. Secondary treatment (HRAOP; ASP; ADB) involves the reduction of dissolved biodegradable organic matter and lowers the biochemical oxygen demand (BOD) to a level where the oxygen is not depleted completely within the effluent flow. Tertiary treatments (MP, SSF, and RF) are required for the removal of nitrogen and phosphorus, so that the growth of algae and other aquatic plants is reduced. Quaternary treatment systems (QTS) (chlorination, ozone, ultraviolet light) are important for the removal of obstinate organic compounds while quinary treatment involves the removal of dissolved organics, salts and heavy metals (Cowan and Render, 2012).

The advanced facultative pond (AFP) has two separate layers within the pond: anaerobic bottom layer and a surface aerobic layer. At the base of the AFP, there is an in-pond digester (IPD), where solid sewage sedimentation and anaerobic procedures are carried out (Rose *et al.,* 2002b). With these anaerobic procedures, organic solids are converted microbially into organic nutrients and methane (Bolan *et al.,* 2009). The IPD is surrounded by a wall so that there is no mixing with the oxygenated water (Rose *et al.,* 2002b). The volume of the AFP is 1500 m^3 with a surface area of 840 m^2 and has a HRT of 20 d. If designed properly, the AFP has the capability of reducing between 60% and 80% of BOD by methane fermentation, and almost all suspended solids (Green *et al.*, 1996; Wells, 2005).

The IPD is deep (4.5 m) and therefore most of the sewage solids cannot be expelled. The volume of the pit is 225 cm^3 (giving the pit a 3 d HRT) and is designed in such a way, that it increases the amount of settleable solid deposits within the IPD (Oswald *et al.,* 1994; Wells, 2005). The upflow velocity is usually less than 1.5 $m.d^{-1}$. This is slow in order to prevent parasites and pathogens from escaping the pit (Rose *et al.*, 2002b). The velocity is also so low so that there is almost 100% removal of suspended solids as well as 70% of BOD removal (Oswald, 1990). Bubbles of biogas lift the solid waste to the surface of the pit and as the biogas bubbles expand, they break away from solid matter, leaving the solid waste to sink and resettle again. This produces anaerobic sludge through solid accumulation where the initial wastewater flows through (Rose *et al.*, 2002b).

The surface aerobic layer of the AFP has a vast quantity of algal growth which is supported by carbon dioxide formed as a component of the biogas. Algal growth and the associated increase in photosynthetic oxygen production provide the aerobic function. This surface aerobic layer causes the oxidation and entrapment of odour compounds found within the raw waste due to the anaerobic gases being oxidised by the aerobic layer. Thus, these systems can be located very close to urban areas (Rose *et al.*, 2002b; Wells, 2005).

Preliminary data derived from the EBRU IAPS has indicated the potential of this technology to produce a biogas stream comprising more than 80% methane (Cowan and Render, 2012). Since a value of 70% methane is traditionally regarded as good, all indications are that an above average biogas stream can be routinely obtained from this system. The only reliable data on methane production by the fermentation pit of an IAPS was recorded from a plant in Richmond, California, where 0.22 m^3 CH_4. kg^{-1} of biogas was produced. However, only a fraction of the wastewater influent was passing through the digester, producing a low methane yield (Green *et al.*, 1995).

The high rate algal oxidation pond (HRAOP) is a paddle-mixed raceway and it is more efficient and cost effective than a conventional secondary facultative pond due to the HRAOP producing more DO (Craggs *et al.*, 2012). The total volume of the HRAOP's is 150 m^3 and the water depth of each HRAOP is 30 cm, therefore the shallow water allows the entire water body to become oxygenated (Wells, 2005). The retention time is very short, normally three to five days, but currently the configuration of the Belmont Valley IAPS is 2 and 4 d in HRAOP (A) and (B) respectively.

The pH in both HRAOP systems tends to increase to above 9.5 due to algal photosynthesis, therefore killing all *E. coli* (Wells, 2005). COD levels are increased due to the large abundance of algae content found in the HRAOP's. Algae are known to excrete small photosynthetic organic molecules, which increases the COD concentration levels (Wang *et al.*, 2010). The HRAOP system also increases the DO levels caused by photosynthesis of the algae which converts sunlight, carbon dioxide and water into oxygen (Wells, 2005).

Electrical energy is used to drive the paddle wheels. Each paddlewheel requires a 250 to 370 watt electrical motor to provide a linear velocity of 30 $cm.s^{-1}$ (Wells, 2005). This system provides a gentle flow which continuously mixes the algae and allows for formation of algal flocs within the channels which remain in suspension close to the water surface and within range of light penetration. Larger bacterial flocs move more slowly along the bottom of the channels, where they utilize photosynthetic oxygen to oxidise BOD influent. Thermal stratification is also prevented by the paddle wheels, which allows the pH and DO to be consistent throughout the HRAOP system (Green *et al.*, 1995).

From HRAOP (A), the water gravitates into the algal settling pond (A) (ASP (A)) as partially treated wastewater. From ASP (A), water then gravitates to a splitter box (SB) (as partially treated wastewater), where it is then transferred to HRAOP (B) (Fig 1.2). Coagulation/

Flocculation (C/F) of the biomass occurs in HRAOP (B) and is collected in ASP (B) where the suspended solids are separated. ASP (B) then gravitates the water via a SB where it is released as the final effluent.

Figure 1.2: SB from the IAPS final effluent. From here the effluent flows into the different tertiary treatment systems: Maturation Pond; Slow Sand Filter; Rock Filter.

The algal settling pond (ASP) has a designed HRT of 0.5 d, which allows 50-80% of algae to settle to the bottom of these ponds. The algae can then be removed to the algal drying beds (ADB's) when required (Cowan and Render, 2012). If the effluent of this system is used for agricultural purposes (e.g. irrigation) then algae does not have to be removed but the mean probable number (MPN) for bacteria must be less than one thousand. Little evidence has been obtained to suggest that the biomass is harmful but precautionary steps still need to be provided. Wastewater algae can also be harvested and utilized as a resource for other purposes. For example, e.g. plant natural product production, pigment extraction; animal feeds etc. (Rose *et al.,* 2002b).

Algae dewatering occurs on the algal drying beds (ADB's) which are sand beds, a biomass/sand mix is produced that can be utilized as a soil conditioner, a biofertilizer, animal feedstock or as a substrate in biological methane generation (Kothandaraman and Evans 1972).

Tertiary treatment units (TTU) are constructed to further improve the microbial quality of secondary treated water, especially with regard to wastewater that is to be re-used and then discharged into the environment (Gerba and Pepper, 2011). There are many different types of systems that can be used for tertiary treatment, which include a MP series, Slow Sand Filter (SSF), Duckweed Ponds (DWP), Rock Filter (RF), Constructed Wetlands (WL) etc. Tertiary treatment units are used to remove nutrients, residual organics and pathogens. These systems are normally followed by Quaternary Treatment Systems (QTS) which have the capability to remove salts, metals and pesticides (Wells, 2005). Examples of QTS include downstream chlorination, ozone, and ultraviolet (UV) treatment.

Chlorination is used to prevent waterborne diseases from spreading and is considered a very important process (Spellman, 2008). During the 1960's and 1970's, there had been many studies with regard to chlorine disinfection and these studies revealed that this type of QTS is feasible and has the added benefit of reducing suspended solids and improving the water turbidity (Davies-Colley, 2005). Chlorine also has the ability to abolish 'bad' odours e.g. mercaptans, hydrogen sulphide etc. (Wells, 2005). There are however a few concerns with this treatment process; chlorine has the capability to create unwanted disinfection by-products (DBP's) when it reacts with organic and inorganic compounds within the water. It is also known to cause odour and taste problems in the water if high doses are added (Spellman, 2008). Chlorination was a common practice for disinfecting water all around the world. However, studies eventually showed that chlorine (with the reaction of organic and inorganic compounds) formed toxic disinfection by-products (DBP's). For example, trihalomethanes, haloacetic acids, chlorite etc. (Bayo *et al.,* 2009).

Ozone has been used as a disinfectant and was first developed as a water purifier in Oudshoorn, Holland in 1893 (Vigneswaran and Visvanathan, 1995). Davies-Colley (2005) found that ozonation is the preferred choice over chlorination when it comes to disinfecting water from mechanical secondary sewage treatment works due to its 'potent virucidal action'. Many companies (e.g. Ozonia) have used ozonation for the treatment of municipal and industrial wastewater because this treatment is known to improve the DO content and to oxidise sulphides. However, it is not an appropriate post-treatment technology for the IAPS due to being energy intensive, operationally expensive, toxic and corrosive (Wells, 2005).

Ultraviolet light is known as an efficient disinfecting technology and no DBP's are expected to be produced (Vilhunen *et al.,* 2009). UV treatment is becoming a popular disinfectant technology for wastewater treatment because it is known as the safer option as it does not contain any toxicity which may affect the water effluent (Davies-Colley, 2005; Wells, 2005).

Oparaku *et al.* (2011) have mentioned that UV disinfection is often selected for wastewater treatment due to having low energy costs, no harmful by-products, no chemical consumption etc., which is beneficial towards safety and environmental problems.

The IAPS has been researched and deployed around the world in various countries. It has been used to treat different wastewaters e.g. abattoir, piggery, tannery, aquaculture, sewage, cattle, winery etc. Table 1.1 summarises the global distribution of the IAPS for the treatment of different wastewaters.

Table 1.1: The global distribution of IAPS systems used for the treatment of wastewater (Craggs, 2005).

Country	Climate	Origin of waste	Treatment	Performance	Effluent Use	Year	Reference
Australia	Semi-arid- Desert	Abattoir	HRAOPs		Discharge to environment	2002	Evans *et al.*, 2003
Brazil	Tropical-Temperate	Domestic	HRAOPs		Discharge to environment	1983	Kawai *et al.*, 1984
France	Mediterranean	Domestic	HRAOPs	41- 45% COD 67-72% N-NH_4 59-60% P-PO_4^{3-}		1997	Bahlaoui *et al.*, 1997
New Zealand	Temperate	Domestic	Two HRAOPs	100 % faecal coliform disinfection	Nutrient rich biomass as fertilizer	2007	Craggs *et al.*, 2003
Germany	Temperate	Domestic	HRAOPs		Discharge to environment	1986	Grobbelaar *et al.*, 1988
Singapore	Tropical	Piggery	HRAOPs			1992	Taiganides, 1992
United States of America	Mediterranean	Domestic	AFP, HRAOP, ASP and then a sand filter or a DAF point	99 % BOD 99 % TSS 78 % nitrogen 92 % phosphate 99.999% coliform removal	Discharge to environment	1959	Oswald *et al.*, 1960
Kuwait	Desert	Municipal and Industrial	Oil and sand traps, AFP, two HRPs and four ASPs.	95 % BOD 85 % COD 99 % coliform removal pH 9.5 to 10	Discharge to environment	1990	Al-Shayji *et al.*, 1994
India	Tropical	Domestic	AFPs, HRAOP, ASP, MP	98 % BOD, 92 % SS, 91 % nitrogen 96 % E. Coli removal	Discharge to environment	1986	Mahadevaswamy & Venkatamaran 1986
Ethiopia	Tropical	Tannery	AFP, SFP and MP	95 % BOD 93 % COD 57 % ammonia 76 % phosphate 89 % sulphates 95 % chromium removal	Discharge to environment	1990	Tadesse *et al.*, 2003

There are over twenty countries that have deployed an IAPS for the treatment of different wastewaters. The studies made on these IAPS systems have shown a decrease in certain parameters (e.g. COD, BOD, faecal coliforms, phosphate, nitrate etc.), in terms of percentage removal. These systems showed a significant percentage of nutrient and pathogenic removal and proved to be effective in most cases. Many communities and farms around the world have used multiple pond systems for the treatment of wastewater due to the system's ability to remove organic solids (Craggs *et al.,* 2012). However, very few countries have implemented this technology for commercial use. Multiple pond systems are generally inconsistent with regard to algal solids removal, nutrient removal and disinfection. Another disadvantage for advanced pond systems is the large land requirements needed compared to other electromechanical treatment systems (e.g. activated sludge). The USA has built a full scale HRAP's as a component of advanced pond systems for the last 50 years. In New Zealand, the National Institute of Water and Atmosphere Research Ltd has conducted research on pilot-scale and full-scale HRAP systems for the last 13 years and have found that the HRAP system is more consistent and improved than oxidation ponds in terms of wastewater treatment (Craggs *et al.,* 2012). Craggs *et al.* (2012) had also recommended additional polishing treatments in order to meet discharge requirements. For example, maturation ponds, rock filters, UV disinfection and membrane filters.

1.3.3 The IAPS at Belmont Valley, Grahamstown

The IAPS was installed in February 1996 at the Belmont Valley Wastewater Treatment facility (33º 19' 07" South, 26º 33' 25" East) in Grahamstown (Fig 1.3). This system was constructed as a secondary treatment facility for Grahamstown's municipal wastewater. The purpose behind the project was to re-design the technology for South African operating conditions as well as demonstrate the technology and provide an engineering support base for the development of the IAPS process. It was envisaged that the design would be implemented in South Africa for treating wastewater (Rose *et al.,* 1996) but this unfortunately has not been the case. This pilot scale IAPS continues to operate and treats 80-100 m^3 domestic wastewater daily. Data from a series of investigations to determine efficiencies has revealed that this system has the potential to comply with the South African DWA discharge standards (Table 1.2) (Rose *et al.,* 2002a, 2002b, 2007).

Figure 1.3: The pilot IAPS at the Institute for Environmental Biotechnology, Rhodes University (EBRU).

The difference between IAPS around the world and the pilot IAPS constructed and located at the Belmont Valley WWTW in Grahamstown was that it was designed without a final tertiary treatment component e.g. maturation pond. The pilot system is then most closely allied to the IAPS Secondary Process (Fig 1.4). Therefore the final effluent generated by the Belmont Valley IAPS can only be described as a 'secondary treated' water, which may explain why the system is inconsistent in terms of water quality following treatment of wastewater (Rose *et al.*, 2007).

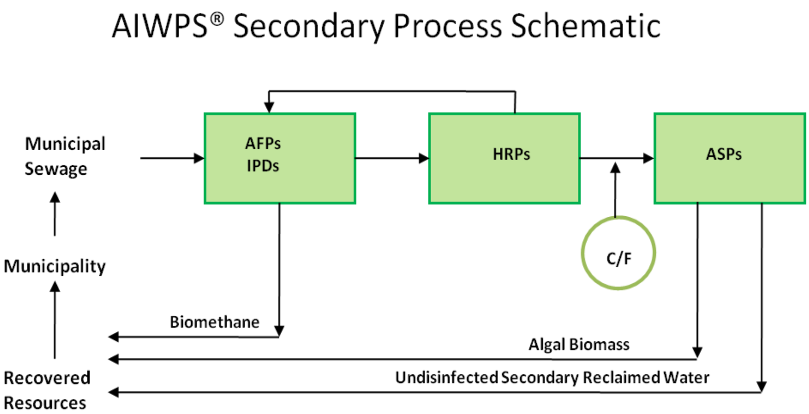

Figure 1.4: Schematic diagram illustrating the process flow for various IAPS designs based on technology developed by Oswald to recover nutrients, energy and water from influent wastewater. AFP=Advanced Facultative Pond; IPD=In-Pond Digester; HRP=High Rate Pond; C/F=Coagulation/Flocculation; ASP=Algae Settling Pond. URL: http://www.go2watersolutions.com/process-schematic.html

Table 1.2: DWA's recommended standards for environmental discharge.

PARAMETERS	STANDARD
Ammonia (mg. L^{-1})	3
Phosphate (mg. L^{-1})	10
Nitrate (mg. L^{-1})	15
COD (mg. L^{-1})	75
pH	5.5 – 9.5
Faecal Coliforms (cfu/100 mL)	< 1000
Total Suspended Solids (mg. L^{-1})	25
Dissolved Oxygen (mg. L^{-1})	>2
Electrical Conductivity (mS.m^{-1})	70 - 150

The original IAPS designed by Oswald always included a polishing step comprising of either a MP or other which would allow the final effluent to meet the discharge specifications as required by DWA. Craggs *et al.* (2012) had designed and implemented a hectare-scale HRAOP for enhanced wastewater treatment and recommended that additional treatment is be required as an extra polishing step to meet specific discharge standards. These authors recommend the inclusion of one or a combination of different post-treatment systems e.g. combination of MP and UV treatments prior to discharge, MP and rock filtration in series or direct UV treatment if insufficient land is available. If funds are available, then the use of membrane filtration is the choice to achieve a high quality final effluent for re-use.
Without a final polishing step, and as demonstrated in other studies, the COD of the final effluent remains elevated resulting in the potential that if discharged water from an IAPS will be detrimental to any receiving water bodies (Park and Craggs, 2011b). Clearly, any considered implementation of IAPS technology for treatment of domestic wastewater must include in the process design a final effluent polishing process.

1.4 Post-treatment Systems for Integration into the IAPS Wastewater Treatment Technologies

1.4.1 Maturation Ponds

MP’s are known for their polishing effect after a conventional system and tend to be constructed in series following a facultative pond (Fig 1.5) (Shilton and Walmsley, 2005). MP’s are known to decrease pathogens e.g. viruses and faecal bacteria which are found in the effluent of facultative ponds. Removal of pathogens is normally based on the size, design and number, geographical location and climate (Liu, 2008). MP’s are an aerobic system and therefore provide less biological stratification than the facultative ponds and also allow the ponds to be fully oxygenated throughout the day.

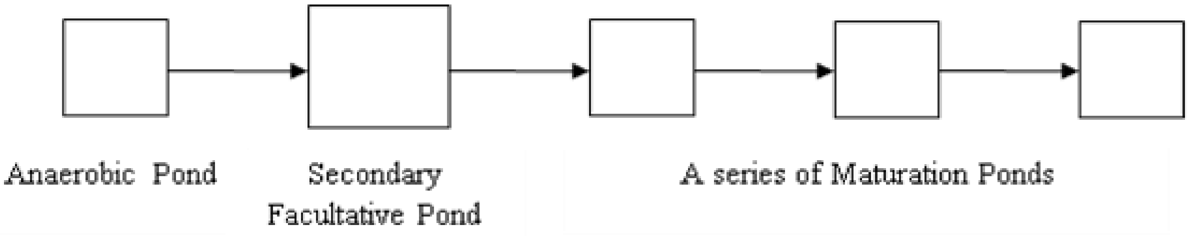

Figure 1.5: A typical standard pond system and associated maturation pond series (Shilton and Walmsley, 2005).

Maturation ponds normally have a significant removal rate of phosphorus and nitrogen (Liu, 2008). Picot *et al.* (2009) confirmed that MP's removed the majority of nitrogen and removal rates were higher in the summer months than in the winter. Phosphorus is normally reduced by algae in MP's but this can also have a negative effect on the MP's suspended solid and turbidity count in the final effluent (Liu, 2008). Removal of phosphorus in WSP's (which normally include an MP) usually occurs through precipitation as well as a high pH (>9.5) and aerobic conditions caused by algae photosynthetic activity in the MP's. This inevitably causes the formation of insoluble hydroxyapatite at high pH levels (Pearson *et al.,* 2005; Mara, 2013).

Algae diversity increases across the MP series but the algal biomass decreases in the final effluent due to the baffle systems, which forces the algae to sink and settle at the bottom of each pond (Mara, 2005; Mara 2013). Maturation Ponds need to disallow prolific algae growth/production to maximise light penetration and promote zooplankton grazing for removal of unsettled algae within the pond. Other natural disinfectants of these ponds are protozoan grazing, solar- and UV radiation and sedimentation (Craggs, 2005; Liu, 2008).

According to the WHO (World Health Organization, 2012), the faecal coliform (FC) concentration of water must to be <1000 per 100 ml to prevent water related illnesses when consumed by humans. The main principles for faecal bacterial removal are:

- Long time and high temperature
- High pH (higher than 9)
- High dissolved O_2 concentration with high light intensity (Kayombo *et al.*, 2005)

Liu (2008) mentioned that removal of pathogens in MP's is caused mainly by predation, adsorption, natural die-off and sedimentation. Normally these systems allow most of the pathogens to settle to the bottom of the pond, where sludge is accumulated, (if the system has a high retention time). It has been suggested that bacterial reduction mainly depends on climatological and environmental constraints (Polprasert *et al.,* 1983). Therefore, long time periods and high temperature do increase the mortality of faecal bacteria. MP's are an open system and therefore unprotected from direct UV exposure, which contributes to coliform reduction (Wells, 2005). The bulk of ammonia is found in the algal biomass and with an increase in pH, ammonia tends to be extracted from the pond via volatilization (Kayombo *et al.,* 2005). Therefore the MP, if designed correctly, can be recommended as a possible polishing system to meet specific discharge requirements.

1.4.2 Sand Filtration

Filtration is a process where porous material is used to purify water (di Bernardo, 2002). The effluent quality from pond systems is usually insufficient to meet the environmental objectives and local discharge standards and therefore needs extra treatment to reach these objectives (Middlebrooks *et al.,* 2005). Sand filtration systems for wastewater treatment are regarded as fixed media bioreactors. Active biofilms on sand particles underpin performance of this technology. These biofilms are relatively resistant to changes in concentration of metals or fluctuations in pH within the wastewater. In addition, systems like these need very little maintenance and the operating costs are reported to be very low compared to other conventional wastewater systems. Sand filtration is thus a biofilm-driven process and is used for the integration of nutrients into diverse microbial populations to achieve the mineralization and biodegradation of organic matter (Gaur *et al.,* 2010). There are many types of sand filtration process but the main types are rapid sand filtration (RSF) and slow sand filter (SSF).

1.4.2.1 Rapid Sand Filtration

A rapid sand filter (RSF) has the same mechanism of filtration as a slow sand filter (SSF), except that the biological processes are decreased. This is due to a shorter filter run time between cleaning periods, which prevents biological growth (Scholz, 2006). Sand particles are usually larger and due to the higher filtration rate, the filter run only lasts from a couple of hours to a few days (Table 1.4). The filtering medium of the RSF is the whole bed depth as opposed to the SSF, which uses only the top few inches of the bed (Pizzi, 2011). This type of filtration system often achieves poor water quality results compared to the slow sand filter. Improvement can occur if coagulation, flocculation or the use of chemicals prior to filtration takes place to break up the suspended solids (Middlebrooks *et al.,* 2005).

1.4.2.2 Slow Sand Filtration

Slow sand filters (SSF) were one of the first modern treatment techniques used for the purification of drinking water. It is known that SSF produces a high class filtrate and this technology is employed extensively throughout the potable water industry. These filters are also able to decrease up 99.9% of bacteria within the water (Ellis, 1987). Thus, SSF is seen as a very promising post treatment option, in terms of the cost efficacy, effluent quality and operational simplicity and one of the best solutions for wastewater problems (Table 1.3) (Gunes and Tuncsiper, 2009).

According to various studies, SSF are able to remove 86% BOD, 68% suspended solids, 88% turbidity and 99% total coliforms (TC). It is therefore not surprising that SSF have been used to treat high quality surface waters and in the treatment of secondary effluents. Sand filters are designed based on the hydraulic load as well as organic load and this system is simple enough that less skilled manpower can be used in the day-to-day operation (Tyagi *et al.,* 2009). Reducing pathogens from wastewater, at a low cost and low maintenance, makes this type of filtering system very appealing, especially in developing countries (Bauer *et al.,* 2011). The SSF's filtration rate has been estimated to be 50-150 times lower than that of the RF (Table 1.4). The flow retention periods for the SSF are 30-90 times longer than that of the RF (Galvis *et al.,* 2002).

- Sand in the filtering system has mechanical techniques in straining out solid substances within the raw wastewater and this is done effectively through the upper layer of the filter.
- Nitrifying microorganisms produce a chemical reaction in the sand which oxidise organic matter.

Continuous filtering allows for a slimy gelatinous layer (mainly algae, bacteria and plankton) to form on the surface of the filter. This gelatinous layer tends to oxidise ammoniacal nitrogen to nitrates, remove organic matter and yield bacteria-free water. This layer (which extends 2- 3 cm into the top part of the bed) acts as a retaining mechanism for all the bacteria in the water (Roday, 1998).

Table 1.3: The advantages and disadvantages of the slow sand filter (Roday, 1998; Scholz, 2006).

Advantages	Disadvantages
No pre-treatment is needed, except for preliminary sedimentation	Large area of land needs to be used
Easy to construct and operate	Colour removal seems to be poor
Bacteriological, physical and chemical quality of water is high	Removal of turbidity is poor
Total bacteria count and *E.coli* is reduced to 99.9%	
No chemicals	
Less corrosive effluent compared to RF system	
Cheap	
Cleaning of filters is not done on a regular basis	

Table 1.4: Comparative design between the different sand filtration systems (Purcell, 2003).

Design of filtering system		
Filter type	**Slow**	**Rapid**
Water depth (m)	1.0 - 1.5	1.0 - 1.5
Bed thickness (m)	0.6 - 1.2	0.6 - 1.2
Underdrainage depth (m)	0.5	0.5
Effective sand size (mm)	0.15 - 0.4	0.5 - 1.5
Uniformity coefficient	2.0 - 3.0	< 1.5
Filtration rate (m.h^{-1})	0.1 - 0.25	5.0 - 7.0
Filter run (d)	20.0 - 60.0	1.2
Cleaning method	Surface skim	Backwash

1.4.3 Rock Filtration

Rock Filters (RF) are a simple operation process. Effluent from previous pond systems will enter and travel throughout the submerged porous rock bed. The algae that enter are stored in the void spaces when they settle on the rock surface (Middlebrooks, 1988; Crites *et al.,* 2005). One of the main advantages of a RF is that it is a simple operation at fairly low cost and local material can be used for construction (Crites *et al.,* 2005; Davies-Colley, 2005). When liquid media flows through the RF, algal biofilm coatings are formed around the rock surface, which enable the entrapment of microbial contaminants (bacteria, fungi, protozoa etc.) (Davies-Colley, 2005).

All over the world, many communities have used rock filters systems for the treatment of wastewater. Saidam *et al.* (1995) operated RF's as a post-treatment to a WSP in Jordan and demonstrated a reduction in TSS and BOD_5 by 60% from the pond final effluent. Mara and Johnson (2006) also used RF's as a post-polishing system for WSP's over an 18 month period. The concentration of ammonia and nitrate after filtration was <3 and 5 $mg.L^{-1}$ respectively and the FC count was 65 cfu per 100 ml.

Rock Filters have the ability to become anoxic if the system is not aerated; decreasing the nitrification process which therefore discourages the removal of ammonia. Aeration of the system also improves TSS removal (Hamdan and Mara, 2009). Most odours contain hydrogen sulphide and are caused by the anaerobic environment of the system. Studies have shown that hydrogen sulphide levels above 1 $g.S.m^{-3}$ in pond water surfaces can cause a significant reduction in algal growth (Middlebrooks *et al.*, 2005).

1.5 Aim

The aim of the current research project was therefore to design and construct post-secondary treatment technologies for incorporation into the IAPS system design and find out whether these systems can provide the polishing required allowing for the treated water to meet the standards set by DWA for discharge to the environment. The parameters that were considered in the design of the tertiary treatment units included maturation ponds, slow sand and rock filters. Efficiency of operation of the tertiary treatment units was determined by measuring COD, nitrate-N, ammonium-N, phosphate-P, TSS, pH, FC, TC, electrical conductivity (EC) and DO. The research hypothesis is: "Post-secondary treatment systems will have an optimal influence on further improving nutrient removal as well as disinfecting the final effluent from IAPS to allow for discharge to a water course that is not a listed water course as defined in the Water Act and required by the Department of Water Affairs.”

CHAPTER 2

Materials and Methods

2.1 IAPS configuration and operation

The Integrated Algae Pond System (IAPS) used in this study is located at the Institute for Environmental Biotechnology Rhodes University (EBRU), adjacent to the Belmont Valley Wastewater Treatment Works (33° 19' 07" South, 26° 33' 25" East). This IAPS operates continuously to treat 75 $m^3.d^{-1}$ of municipal sewage and a schematic showing the operating configuration and process flow is presented in Figure 2.1. The complete system comprises of an advanced facultative pond (AFP) with surface area of 840 m^2, which contains a single in-pond digester (IPD) or fermentation pit (225 m^3), two 500 m^2 high rate algae oxidation ponds (HRAOP), and two algal settling ponds (ASP). Up-flow velocity in the fermentation pit is maintained at 1-1.5 $m.d^{-1}$ while hydraulic retention times (HRT) in the fermentation pit and AFP are 3 and 20 d, respectively. Screened raw sewage is sourced directly from an off take immediately after the inlet works and enters the system via the IPD, where suspended and dissolved solids are anaerobically degraded. Effluent then flows into the buffering AFP and is detained for 20 d before gravitating to the first HRAOP which has HRT of 2 d and then to an ASP for half a day. Mixing, or turbulent flow, is essential to maintain optimum conditions for maximum algae productivity in the HRAOP's, which is currently flowing at 0.15 $m.s^{-1}$. Typically linear velocity is required to prevent stratification and is achieved using paddle wheels powered by a small electrical motor (0.25 kW). Due to configuration of the pilot demonstration and in accordance with original design parameters (Rose *et al.,* 2002b), partially treated water from the first ASP is pumped to the second HRAOP, where it is detained for 4 d before release to the second ASP. The latter is where the bulk of suspended algae biomass is removed by sedimentation prior to tertiary treatment and eventual discharge of the treated water.

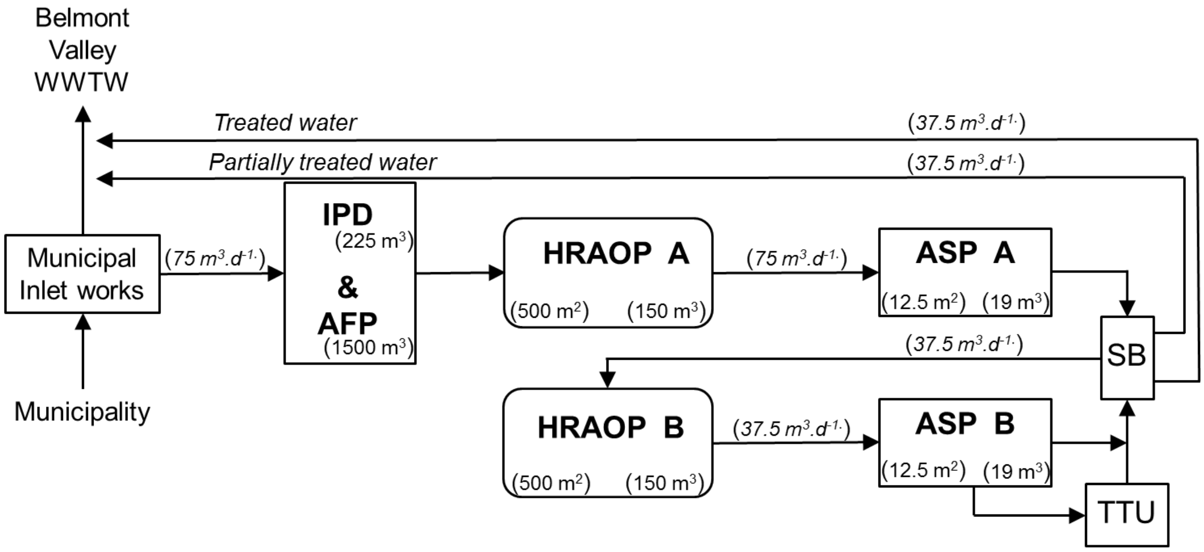

Figure 2.1: Schematic illustrating the process flow for the pilot IAPS designed, constructed and operational at the Belmont Valley WWTW, Grahamstown. The system receives 75 m^3 of raw sewage daily, screened for the removal of plastics, and a grit or detritus channel (in duplicate - one operating, one cleaning). Pond and reactor surface area, volume and flow rates are shown in parentheses. Effluent enters at the bottom of the AFP some 6 m below water level. AFP=Advanced Facultative Pond; IPD=In-Pond Digester; HRAOP=High Rate Algal Oxidation Pond; C/F=Coagulation/Flocculation; ASP=Algae Settling Pond; SB=Splitter Box; TTU=Tertiary Treatment Unit (Maturation Ponds, Slow Sand Filters and/or Rock Filters).

2.2 Design and Construction of Tertiary Treatment Systems

2.2.1 Design of the Maturation Pond Series

The quantity and size of the MP was determined by what the final bacteriological quality of the effluent should be. In South Africa, the recommended concentration of FC is 1000 counts/100 ml (Republic of South Africa, *Water Act 1998*). Therefore, the MP's were constructed in a series of 3 (Fig 2.2 & 2.5) based on suggestion that 2 to 3 MP's are most suitable for treating wastewater originating from a single facultative pond (Mohammed, 2006).

The depth of a MP is typically between 1 and 3 m with a long retention time for maximum pathogen removal. Shallower ponds (0.4 m) are more effective for decreasing pathogenic organisms and less land area is required (Kruzic and White, 1996). The first maturation pond MP 1 was designed with a diameter of 5 m and depth 1.2 m with the water level at 1 m, to prevent water overflow and also to increase UV light penetration (Table 2.1). It was reasoned that this would allow MP 1 to have an area of 19.6 m^2 and therefore a volume of 20 m^3. The positioning and depths of the inlet and outlet pipes tend to have a better outcome on the treatment competence than the actual pond geometry itself (Pearson *et al.,* 1995). The 2nd and 3rd maturation ponds MP 2 and MP 3 were designed to a smaller surface area of 1 m^2. The height of the plastic containers used for these ponds was 1 m and to prevent overflow, the water level is maintained at 0.8 m (Table 2.1).

The pond system must have a sufficiently long retention time in order to remove a significant amount of pathogens. Retention time needs to be at least 11 d to be effective, but for more effective removal, 37 d has been recommended (Wells, 2005). The retention time for the MP's in series in the current study was designed at 12 d, with a flow rate calculated to allow a retention time of 4 d in each pond (Table 2.1). According to Craggs (2005), total retention time of between 10 and 20 d is adequate for FC removal to levels less 1000 MPN per 100 ml.

A single vertical baffle system (Fig 2.2) was inserted in each of the MP's in order to prevent short circuiting (Bracho *et al.*, 2006). Introducing a baffle system allows for optimisation of the hydraulic behaviour of the pond through changes in configuration (Bracho and Casler, 2008). A single baffling system shows a vast improvement compared to unbaffled systems with regard to hydraulic efficiency but other studies have revealed that having more than one baffle improves the hydraulic efficiency significantly (Shilton and Sweeney, 2005). Nevertheless, a single baffled system was used in this study due to the geometry of the ponds and the cost involved to building multiple baffles.

Therefore and in order to construct the first MP in a circular shape, the following equations were applied:

$$\mathbf{Area = \pi\, r^2}$$
$$= (3.14)\,(6.25)$$
$$= \underline{\mathbf{19.63\ m^2}}$$

$$\mathbf{Volume = \pi\, (r^2)\, h}$$
$$= \underline{\mathbf{23.56\ m^3}}$$

A working volume of 20 m^3 was selected for the first MP

$$20 = \pi\, (6.25)\, h$$

h = **<u>1.02 m</u>**

To determine the flow into the first MP the following equation was used;

$$\mathbf{A = Q\, \Theta_{m1}/D}$$

Where A= Area (m^2)

Q= Influent Flow ($m^3.d^{-1}$)

Θ_{m1}= Retention time (d)

D = Depth

19.63 = Q (12)/1.0

12Q = (19.63) (1.0)

Q = <u>1.64 $m^3.d^{-1}$</u> (without the other 2 MP's)

For 3 MP = **<u>4.9 $m^3.d^{-1}$</u>** (retention of 4 d) (Mara, 2005).

Therefore the influent needed into the 2nd and 3rd smaller MP's to achieve a 4 d retention time:

$$\mathbf{\Theta_{m1} = AD/Q}$$

4= (1) (0.8)/Q

Q = **<u>0.2 $m^3.d^{-1}$</u>**

2.2.2 Construction of the Maturation Pond Series

MP 1 was built using PVC lining (5 × 1.2 m) which was supported by steel fencing on the outside. The baffle was also made of PVC lining. MP 2 and 3 were constructed from 1 m^3 plastic containers with their baffle systems also being made of plastic. The baffles were positioned 0.2 m above the ground level which allowed for flow to be directed underneath the baffles (Fig 2.2).

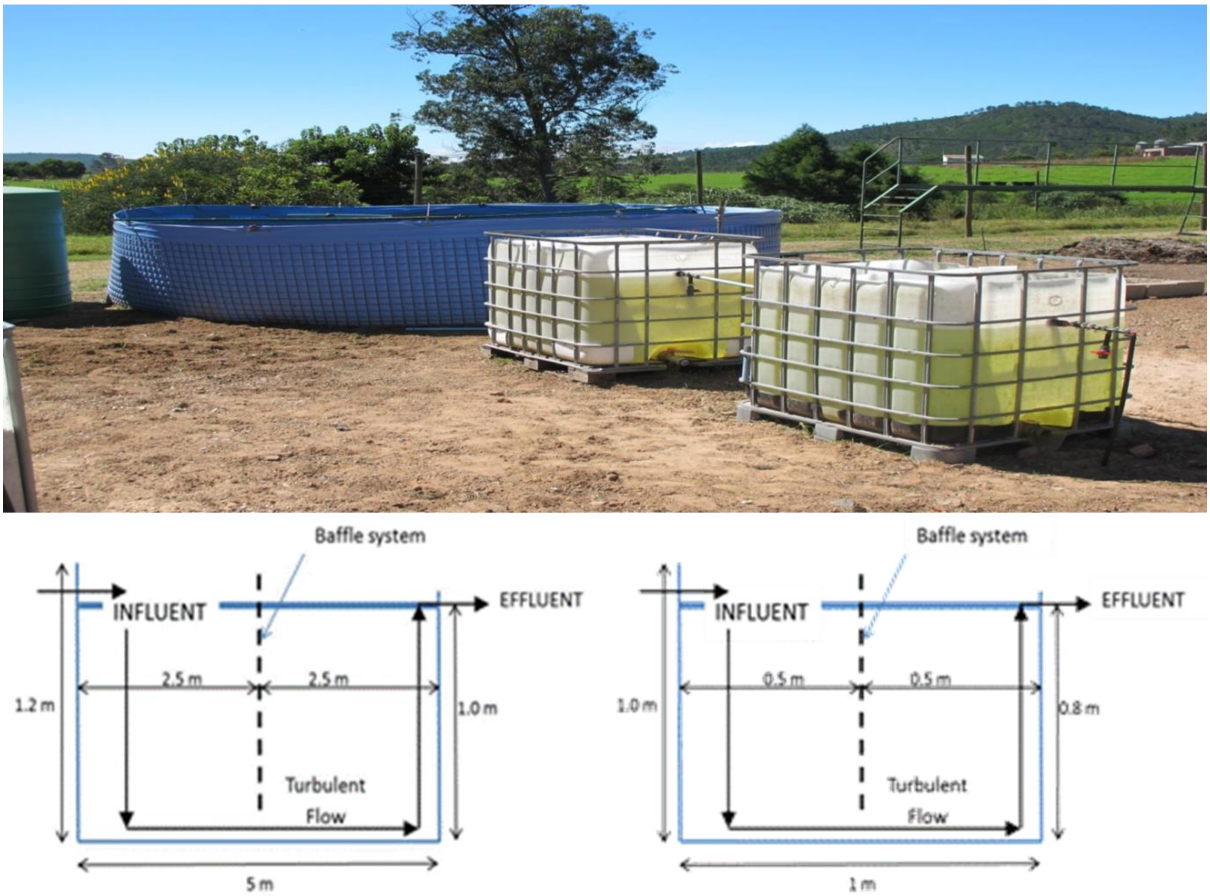

Figure 2.2: Schematic diagram of MP 1 on the left and MP 2 and 3 on the right. All 3 MP's have a single baffle which prevents water from short-circuiting and causes a turbulent flow. In MP 1, the water level goes up to 1 m, which gives a water volume of 20 m^3. In MP 2 & 3, the water level is at 0.8 m, which gives a water volume of 0.8 m^3.

2.2.3 Design of a Slow Sand Filtration Unit

The SSF was constructed using a 1500 L JoJo® tank (1 × 1.5 m) with a volume of 1.5 m^3 (Table 2.1). Gravel sand (15-22 mm) was used as the first layer with fine sand (1-2 mm) used as the second and then 0.8 m of water head (Fig 2.3). A two-layer filtering design was used because a SSF does not usually have more than one layer of sand above the supporting media and to provide sufficient support to the schmutzdecke, the main filtration layer. Both sand types were obtained locally from Trencrush Quarries in Grahamstown. The reason why the water head is so deep is because over time, the schmutzdecke causes clogging which decreases water flow into the system, hence an increase in water volume on the surface (Massmann *et al.*, 2004). Woven type BIDIM®, obtained from a BUCO hardware outlet store (originally from the GEOfabrics company), was used as a filter matrix between the two layers of sand and to cover the top surface to ease cleaning and to prevent mixing of the sand layers (Fig 2.3). The system was plumbed using 25 mm piping which connected outflow from the

IAPS splitter box (SB) to a second splitter and the various TTU's connected to the SSF unit using 15 mm piping.

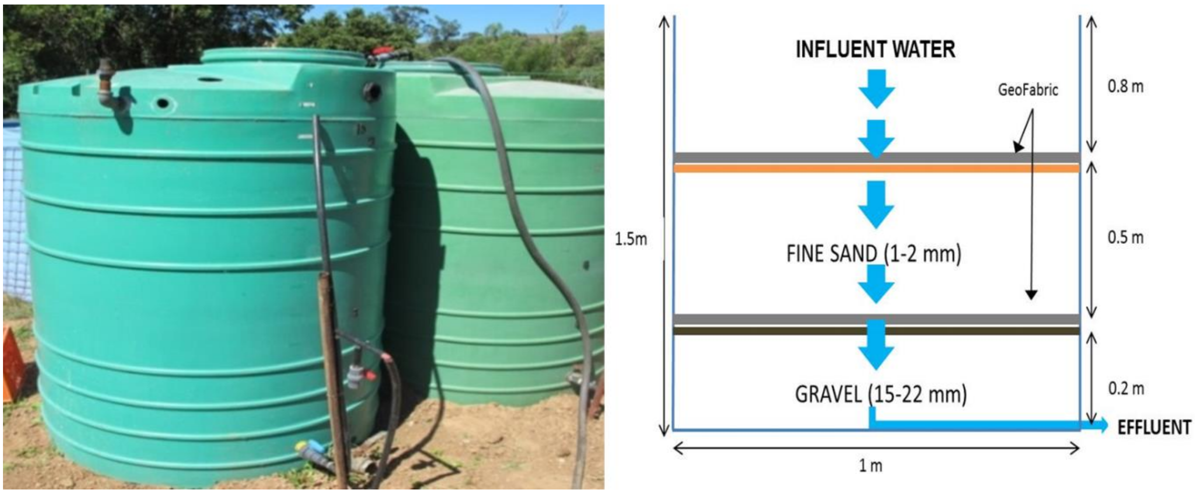

Figure 2.3: Schematic diagram of the SSF's. Both SSF's contain two layers of sand (fine sand and gravel) which are both supported by a layer of BIDIM® for easier cleaning and less mixing. Gravel is 0.2 m in depth; fine sand is 0.5 m in depth and 0.8 m water head.

In the present study two SSF's were designed and constructed and used separately. The reason for approaching implementation of SSF in this way was due to the high amount of algae present in the effluent stream which forms a biological mat, also known as a schmutzdecke on the surface of the sand filter. Small unicellular algae increase resistance to flow into the filter, causing clogging within the system (McNair *et al.*, 1987). Therefore, two SSF's were used to ensure continuous operation and avoid down-time required for removal of the schmutzdecke. Slow sand filters are normally cleaned by scraping the biological layer from the surface of the sand (di Bernardo, 2002). Furthermore, BIDIM® was used to cover the surface of the SSF to minimize ingress of algae and for easier cleaning.

The design criteria for the SSF units used was based on the following;

Volume water = π r^2 h

$$= (3.14)\ (0.25)\ (0.8)$$
$$= 0.63\ m^3$$
$$= (2)\ (0.63)$$
$$= \underline{\mathbf{1.26\ m^3}}$$

Volume fine sand = π r^2 h

= (3.14) (0.25) (0.5)

= 0.39 m^3

= (2) (0.39)

= **<u>0.78 m^3</u>**

Volume gravel = π r^2 h

= (3.14) (0.25) (0.2)

= **<u>0.16 m^3</u>**

= (2) (0.16)

= **<u>0.32 m^3</u>**

Area = π r^2

= **<u>0.785 m^2</u>**

The hydraulic loading rate (HLR) plays an important role in the operation of filters. A loading rate of 0.1-0.32 $m.h^{-1}$ is usually recommended but the literature has also suggested that a rate of 0.6 $m.h^{-1}$ is viable (Tyagi *et al.,* 2009). Therefore the following equation was implemented to determine the HLR for the sand filtration components (McDowall, 2008):

A = Q/ HLR

A = Area (m^2)
Q = flow rate ($m^3.d^{-1}$)
HLR (hydraulic loading rate) ($m.h^{-1}$)

0.785 = 0.3/ HLR
HLR= 0.3/ 0.785

HLR= **<u>0.38 $m.h^{-1}$</u>**

2.2.4 Design of a Rock Filter Unit

The rock filters were constructed using a series of 3 plastic containers each measuring a 1.0 × 1.0 m each filled with gravel sand to a depth of 0.6 m (Table 2.1). It is suggested that ponds in series show good efficiency with regard to the removal of parasites as well as the eggs (Konaté *et al.*, 2013). Rock filters have an average particle size of between 5 – 20 mm; therefore gravel particles ranging from between 15 – 22 mm in diameter were used (Hussainuzzaman and Yokota, 2006). The positioning of the inlet piping (15 mm) to the 3 gravel sand filters was placed at the bottom of the RF's which allowed an upflow of water into the systems and tends to give a better result (Fig 2.4) (Middlebrooks *et al.*, 2005).

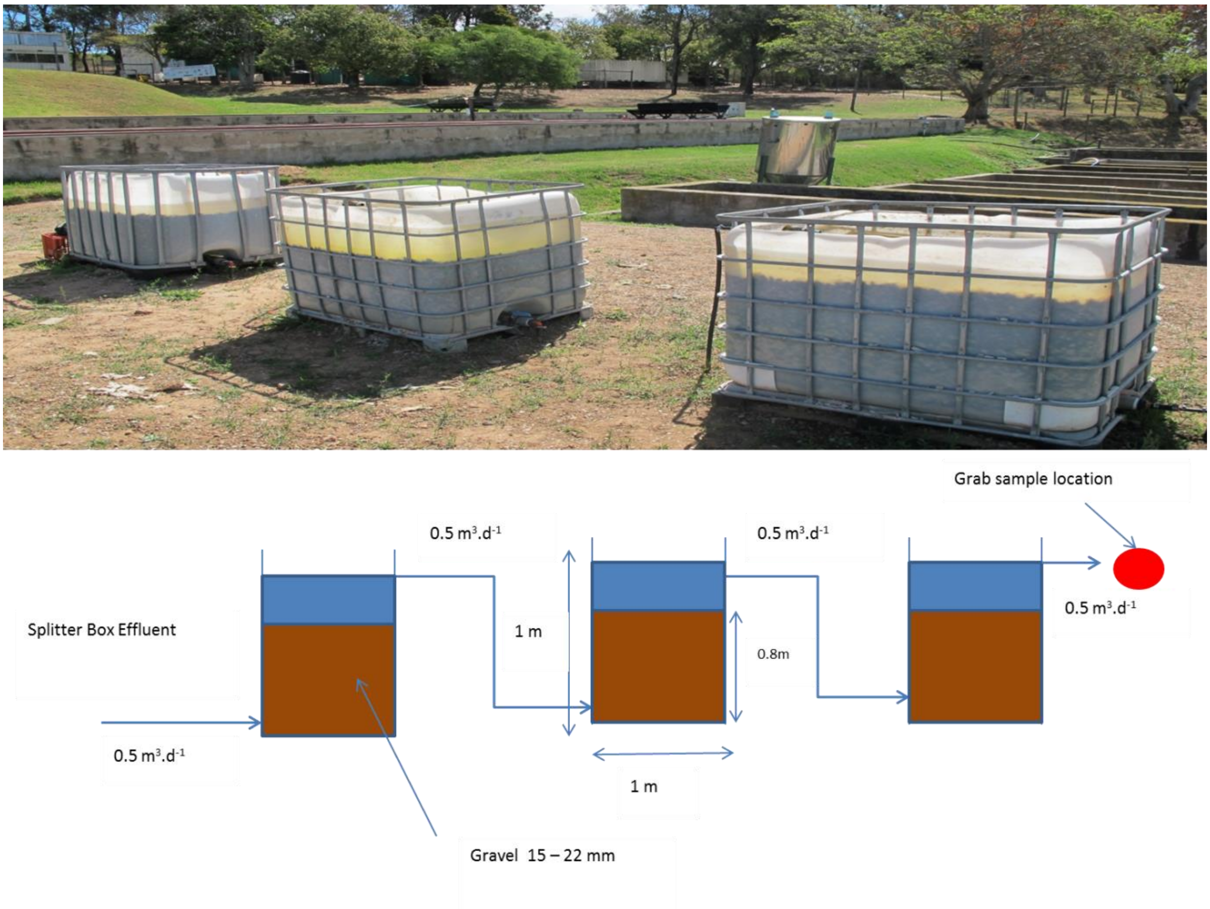

Figure 2.4: Schematic diagram of the rock filters in series. Each container is 1 × 1 m with the gravel sand being 0.6 m in depth and a water head of 0.3 m. The flow rate was 0.5 $m^3.d^{-1}$ and the rock particles ranged between 15 – 22 mm.

Hydraulic loading rate (HLR) is one of the most critical factors when it comes to RF design and it is important that the flow of the wastewater is under the rock surface to prevent algal growth and insect annoyance. It has been suggested that in order to achieve the required efficiency with 1- 2 cm diameter rock, the HLR must be between 0.15-0.30 $m^3.d^{-1}$ (Middlebrooks *et al.*, 2005). Thus for the RF unit to be used in the present study the following parameters were applied;

A = Q/ HLR

A = Area

Q = flow rate ($m^3.d^{-1}$)

HLR (hydraulic loading rate) ($m.h^{-1}$)

1.0 = 0.5/ HLR

HLR= 0.5/ 1.0

HLR= **<u>0.5 $m.h^{-1}$</u>**

Table 2.1: Design parameters for the TTU's; Maturation Ponds; Slow Sand Filters; Rock Filters.

	Maturation Pond		Slow Sand Filter	Rock Filter
	MP 1	**MP 2 & 3**		
Area (m^2)	19.63	1	0.785	1
Volume (m^3)	20	0.8	1.5	0.9
Depth (m)	1.02	0.8	1.5	0.9
No of Units	1	2	2 (separate)	3
Flow Rate ($m^3.d^{-1}$)	4.9	0.2	0.3	0.5
HLR ($m.h^{-1}$)	0.25	0.2	0.38	0.5 (per filter)
HRT (d)	4	4 (in both)	5	5.4 (in total)

2.3 Experimental design

Composite water samples were collected from secondary and tertiary treated effluent at intervals spanning 24 h from outlet points of discharge from the various systems. These included; the final effluent from the IAPS, the final effluent of the MP (after MP 3), the effluent emerging from the bottom of the SSF as well as the effluent from the final RF in series (Fig 2.5). Composite samples were prepared using a large container next to each treatment unit into which the effluent water would flow over a 24 h period. A thoroughly mixed 500 mL grab sample was taken from each of the composite sampling points on a weekly basis for a 10 month period from February-November 2013.

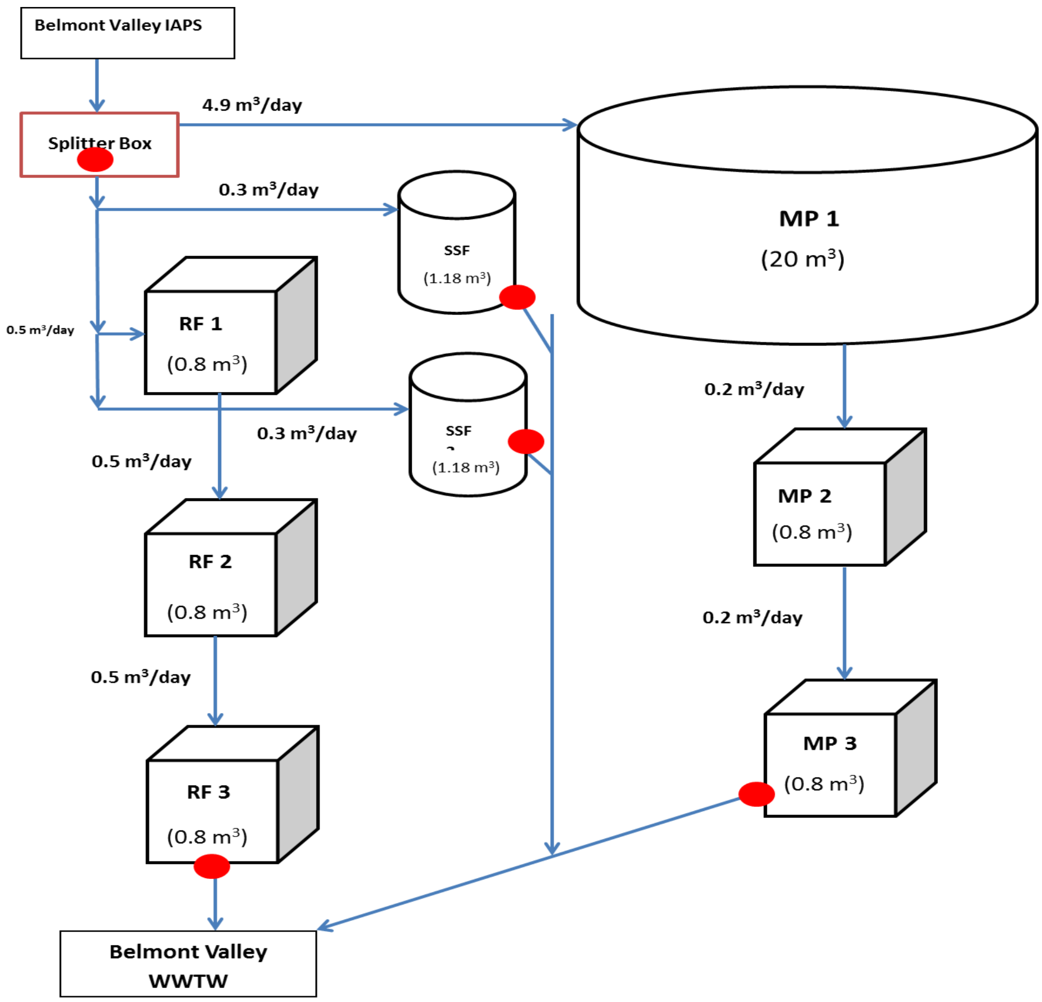

Figure 2.5: Design layout and process flow of the TTU's positioned after the IAPS treatment of domestic wastewater. IAPS effluent, after algae settling, is distributed to the MP series, SSF's and RF series from a SB. SSF's receive 0.3 $m^3.d^{-1}$, while the RF's receive 0.5 $m^3.d^{-1}$. The MP 1 receives 4.9 $m^3.d^{-1}$ and MP 2 and 3 each 0.2 $m^3.d^{-1}$. SSF = Slow Sand Filter; MP = Maturation Pond series; RF = Rock Filter. Sampling points are shown in red.

2.3.1 Replicates

All sampling and analyses of the water were in triplicate and carried out within 1 hour (Mambo *et al.*, 2014). The pH, DO and EC measurements were analysed 3 times during the course of the day (morning, midday and late afternoon) from the composite sampling points.

2.3.2 Sample Preparation

pH, EC and DO were measured in situ. EC and temperature were measured immediately using an OAKTON EC/TDS/SALT Testr 11 Dual range 68X 546 501 meter (Eutech Instruments, Singapore), while DO content was determined using an EUTECH DO 6+ 859

346 meter (Eutech Instruments, Singapore). The pH was measured using a Hanna HI8 424 microcomputer pH meter (Hanna Instruments, Woonsocket, RI) previously calibrated using Merck Buffer solution pH 4. SAAR 1461040KF and pH 7. SAAR 1461070KF (Mambo *et al.*, 2014).

TSS was measured according to APHA (1998). This was done by filtering 50 mL of wastewater through Whatman GF/C microglass filters, drying the filters for 24 h in an oven at 105ºC and determining the mass differential. Filter weight was determined before and after filtration and TSS calculated using the following formula:

$$\text{mg total suspended solids/L} = \frac{(\text{A}-\text{B})\times 1000}{\text{sample volume (mL)}} \times 1000$$

where:

A = weight of filter + dried residue (mg)

B = weight of filter (mg)

Nutrient analyses were carried out using test kits; Ammonium-N (1.14752.0001), ortho-phosphate (1.14848.0001), nitrate-N (1.14773.0001) and COD (1.14538.0065 & 1.14539.0495) according to the manufacturer's instructions (Merck Chem. Co., Darmstadt, Germany). These were analysed in triplicate using a Thermo Spectronic Aquamate spectrophotometer (ThermoFisher Scientific, Waltham, MA).

Concerning the testing of COD, an estimated 20 ml of effluent water from all the treatment systems was filtered separately through Whatman No. 1 filter paper with a pore size 11 µm. The COD testing kit was then used for the effluent water in sealed test tubes according to the manufacturer's instructions. Once the sample was prepared, the sealed test tubes were then placed into a Merck thermoreactor (preheated for 30 min) for 2 h at 148°C. Samples were then allowed to cool to room temperature before analysis using the Thermo Spectronic Aquamate spectrophotometer.

The effluent water was also filtered separately through Whatman 2 filter paper (8 µm), where the filtered water was then used to test ammonium-N. After filtration, the different effluents were then placed in test tubes and analysed using the ammonium-N testing kit in the Thermo Spectronic Aquamate spectrophotometer (Mambo *et al.*, 2014).

Microbial analyses were carried out in triplicate using either MacConkey agar for total coliforms or m-Fc agar for FC, which were obtained from BIOLAB CHEMICALS CC, South Africa and prepared according to the manufacturer's instructions. Aliquots of water sampled from the IAPS, MP, SSF and RF effluents were diluted using 9 ml sodium chloride solution and 1 ml of water sampled effluent. Only the IAPS water effluent was diluted again to a dilution of 1/100. From these dilutions, 100 µL was inoculated on petri dishes using a spreader. The MacConkey and m- Fc agar plates were then incubated at 30 and 45 ºC respectively for 24 h prior to estimation of colony forming units (cfu). All the plates were counted and then divided by 3 to get an average for each system. These averages were then multiplied by their dilution factor to get the correct microbial count estimation (Mambo *et al.,* 2014).

2.3.3 Statistical analysis

Composite samples were taken as mentioned in the beginning of section 2.3 above. Where triplicate samples were taken as mentioned in section 2.3.1. All statistical analyses were computed using Microsoft Excel 2010. All measurements are the mean ± the standard deviation. Since all the measurements were in triplicate, all 3 data points were used to determine the mean and standard deviation. The standard error bars on the graphs (Chapter 3) were derived using the standard deviation calculations (Microsoft Excel 2010), for all the measurements (with the exception of faecal and total coliform counts).

A one-tailed distribution t-test was used to analyse two- sample unequal variances (heteroscedastic) on Microsoft Excel 2010, to determine the level of significance between the IAPS and the TTU systems. Analyses of variance (ANOVA) were also conducted using Microsoft Excel 2010 to test the differences between all the data sets.

CHAPTER 3

Effect of tertiary treatment units on water quality from an IAPS treating municipal sewage

3.1 Introduction

In this chapter, the effect of tertiary treatment on final water quality of the effluent from an Integrated Algae Pond System (IAPS) treating municipal sewage is investigated. Most wastewater treatment systems (WWTS) have as a component part of the process a tertiary treatment step and in South Africa; this typically includes maturation ponds in series (MP's). The most common form of municipal sewage treatment in South Africa is WSP's followed by polishing in maturation ponds. Indeed, almost 50% of all WWTS in South Africa are WSP systems and for the most part these treat volumes < 1 ML per day and are rural. In many instances these systems are dysfunctional due to overloading, poor maintenance or simply through neglect. One possibility that has emerged recently is the upgrading of these WWTS by implementation of IAPS technology. Unfortunately, IAPS technology in South Africa is perceived as being unsuitable due to the apparent inability of this process to produce a final effluent of sufficient quality for discharge to the environment (Rose *et al.*, 2007). The Water Act (Republic of South Africa, *Water Act 1998*) requires that WWTS discharge effluent to water resource that are not listed water resources according to the following specification: ammonia nitrogen ≤ 3 mg. L^{-1}; *ortho*-phosphate ≤ 10 mg. L^{-1}; nitrate/nitrite nitrogen ≤ 15 mg. L^{-1}; COD ≤ 75 mg. L^{-1} (after removal of algae); pH 5.5-9.5; faecal coliforms (per 100 mL) ≤ 1 000 and electrical conductivity 70-150 mS.m^{-1}. Following commissioning of a pilot IAPS system at the Belmont Valley WWTW, Grahamstown and after operation of the components to test the suitability of IAPS under South African conditions the following was concluded:

- The system did not achieve the 75 mg. L^{-1} discharge standard for COD_t,
- Although a reduction in phosphate was observed, it was not within the 10 mg. L^{-1} required for discharge,
- Residual ammonia levels exceeded the 3 mg. L^{-1} discharge standard,
- Nitrate removal was at best erratic and at times, nitrate concentration increased (Rose *et al.*, 2007).

These findings are in contrast to many published studies on the operational efficiency of IAPS for treatment of municipal sewage including systems located in Kuwait (Esen *et al.,* 1987; Banat *et al.,* 1990; Al-Shayji *et al.,* 1994), the U.S.A. (Lundquist *et al.,* 2010) and New Zealand (Park and Craggs 2010; 2011a;2011b).

A detailed evaluation of IAPS as a technology and of the pilot system at Belmont Valley revealed that the latter was designed and commissioned as a system that would at best deliver secondary treated water (Mambo *et al.,* 2014). Although intended as a demonstration unit, it was sized to provide credible performance data and is purportedly suitable for engineering scale-up requirements. Design rationale and calculations were provided by Prof William Oswald and Dr. Bailey Green consulting then as "Oswald Green", and were used as the basis for the conceptual plan for construction of the pilot plant (Rose *et al.,* 2002). The system was built to have the capacity to treat the liquid wastes of 500 person equivalents (P.E.) and an average water consumption and disposal per capita of approximately 150 $L.d^{-1}$ was assumed. Accordingly, the design influent flow was calculated at 75 $m^3.d^{-1}$. With an ultimate Biochemical Oxygen Demand (BOD_{ult}) assumed to be 80 g BOD_{ult} P.E. per day, the organic loading to the system is 40 $kg.d^{-1}$. Using an assumed conservative BOD loading to the HRAOP from the AFP, the depth in the two HRAOP's is maintained at 30 cm. The total volume of each pond is therefore 150 m^3, with a surface area of 500 m^2. Using adjustable overflow weirs the hydraulic loading and thus HRT, in the HRAOP's can be adjusted to equal or less than influent flow (up to a maximum of 75 $m^3.d^{-1}$ for this design specification) for experimental purposes, but is generally operated between 3 and 6 d. The algae floc is kept in suspension in the raceways of the HRAOP's by a paddlewheel which serves as a pump to maintain a linear velocity of 30 $cm.s^{-1}$.

As shown in Table 3.1, the Belmont Valley WWTW pilot IAPS when operated in a fully managed mode and without any tertiary treatment, routinely yields treated water close to the standard for discharge to a water course that is not a listed water course according to the General Authorisations in terms of Section 39 of the national water act (Republic of South Africa, *Water Act 1998*). Efficiency of the system is largely due to COD reduction and nutrient abstraction in the HRAOPs which is inextricably linked to algae productivity.

Table 3.1: **Water quality parameters for discharge into the environment as specified by department of water affairs (Republic of South Africa, Water Act 1998).**

PARAMETERS	STANDARD
Ammonia-N (mg. L^{-1})	3
Phosphate (mg. L^{-1})	10
Nitrate-N (mg. L^{-1})	15
COD (mg. L^{-1})^{A}	75
pH	5.5 – 9.5
Faecal Coliforms (cfu/100 mL)	< 1000
Total Suspended Solids (mg. L^{-1})	25
Dissolved Oxygen (mg. L^{-1})	>2
Electrical Conductivity (mS.m^{-1})	70 - 150

^{A}Following removal of algae

Even so, this pilot plant affords a secondarily treated water for reclamation according to its design specifications which most closely resemble those of the AIWPS® Advanced Secondary Process developed by Oswald. As a consequence, and as might be expected, while the technology performs well and delivers a final effluent superior to most pond systems deployed in South Africa it remains unable to meet The Department of Water Affairs (DWA) General Standard for nutrient removal and effluent discharge. Thus, it is foreseen that the addition of an appropriate tertiary treatment unit with the capability to remove nutrients, residual organics and pathogens will provide the necessary effluent polishing to ensure that this technology is compliant. In the present study, several tertiary treatment units were investigated to determine whether these would enhance water quality of the IAPS effluent and meet the General Authorization limits in terms of both TSS and COD for discharge to a water resource.

3.2 Results

One of the problems for the IAPS is not producing quality effluent for discharge into the environment. After due consideration of available technologies, time, cost of construction, implementation and suitability for use, MP's, SSF's and RF's were selected as the process units for tertiary treatment of the Belmont Valley IAPS final effluent. Thus, the purpose of this chapter is to compare the IAPS final effluent to the TTU's final effluent and determine if they are effective in further polishing the water from the IAPS final effluent. Parameters considered important for due diligence and measured routinely to enable evaluation included; chemical oxygen demand (COD), nitrate, ammonium, phosphate, total suspended solids

(TSS), pH, faecal coliforms (FC), total bacteria count (TBC), electrical conductivity (EC) and dissolved oxygen (DO).

3.2.1 Maturation Ponds as a tertiary treatment unit

Maturation ponds are probably the most common form of tertiary treatment in the wastewater treatment process. As shown in Figure 3.1, a MP's reduced the concentration of ammonium-N, phosphate-P and nitrate-N in the final effluent during the course of the sampling period. The ammonium-N data for the MP final effluent was averaged at 1.08 ± 1.2 which represents a 68% removal rate (Fig. 3.1 A). Furthermore, water quality of the final effluent showed that using a MP series as a tertiary treatment resulted in a 17% reduction of phosphorus (6.97 ± 5.4) (Fig 3.1 B). Therefore there was no significant difference between the IAPS and the MP (t-test: $p= 0.1$). (Appendix A) ($r^2= 0.8$; $v= 29.02$; $p= 8.06E-10$). Nitrate-N level in the final effluent revealed that the MP's had reduced levels of this nutrient by 40 % (Fig 3.1 C). Thus, nitrate-N levels were generally lower than the statutory limit (4.50 ± 3.0) required for discharge to a water course.

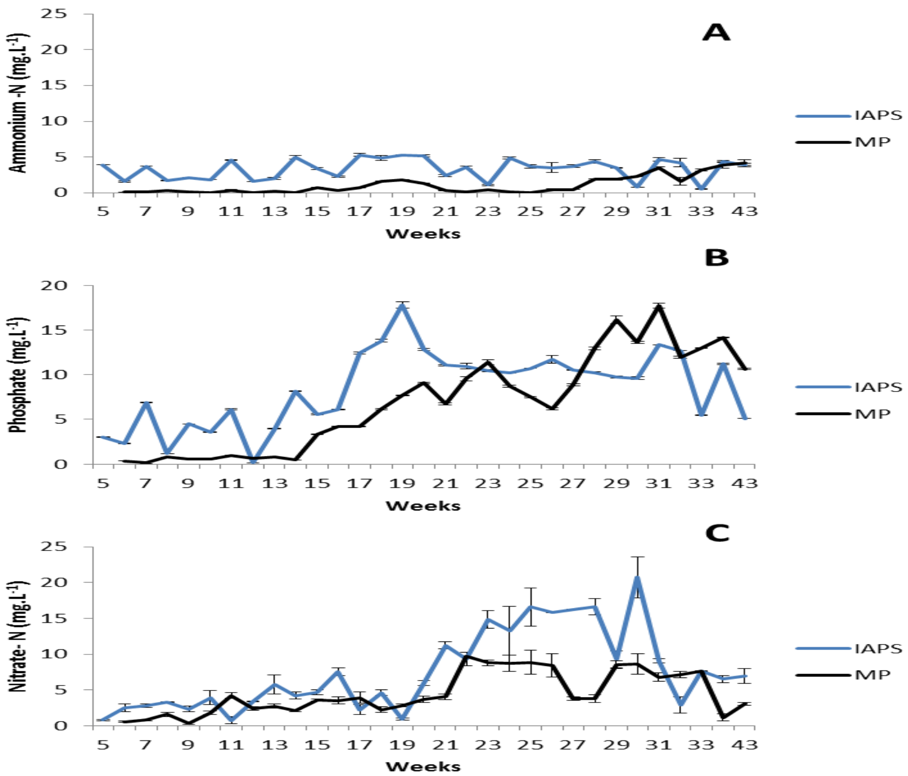

Figure 3.1: Comparison of the nutrient content and composition of treated water between the maturation pond and the IAPS at the point of discharge. A) Ammonium-N, B) Phosphate-P, and C) Nitrate-N concentrations in composite samples harvested

weekly over a 24 h period for 10 months were determined using testing kits as described in the Chapter 2. Data presented are the average of triplicate measurements.

Chemical oxygen demand removal ($COD_{filtered}$), after the removal of algae had a 17% removal rate from the IAPS final effluent (Fig 3.2 A). Although COD reduction was significant, the levels still fluctuated (78.25 ± 22.7). Total suspended solids were routinely out of range and well above the required limit of 25 $mg.L^{-1}$ (45.40 ± 31.5) (Fig 3.2 B). However the MP's final effluent did have a significant decrease in TSS levels compared to the IAPS final effluent (61.48 ± 37.7). Total suspended solids also seemed to increase over time, especially from week 16 to week 43.

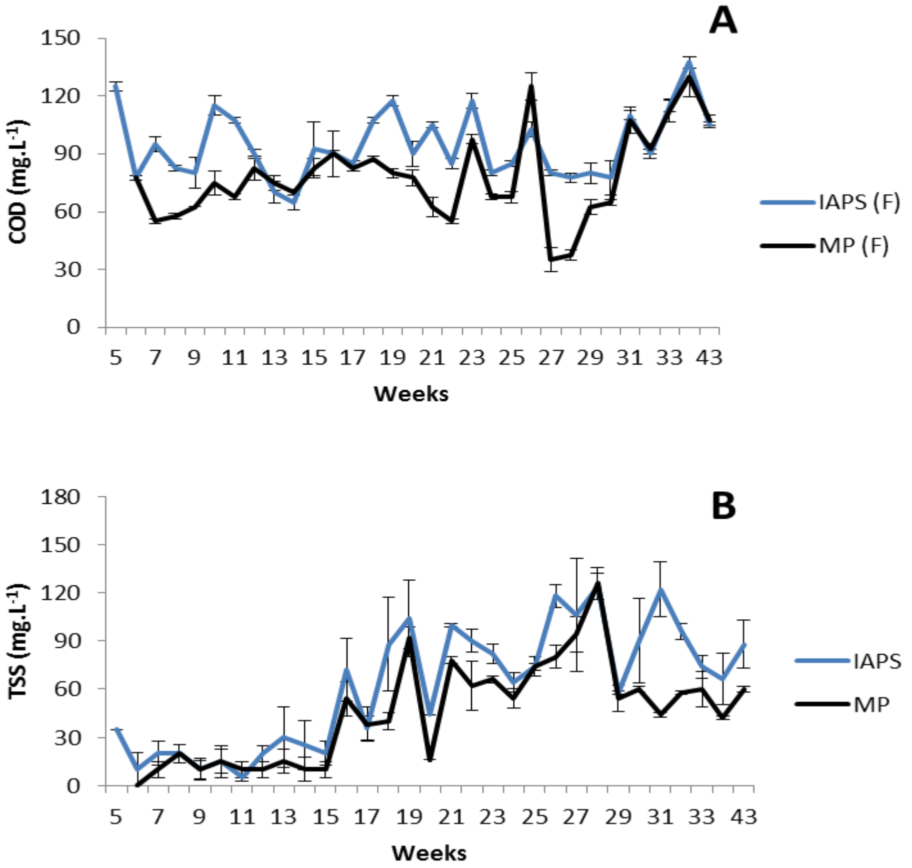

Figure 3.2: Comparison of the chemical oxygen demand and total suspended solids between the maturation pond and IAPS at the point of discharge. A) Chemical oxygen demand ($COD_{filtered}$), B) Total suspended solids were determined from composite samples taken weekly over a 24 h for 10 months. Data presented are the average of triplicate measurements.

Results show that the pH of the MP effluent was routinely above 9.5 and was therefore not within DWA's statutory limit, especially during the initial testing period (week 5-18) (Fig 3.3 A). The mean pH was out of range (9.55 ± 0.8), however there was still significance within the results and therefore the null hypothesis was rejected. From week 27 to week 43, the pH levels of the MP final effluent declined to within the limited range. The effluent from the MP's had high DO levels (11.7 ± 4.6 mg. L^{-1}) (Fig 3.3 B) and dissolved oxygen in this water was significantly higher than that of the IAPS effluent. Regression analyses of data using a 1 way ANOVA revealed that the variance was minimal (r^2= 0.07; v= 20.74; p= 1.95E-12) (Appendix A). Electrical Conductivity for the MP was 117 ± 4.8 (Fig 3.3 C). Data had showed that there was no significance between the MP and the IAPS values (t-test: p= 0.2) and therefore the null hypothesis was not rejected.

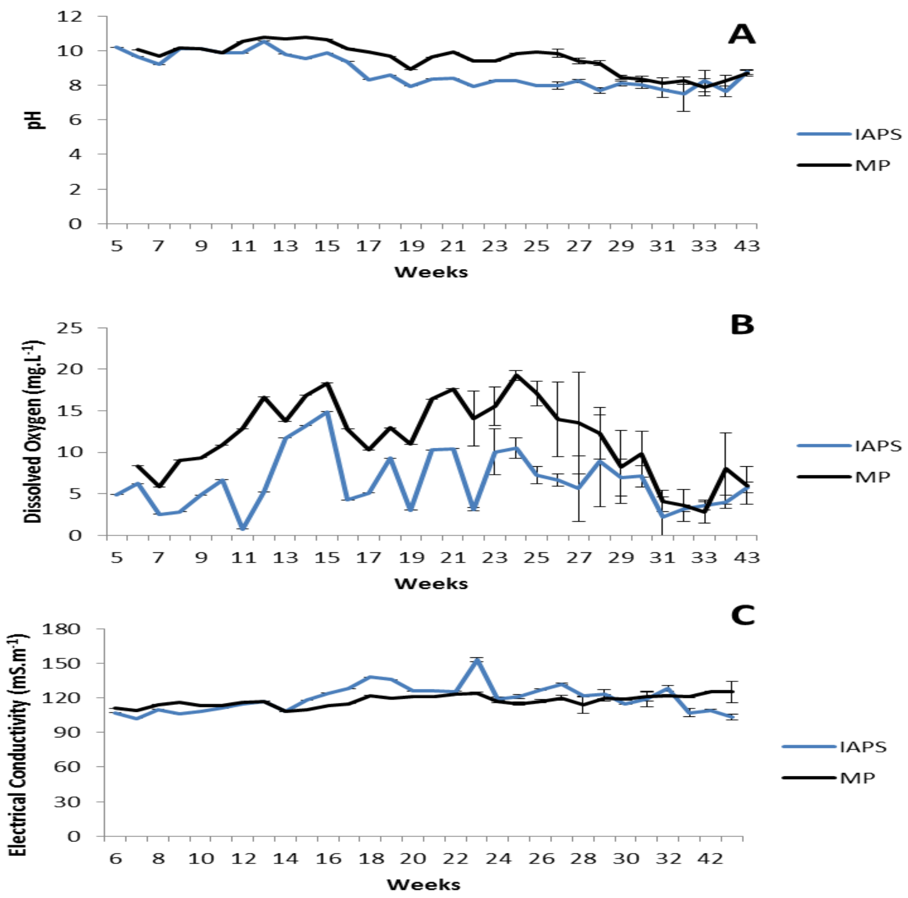

Figure 3.3: Comparison of the physiochemical characteristics of treated water between the maturation pond and the IAPS at point of discharge. A) pH, B) Dissolved oxygen (DO) and C) electrical conductivity (EC) were determined for composite samples collected weekly for 24 h over a period of 10 months. Data presented are the average of triplicate measurements.

In this study, use of a maturation pond series as tertiary treatment resulted in a 92% removal of FC (Fig 3.4 A). From week 19, the FC count showed a good level of consistency (<1000). The regression analyses indicated less variance among the data points (r^2= 0.1; v= 2497519; p= 0.003). Total coliform count in the final effluent after passage through a MP's showed a

94% reduction, which was significant (Fig 3.4 B). A one way ANOVA test showed the data to have very little variance (r^2= 0.02; v= 80558712; p= 0.02).

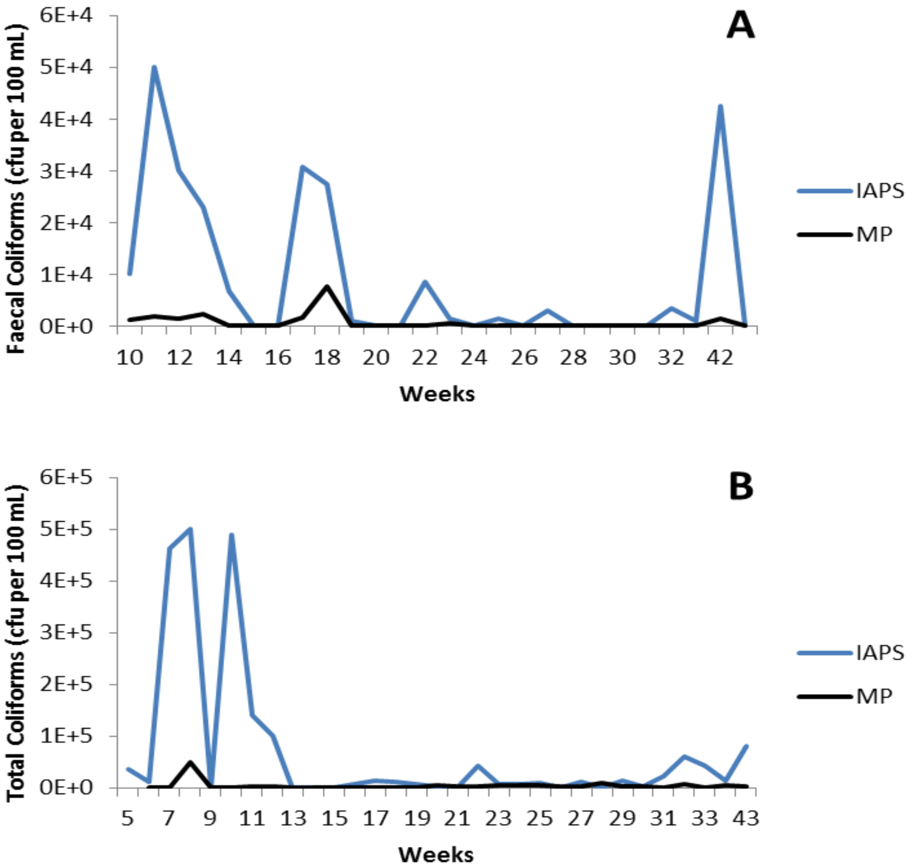

Figure 3.4: Comparison of the different coliform counts between the maturation pond and IAPS at the point of discharge. A) faecal coliforms (FC), B) total coliforms (TC). Composite samples were analysed weekly over a 24 h period. The FC was sampled over a period of 9 months while the TC was sampled over 10 months. Data presented are the average of triplicate measurements.

3.2.2 Slow sand filtration as a tertiary treatment unit

Slow sand filters as a tertiary unit is an efficient method for the treatment wastewater. It is shown in Figure 3.5 that the ammonium-N, phosphate-P and nitrate-N concentration levels were reduced in the SSF's final effluent. The SSF had a 40% removal of ammonium-N with an average of 2.02 ± 1.3 (Fig 3.5 A). Regression analyses using a 1 way ANOVA revealed that the data showed minimal variance (r^2= 0.1; v= 1.64; p= 8.06E-14). Phosphate levels of the SSF's final effluent had an average of 2.57 ± 2.5 which represents a 70% removal rate (Fig 3.5 B). Furthermore, the final effluent from the SSF had 9% removal rate of nitrate-N

(6.80 ± 5.5) (Fig 3.5 C). Therefore, the data between the SSF and IAPS was not significant (t-test: p= 0.3).

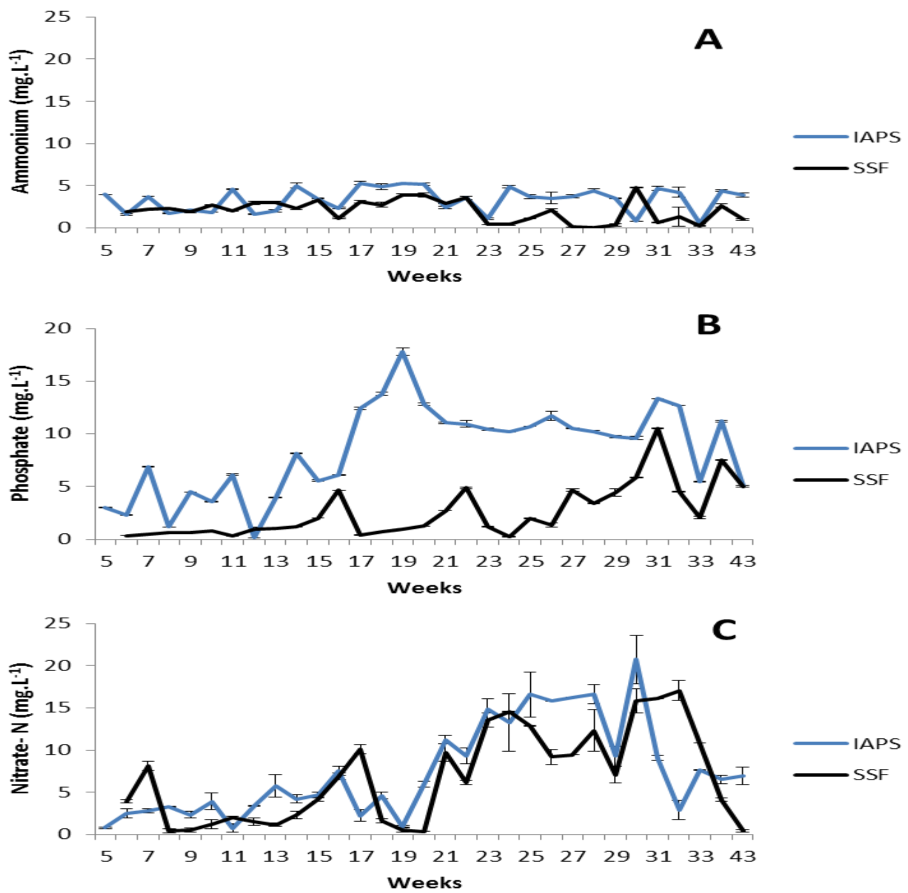

Figure 3.5: Comparison of nutrient content and composition of treated water between the slow sand filters and the IAPS at the point of discharge. A) Ammonium-N, B) Phosphate, and C) Nitrate-N concentrations in composite samples harvested weekly over a 24 h period for 10 months were determined using testing kits as described in the Chapter 2. Data presented are the average of triplicate measurements.

Chemical oxygen demand ($COD_{filtered}$) concentrations from the SSF were generally lower than the statutory limit (58.67 ± 13.0) and therefore had a 38% removal rate (Fig 3.6 A). The $COD_{filtered}$ data values were significant and also had less variance according to the ANOVA statistical analyses (r^2= 0.04; v= 173.98; p= 3.11E-12). However, the TSS levels in the SSF were above the required limit of 25 mg.L^{-1} (44.17 ± 34.8) (Fig 3.6 B). But, although the TSS were generally higher than the statutory limit, there still was a significant decrease in TSS levels compared to the IAPS final effluent (t-test: p= 0.03).

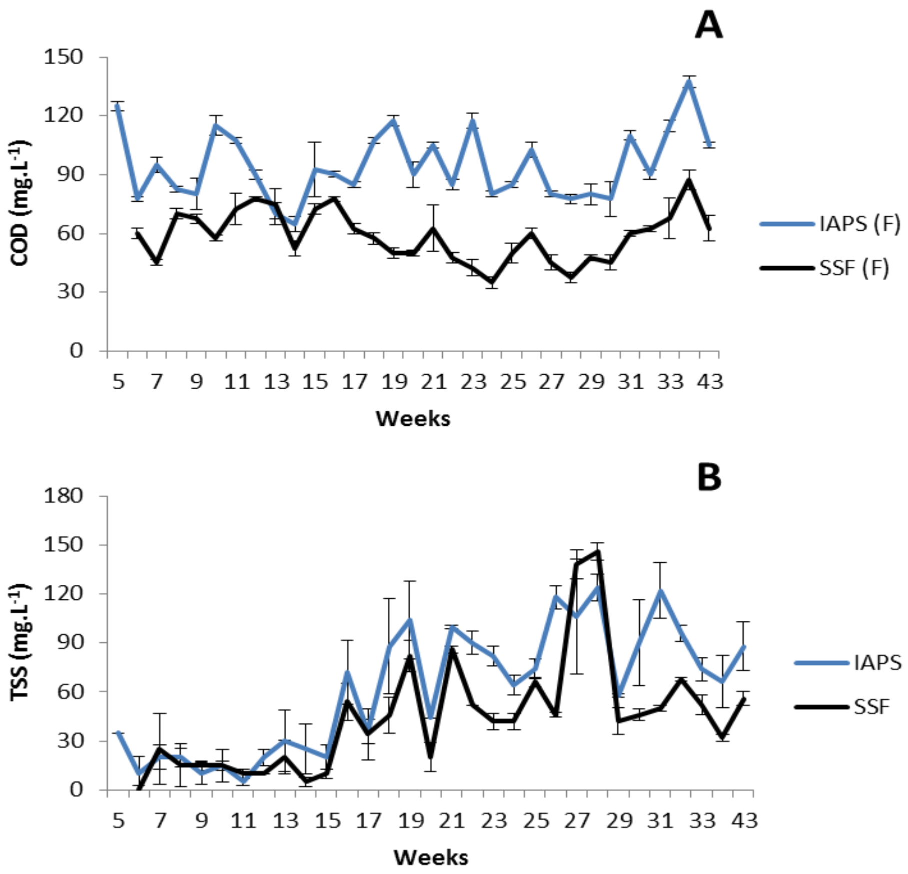

Figure 3.6: Comparison of the chemical oxygen demand and total suspended solids between the slow sand filters and IAPS at the point of discharge. A) chemical oxygen demand ($COD_{filtered}$), B) total suspended solids were determined from composite samples taken weekly over a 24 h for 10 months. Data presented are the average of triplicate measurements.

The pH levels of the SSF final effluent were constantly within DWA's statutory limit (8.29 ± 0.7) (Fig 3.7 A). Regression analyses indicated minimal variance among the data points (r^2= 0.02; v= 0.48; p= 2.77E-07). Dissolved oxygen levels in the SSF final effluent were low compared to the IAPS and the other TTU systems but still within the required limit (6.07 ± 2.5) (Fig 3.7 B). There was also minimal variance between the SSF and IAPS according to ANOVA statistical analyses (r^2= 0.1; v= 6.26; p= 1.95E-12). The SSF had electrical conductivity levels that were routinely within the required limits (118 ± 11.8) (Fig 3.7 C). Data revealed no significant difference between the IAPS and SSF data, therefore, the null hypothesis was not rejected (t-test: p= 0.27).

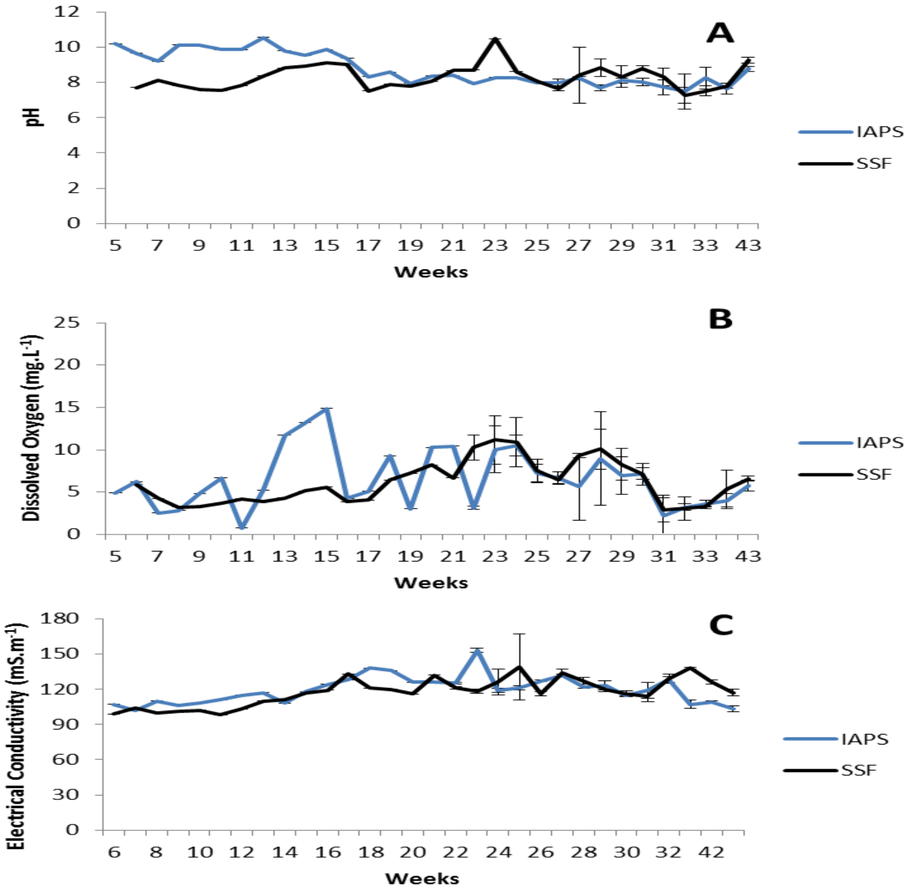

Figure 3.7: Comparison of physiochemical characteristics of treated water between the slow sand filters and IAPS at point of discharge. A) pH, B) Dissolved oxygen (DO) and C) electrical conductivity (EC) were determined for composite samples collected weekly for 24 h over a period of 10 months. Data presented are the average of triplicate measurements.

The SSF's final effluent had a 66% removal rate of FC (Fig 3.8 A). Faecal coliform counts were initially very high (weeks 10-13), but then gradually decreased. From week 19-43, the FC count showed a good level of consistency (<1000). The FC data points between the IAPS and the SSF were significant (t-test: p= 0.04). Total coliforms in the SSF were reduced by 61% from the IAPS final effluent (Fig 3.8 B). The TC counts seemed to elevate during the initial testing stages (weeks 6-9), but then gradually decreased. There was no significant difference between the IAPS and SSF (t-test: p= 0.08). The regression analyses also indicated less variance among the data points (r^2= 0.09; v= 7.45E09; p= 0.02).

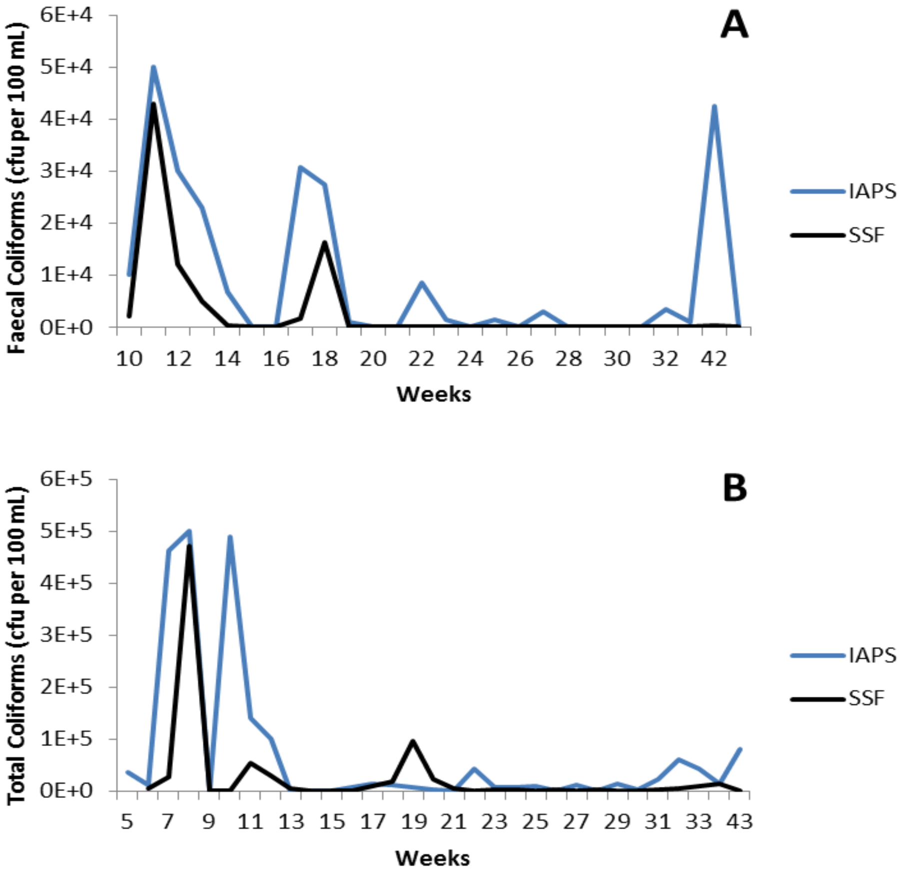

Figure 3.8: Comparison of the different coliform counts between the slow sand filters and the IAPS at the point of discharge. A) faecal coliforms (FC), B) total coliforms (TC). Composite samples were analysed weekly over a 24 h period. The FC was sampled over a period of 9 months while the TC was sampled over 10 months. Data presented are the average of triplicate measurements.

3.2.3 Rock filtration as a tertiary treatment unit

The rock filtration system in series proved to be a successful tertiary treatment with regard to nutrient and pathogen removal. Ammonium-N, phosphate-P and nitrate-N concentrations were reduced significantly in the RF's final effluent. The use of a rock filter system resulted in the ammonium-N levels having a 90% removal rate from the IAPS final effluent (0.32 ± 0.3) (Fig 3.9 A). Data using a 1 way ANOVA revealed that the variance was minimal (Appendix A) (r^2= 0.1; v= 0.07; p= 8.06E-14). Results show that the rock filters were effective in reducing phosphate-P levels (Fig 3.9 B). The phosphate-P data for the RF was averaged at (1.75 ± 1.4) which represents a 79% removal rate from the IAPS final effluent. However, with regard to the 1 way ANOVA test, the variance was large (r^2= 0.8; v= 1.99; p= 8.06E-10). Furthermore, the nitrate-N levels were within the limited range (6.59 ± 4.6) (Fig 3.9 C). Data from the regression analyses indicated a small variance (r^2= 0.06; v= 20.98; p=

0.09). The RF's final effluent had a 12% reduction of nitrate-N and according to the t-test, there was no significance difference between the IAPS and RF (t-test: p= 0.3).

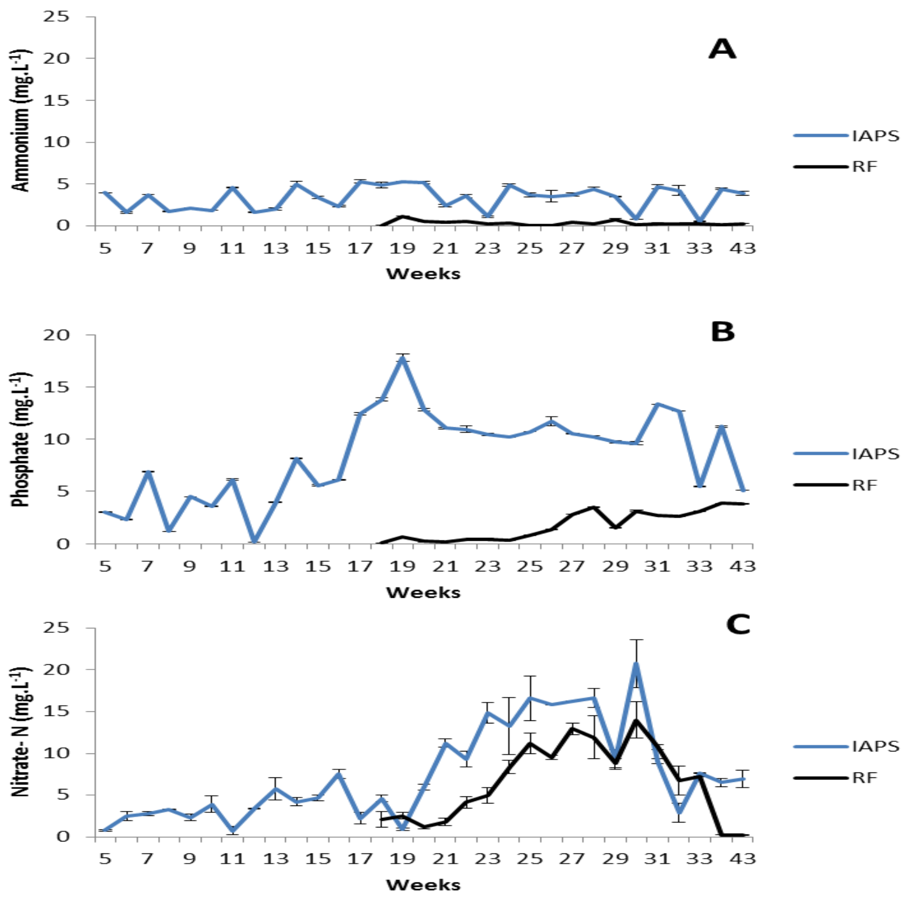

Figure 3.9: Comparison of nutrient content and composition of treated water between the rock filters and IAPS at the point of discharge. A) Ammonium-N, B) Phosphate, and C) Nitrate-N concentrations in composite samples harvested weekly over a 24 h period for 10 months were determined using testing kits as described in the Chapter 2. Data presented are the average of triplicate measurements.

The RF had a 33% removal rate of $COD_{filtered}$ with an average of 63.19 ± 13.2 (Fig 3.10 A). According to the regression analyses of data using one way ANOVA test from February to November 2013, it revealed that the variance was minimal (r^2= 0.0008; v= 171.73; p= 3.11E-12). The TSS levels from the RF were routinely above the required limit of 25 mg.L^{-1} (Fig 3.10 B). The RF had the highest TSS levels on average compared to the other TTU systems (61.0 ± 41.8). Minimal variance was obtained using the 1 way ANOVA test (r^2= 0.002; v= 1744.588; p= 0.13). The t-test also revealed that the data was not significant (t-test: p= 0.48).

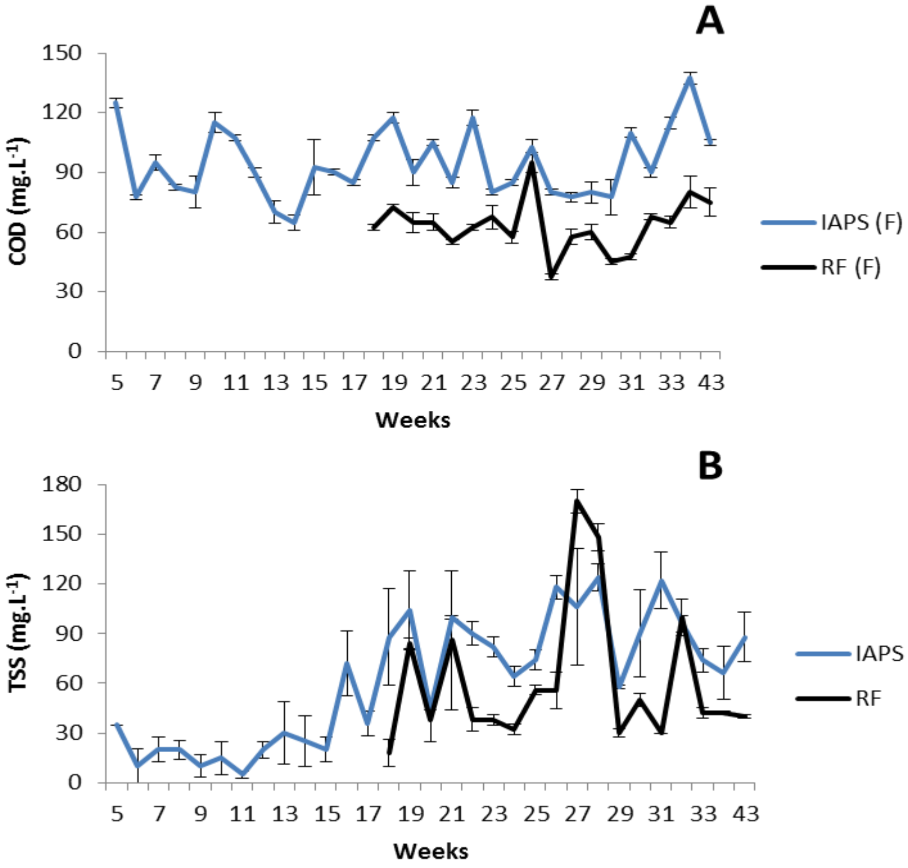

Figure 3.10: Comparison of the chemical oxygen demand and total suspended solids between the rock filters and IAPS at the point of discharge. A) chemical oxygen demand ($COD_{filtered}$), B) total suspended solids were determined from composite samples taken weekly over a 24 h for 10 months. Data presented are the average of triplicate measurements.

Results show that the pH of the RF was within the recommended range (Fig 3.11 A). The RF's had a mean value of 8.56 ± 0.7 mg. L^{-1} and therefore the data was not significant (t-test: p= 0.18). High DO levels were monitored in the RF systems final effluent (12.34 ± 1.8) (Fig 3.11 B). Less variance within the data was attained through the 1 way ANOVA test (r^2= 0.04; v= 3.16; p= 1.95E-12). The electrical conductivity (104 ± 11.5) from the RF was within the limits (70-150 mS.m^{-1}) (Fig 3.11 C). Data between the IAPS and the RF were significant (t-test: p= 3.01E-5).

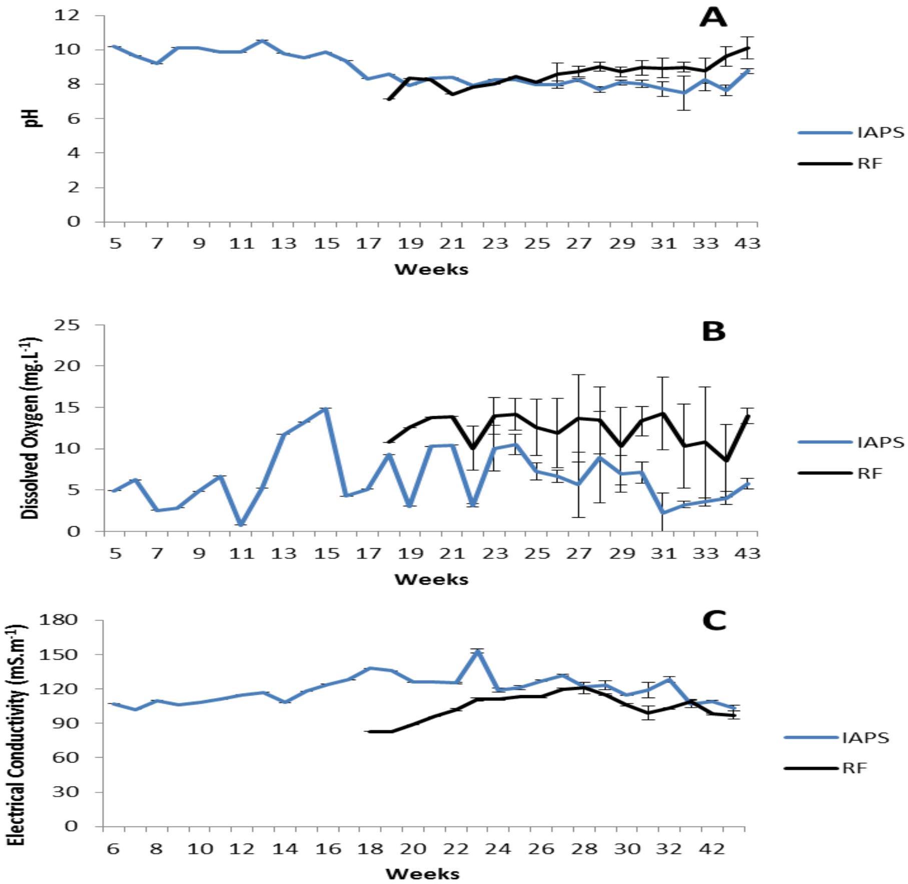

Figure 3.11: Comparison of the physiochemical characteristics of treated water between the rock filters and IAPS at point of discharge. A) pH, B) Dissolved oxygen (DO) and C) electrical conductivity (EC) were determined for composite samples collected weekly for 24 h over a period of 10 months. Data presented are the average of triplicate measurements.

Faecal coliform count in the RF's final effluent showed a 98% reduction therefore making all the data significant (p=0.002) (Fig 3.12 A). From week 19-43, the RF's FC count showed a good consistency of being within the required limits (<1000). The RF had reduced the TC count by 97% from the IAPS final effluent (Fig 3.12 B). Regression analyses of data using a 1 way ANOVA revealed that the variance was minimal (r^2= 0.1; v= 9168897; p= 0.02). Data between the IAPS and the RF was also significant (t-test: p= 0.008).

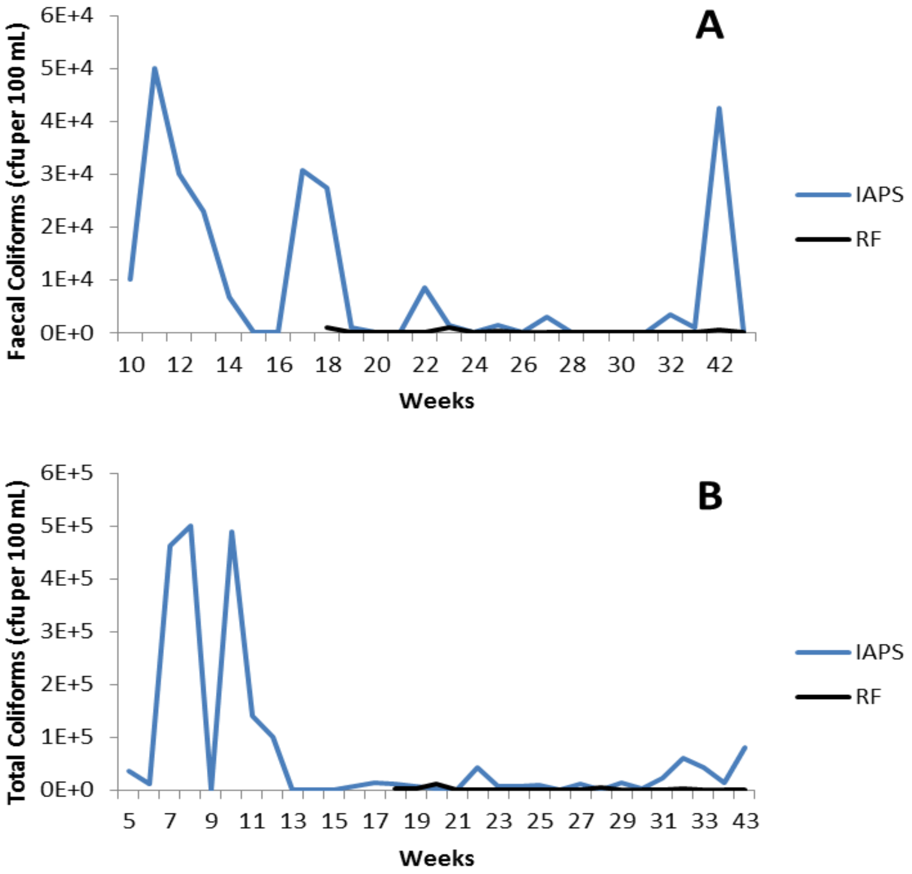

Figure 3.12: Comparison of the different coliform counts between the rock filters and IAPS at the point of discharge. A) faecal coliforms (FC), B) total coliforms (TC). Composite samples were analysed weekly over a 24 h period. The FC was sampled over a period of 9 months while the TC was sampled over 10 months. Data presented are the average of triplicate measurements.

Table 3.2 presents the mean values for all the data collected (IAPS, MP, SSF, RF) from February 2013 to November 2013. This data determines the quality of the final effluent from each system at the Belmont Valley WWTW and then compared to DWA (2010) General Authorization limits for discharge into the environment.

Table 3.2: Summary of the water quality of IAPS and the tertiary treatment units; Maturation Ponds (MP), Slow Sand Filtration (SSF) and Rock Filters (RF). DWA's standard limit for discharge into the environment (DWA, 2010) is also shown. Data was collected on a weekly basis over the course of 10 months.

Parameters	DWA standard	IAPS	MP	SSF	RF
Ammonium - N (mg. L^{-1})	**3**	3.34 ± 1.4	1.08 ± 1.2	2.02 ± 1.3	0.32 ± 0.3
Phosphate (mg. L^{-1})	**10**	8.43 ± 4.2	6.97 ± 5.4	2.57 ± 2.5	1.75 ± 1.4
Nitrate - N (mg. L^{-1})	**15**	7.50 ± 5.6	4.50 ± 3.0	6.80 ± 5.5	6.59 ± 4.6
COD (mg. L^{-1})	**75**	94.76 ± 17.5	78.25 ± 22.7	58.67 ± 13.0	63.19 ±13.2
pH	**5.5- 9.5**	8.78 ± 0.9	9.55 ± 0.8	8.29 ± 0.7	8.56 ± 0.7
Faecal Coliforms (cfu per 100 ml)	**1000**				
Total Coliforms (cfu per 100 ml)					
Dissolved Oxygen (mg. L^{-1})	**>2**	6.48 ± 3.5	11.7 ± 4.6	6.07 ± 2.5	12.34 ± 1.8
Total Suspended Solids (mg. L^{-1})	**25**	61.48 ± 37.7	45.40 ± 31.5	44.17 ± 34.8	61.0 ± 41.8
Electrical Conductivity (mS.m^{-1})	**70 - 150**	119 ± 11.6	117 ± 4.8	118 ± 11.8	104 ± 11.5

3.3 Discussion

In this chapter, the data reflect on how tertiary treatment systems have an effect on water quality after passage of domestic sewage through an IAPS designed and operated for the treatment of municipal wastewater. Several sources have suggested that the IAPS does not yield a final effluent that is acceptable for discharging into the environment (Rose *et al.*, 2007; Meiring and Oellermann, 1995). Tertiary treatment units, which included MP's, SSF's and RF's were incorporated into the IAPS design to determine the effect of these on the water quality of the effluent. Each system was designed and built accordingly to suit the required loading and flow rates. Once the systems were equilibrated (about four weeks), water samples were collected from a sampling point after each TTU, and prior to discharge, to determine water quality. Nutrient removal by the various TTU systems was shown to be effective in the present study and for the most part, the concentration of ammonium-N, nitrate-N, and phosphate-P were within the range permissible for discharge to a water course. By comparison, while some COD was removed by SSF and RF, none of the TTU's used was able to reduce TSS.

3.3.1 Maturation Pond

The MP showed a good level of ammonium-N reduction (68%) throughout the testing period (Fig 3.1 A). According to Pearson (2005), low organic loading and high pH in MP's contribute to a high ammonia removal rates. Craggs (2005) illustrated that a MP added to a WSP can improve the removal rate of phosphorus. He also suggested that through algal biomass assimilation and the increase in pond pH, nutrient removal is improved and that phosphorus through precipitation is stimulated (Appendix B).

Nitrate-N levels in the water emerging from the MP's were the lowest compared to the other tertiary treatment systems, with a 40% removal rate (Fig 3.1 C). MP's are known to decompose organic matter, where algae are present and therefore absorb the nitrogen and phosphorus as well as increase the pH levels (Camargo Valero *et al.,* 2009; Shilton *et al.,* 2012). Camargo Valero *et al.* (2010) found that nitrification-denitrification as well as algal uptake of nitrogen were considered the 2 main functions for nitrogen removal in MP's.

The MP's had a very low COD removal rate (17% reduction). El-Deeb Ghazy *et al.* (2008) also showed a weak removal of COD using a WSP system with MP's in series. They illustrated that poor pond design may be to blame e.g. one point entrances into the MP systems which causes poor mixing and circulation with the pond microorganisms. Algae can increase the COD of the water due to excretion of small organic molecules produced by photosynthesis and general metabolism (Wang *et al.,* 2010). However the MP's did have an effect on COD removal. Since the MP's are an open system, the wastewater is exposed to direct sunlight. MP's use UV radiation as a natural disinfectant and a study has shown that UV radiation has the ability to reduce the COD concentration of industrial wastewater to 60 – 70 % (Craggs, 2005; Chen *et al.,* 1997). MP's are known to develop large algal blooms (Van Vuuren, Van Duuren, 1965), which determines why the COD levels increased in the MP. It also illustrates why the MP's TSS count was above DWA's statutory limit, with only a 26% removal rate from the IAPS final effluent (45.40 ± 31.5) (Fig 3.2 B).

The MP's pH is high during the summer months and then gradually decreases as the winter months approach (Fig 3.3 A). Like the IAPS systems, the MP is an open system and therefore exposed to the environmental variables e.g. direct sunlight. According to Pearson (2003), FC depletion and pH are higher in MP's than facultative ponds.

Maturation ponds have high DO levels compared to the other TTU effluents due to the high rate of photosynthesis from the algae (Fig 3.3 B). The less turbid the MP, the more sunlight penetration therefore the deeper the photosynthetic activity can extend down into the MP, thus providing more oxygen throughout the pond (Pearson, 2005). Electrical conductivity levels were low for environmental discharge (70-150 $mS.m^{-1}$) in the MP, indicating that there are minimal total dissolved solids (TDS) i.e. salt in the water (Fig 3.3 C) (McCleskey *et al.*, 2012).

The MP's main objective is for the removal of pathogens (Egwuonwu *et al.*, 2014). The MP's FC and TC counts have been reduced by 92% and 94% respectively from the final effluent of the IAPS and therefore met DWA's standards for environmental discharge (Fig 3.4 A & B). A series of a few MP's rather than a large individual MP with a long retention time (12 d) and of shallow depth (1-3 m) is the better solution for pathogen removal (von Sperling, 2007; Mara, 2005).

3.3.2 Slow Sand Filtration

Ammonia-N levels in water from the SSF were low compared to the effluent from the IAPS (40% reduction) (Fig 3.5 A). This may be caused by the schmutzdecke layer which is capable of oxidizing ammoniacal nitrogen into nitrates (Hiremath, 2011). Substantial phosphate removal was achieved by the SSF (70%) and this is mainly caused by temperature, redox potential, soil moisture tension (reduces phosphate diffusion) and pH (Fig 3.5 B) (Tofflemire, Chen, 1977). The nitrate-N had a 9% removal rate in the SSF (Fig 3.5 C). According to Aslan (2008), SSF's have the capability of removing nitrate-N. He found that the SSF decreased nitrate-N levels from 22.6 mg. L^{-1} to below the detection limit. Although removal of nitrates did occur, very little was removed throughout the 10 month trial compared to the other TTU's. A reason for this may be due to the sand (river sand) still containing absorbed fertilizer from agricultural runoff, which leads to an increase of nitrate concentrations (Gormly and Spalding, 1979). The increase in nitrate – N levels also corresponds to the increase in TSS (Fig 3.6 B) (weeks 22-32). Nutrients such as nitrate-N are known to cause proliferation of large quantities of algae (Fried *et al.*, 2003). But this contradicts the perception that SSF's are known to decrease total suspend solids by 70-90% (Ellis, 1987; Langenbach *et al.*, 2009; Tyagi *et al.*, 2009).

The $COD_{filtered}$ levels were reduced by SSF by 38% and this is because of its physical and biological processes (Fig 3.6 A). The COD adheres to the tightly spaced sand particles (1-2 mm) when wastewater is filtered through allowing the organic compound to be trapped (Lwesya and Li, 2010). The schmutzdecke also plays a vital role in absorbing pollutant causing substances such as COD (Xiangsheng *et al.*, 2010). It is known as a surface biological mat which consists of algae, bacteria, protozoa, rotifera as well as other microorganisms. The schmutzdecke has the ability to breakdown organic compounds in the wastewater and disallow these compounds from entering further into the filter bed. It has been reported that 75-80% of COD is removed in the first 25-30 cm of sand, which illustrates the fundamental impact of the schmutzdecke on nutrient removal (Lwesya and Li, 2010; McNair *et al.,* 1987). There was a 28% reduction in TSS levels in the SSF (Fig 3.6 B). Processes such as “biological degradation” and “physical straining” are known to remove TSS and organic matter when wastewater is passed through the system (Middlebrooks *et al.,* 2005). However the TSS levels were still above DWA’s discharge limit (44.17 ± 34.8). The SSF’s particle size at EBRU may have tended to be too large (gravel= 15 – 22 mm; fine sand= 1-2 mm). To get an effective removal of algae (< 30 mg. L^{-1}), an effective sand particle size should be at least 0.3 mm (Middlebrooks *et al.,* 2005). Spellman (2008) also recommended a particle size of 0.25-0.35 mm for an SSF to be effective.

The pH of water emerging from the SSF remained within the recommended limit due to biological activity and CO_2 conversion which occurs within the filter (Fig 3.7 A). This then decreases the pH levels in the SSF (Rooklidge and Ketchum, 2002). The DO levels were the lowest in effluent from the SSF compared to the other TTU’s but this may be due to the overgrowth of microorganisms found within the schmutzdecke which use DO for growth and produce a nitrifying environment, hence the high nitrate levels in the SSF (Fig 3.7 B) (Burlage, 2011). However, the DO levels still complied with the General Limit values because the SSF causes a DO gradient in the supernatant depth, when oxygen gets depleted. This allows oxygen to diffuse from the air into the water and therefore assists oxygen transportation to the biolayers, providing aerobic condition so that bacteria can survive (Collin, 2009). Electrical conductivity levels were within range and therefore complied with DWA’s limits for environmental discharge (Fig 3.7 C).

The SSF’s schmutzdecke plays a vital with regard to removing parasite cysts and oocysts (Fox *et al.,* 2011). However the SSF as a post-treatment to the IAPS only attributed a 66% reduction of FC and 61% reduction for TC which is significantly less than the other 2 post-treatment systems (Fig 3.8 A & B). But this is only because of the increased spikes in both the FC and TC which had an effect on the results. The sudden spikes may have been caused by the changing of the schmutzdecke (Appendix B), giving the biological layer minimal growth and therefore decreases pathogen removal. Schijven *et al.* (2013) found that, although SSF’s can remove pathogenic microorganisms, they do however have complications in removal efficiency due to “variable operational condition” and “detection limits”.

3.3.3 Rock Filtration

Water from the rock filters (RF) contained ammonium-N which was perpetually under DWA's recommended standard throughout the testing period (Fig 3.9 A). The RF had a 90% removal rate. Middlebrooks *et al.* (2005) found that RF's are generally known to be anoxic and therefore produces very little nitrification but due to this rock system being slightly aerated at times (due to the reasonably high DO levels) it provides a more significant reduction in ammonia-N (Crites *et al.,* 2014). The RF's had the highest phosphorus removal rate (79%) in comparison to the MP and the SSF (Fig 3.9 B). Hamdan (2010) stated the RF as being an auspicious technology for wastewater treatment; however she found that phosphorus removal was very limited. This is a contradiction of the results obtained as the average measurement for the phosphate levels from the RF's was 1.75 mg. L^{-1} (Appendix B), which is notably low compared to DWA's recommended standard (10 mg. L^{-1}). The nitrate-N levels, although in compliance throughout the testing period, only had a 12% removal of nitrates. This may be due to RF's having aerobic conditions, which allows nitrification to occur. The RF system had high levels of TSS (algae) and algae are also known to remove nitrates by using it for growth purposes (Fig 3.10 B) (Abdel-Raouf *et al.*, 2012).

The RF's can be seen as an effective technology with regard to COD reduction (33% removal) (Fig 3.10 A). Al-Sa' ed *et al.* (2011) did a design comparison using algae-rock-filter ponds (ARP's) and algae-based ponds (ABP's) in parallel and found that the ARP were more successful in removing organic matter (TSS, FC and COD) than the ABP's. The results of the RF's high TSS levels contradict the results found in other research, because RF's are constructed to remove algae (Liu, 2008). Middlebrooks *et al.* (2005) stated that RF's had been designed as a TTU system after an aerated facultative pond in Paeroa, New Zealand and had effectively removed large quantities of algal biomass. These RF's constantly removed TSS to < 25 mg. L^{-1} with averages of < 12 mg. L^{-1} even when the AFP TSS levels were > 100 mg. L^{-1}. According to Hamdan and Mara (2009), aeration is needed to improve TSS removal. Hamdan and Mara (2009) had found that aerated RF's had a better performance base than a subsurface horizontal-flow constructed WL. The possible malfunctioning of the IAPS, which prevented water flow into the TTU's, may have allowed the water to stagnate and therefore facilitated an increase in algae biomass.

The pH levels of water from the RF remained fairly constant (Fig 3.11 A). This may have been due to the minimal amount of calcium carbonate (lime) in the gravel rock which is known to increase the pH to more alkaline conditions (Barrett *et al.,* 2008). RF's can also be regarded as constructed WL's, only without plant matter and WL's are known to lower pH levels, which is a possible reason why the levels are within range (Davies-Colley, 2005). Like the MP, the RF is an open system and therefore exposed to the atmospheric, explaining why DO levels increased at times (Fig 3.11 B). DO levels were only tested in the winter/spring months and the reasonably high DO levels may have been caused by the low temperatures (Appendix B). Electrical conductivity was within the General Limits but although within range, a good quality deionized water is between 0-70 $mS.m^{-1}$ (Fig 3.11 C). What is seen from the results is that the EC levels are higher than this, which leads to alkalinity and salinity problems (DWAF, 1996; Gunduz *et al.,* 2007).

RF's are known to be a low cost operational system, which can be made from locally obtained materials, especially the rock media. These rocks have the capability to support formation of a biofilm layer on the media which capture pathogens (Davies-Colley, 2005). The RF system as a post-treatment to the IAPS showed excellent results for pathogen removal (Fig 3.12 A & B). Saidam *et al.* (1995) used RF's as a post-treatment to WSP's in Jordan and accomplished a FC removal of 94%, where the RF had a FC and TC of 98% and 97% respectively. Middlebrooks *et al.* (2005) state that Williamson and Swanson (1978), had constructed the Veneta system which treats up to 1000 $m^3.d^{-1}$ and found that after 20 operational years, it still produced an effluent of secondary standard for FC counts.

3.4 Malfunction of the IAPS

Malfunction of the IAPS was unfortunately a common occurrence during the testing period (Appendix B). The paddle wheels presented the most problems and were often stopped for long periods of time especially from weeks 18-37 to repair gears, motors and drive-shafts, where either one HRAOP functioned and not the other or vice versa. Due to malfunctioning problems, testing was postponed during weeks 34-41, so that the IAPS's 30 d retention time could be normalised throughout the configuration. Power outages also played a role in operation of the IAPS but these cuts may have only lasted for one or two days which would still be expected to have a significant effect on water treatment and effluent quality.

The pump from the SB to the HRAOP B was not placed in the correct locality as it was supposed to from week 5 to week 16 (Fig 2.1 in Chapter 2). This could have impacted the results of the analysed water. The pump was placed in such a way, that the effluent from the

SB was not getting the full retention time (2-4 d) because the water had not done a full circulation around the HRAOP system.

Changing of the SSF may have also altered the water quality of this TTU's final effluent. The SSF produces a biological layer on the surface of the filter called a schmutzdecke. This layer is able to treat the water by removing microbial organisms more accessibly. But due to the increased density of biological growth, the SSF tends to clog up over time, allowing no influent to filter through. Therefore the SSF needs to be constantly changed over time thus allowing new schmutzdecke to grow. This initially causes a problem since it does take a couple of days for the schmutzdecke to form and therefore eventually have a prominent effect on microbial reduction.

3.5 Conclusion

The post-treatment systems did have a significant effect with regard to the reduction or removal of certain components. On average, the post-treatment systems reduced the ammonium-N, phosphate-P, nitrate-N and COD levels in the IAPS final effluent (Table 3.2). Water quality of effluent from the three TTU's was within the limit of DWA's recommendations except for COD in water from the MP's. Only the SSF's and the RF's COD levels were within the statutory limit. The pH, electrical conductivity and dissolved oxygen were within the specified range for all the TTU's. However the faecal coliform, total coliform and total suspended solid levels were above the standard limit with the exception to the MP and RF faecal coliform count.

CHAPTER 4

The IAPS footprint: in retrospect

4.1 Introduction

The Integrated Algae Pond System (IAPS) located at the Belmont Valley Wastewater Treatment Works is an experimental concept design. Not only was this IAPS built as a research facility but it was commissioned to demonstrate the various components of the system and to test these under South African conditions (Rose *et al.,* 2002b). Based on a flow rate of 75 $m^3.d^{-1}$ derived from 500 person equivalents (P.E.) it was reasoned that this would approximate the minimum size capable of delivering 'credible performance data suitable for engineering scale-up requirements' (Rose *et al.,* 2002b).

The design specifications used for the pilot demonstration IAPS were based on a per capita water consumption of 150 L/d. This value is close to values reported for many developed countries. In Africa by comparison, it is more likely that individuals receive as little as 20 L/d. The United Nations has indicated a minimum water requirement of 50 L/d to avoid diseases and to retain efficiency (Snell, 2014). Data for the Makana Local Municipality, home of the Belmont Valley WWTW, indicates an average local water use per person of 75 L/d. Ignoring losses *en route* to the WWTW it is reasonable to suggest that the current IAPS design configuration might be sufficient to service 1000 P.E.

Therefore this chapter will evaluate the overall impact and benefit of incorporating TTU's into the pilot demonstration IAPS designed to treat 75 $m^3.d^{-1}$ of municipal sewage (for 1000 P.E.), illustrate and discuss the dimensions (size and volume) of each TTU, if 75 $m^3.d^{-1}$ were to flow through these systems and determine the impact that these may convey, in terms of their footprint.

4.2 Results

The current design of the IAPS at EBRU, Belmont Valley, Grahamstown, is for the treatment of wastewater equivalent to 1000 P.E. The design flow into the IAPS was calculated at 75 $m^3.d^{-1}$ which enters a fermentation pit with a volume of 225 m^3. Wastewater then flows to an AFP with a capacity of 1500 m^3 and surface area of 840 m^2. Thereafter wastewater enters the first of two HRAP systems operating in series each with a volume of 150 m^3 and a surface area of 500 m^2. Algae floc (biomass) from each of the two HRAOPs is collected by passive settling in ASP's which contribute a total of 12 m^2. The biomass is then dried using one of four drying beds which contributes a further 20 m^2 to the footprint of the technology components. Thus, the 1000 P.E. IAPS commissioned at the Belmont Valley WWTW requires a land surface area of 1866 m^2. Accounting for walkways and buffer zones between the various components of the IAPS a conservative estimate of the total land requirement is **2 000 m^2** or 0.2 ha per 1000 P.E.

Maturation pond (MP) systems are relatively inexpensive to construct and maintain, require no electricity and are suitable to countries with a low income (Kumar and Goyal, 2009). However, long narrow ponds or multiple smaller ponds (MP system needs to be in a series of 2 or more ponds to improve 'operational flexibility') have a higher land/property cost compared to the other TTU's (SSF and RF) and therefore this TTU may have an economic disadvantage in that it requires more land, particularly when included in the design of an IAPS system (Fig 4.1) (Mara, 2005; Shilton and Sweeney, 2005).

Figure 4.1: A) Maturation Ponds in series, designed by the Department of Hydraulics, Maritime and Environmental Engineering (DEHMA)© UPC, in Verdú - Lleida, Spain. B) Maturation Pond as a tertiary treatment unit in Korba, Tunisia. URL: http://athene.geo.univie.ac.at/pucher/gallery/view_album.php and URL: http://gemma.upc.edu/

According to Luüs (2001), increased urbanisation in South Africa has resulted in an increase in land prices which have risen substantially since 1994. For example, a couple of years ago, the prices in Johannesburg or Cape Town would range on average between R 10 000 and R 20 000 per 1000 m^2, and these have escalated to almost R 500 000 at present (Szymanowski, 2006). Therefore building MP systems in or near urban areas where land prices are exceptionally high would clearly increase the cost of implementing an IAPS for municipal sewage treatment. The simplicity of the system suggests that construction costs would not be excessive but would still be more than the estimated costs of other TTU systems. Labour is usually the most expensive overhead cost in construction. Once the system is built, at least one operator is needed to oversee day-to-day operations and management.

If the ponds require clay as a surface liner (prevent filtration through the soil), then it can be easily extracted from a local source. Even so, it is usual that all ponds associated with WWTW are plastic-lined to increase the longevity of the MP's and to protect ground water from contamination. Adding a plastic liner would cost approximately R 65.00/m^2 (Makgae *et al.,* 2013). PVC piping also contributes to the overall costs of the system and can range from R 100.00 and R 170.00 per 6 m of 110 mm piping (April 2014).

The objective is to determine the area needed for a MP series comprising 3 ponds each with minimum of a 4 d retention time to accommodate a volume of 75 $m^3.d^{-1}$. A three series MP tertiary treatment process with minimum 12 d HRT would need to have capacity equivalent to 12 × 75 m^3 or 900 m^3.Using the following equation:

$$\mathbf{A = Q\ \Theta_{m1}/D}$$

A= Area (m^2)
Q= Influent Flow ($m^3.d^{-1}$)
Θ_{m1}= Retention time (d)
D = Depth

Q = **300 m^2** (Mara, 2005)

The area needed for an additional 2nd and 3rd MP with regard to a 4 d retention time would be exactly the same as the first MP - **300 m^2** each.

Total area: 3 × 300 m^2 = **900 m^2**

According to Mara (2005), each MP in the series should have a length- breadth ratio of approximately 10 to 1 to better improve the plug flow condition within the systems (Brissaud *et al.,* 1998); therefore dimensions for the maturation pond should be ± 60 m × 5 m.

The MP as a post treatment system to the IAPS can be classified as having a fairly large footprint. If one conjoins the areas needed for the IAPS and all the MP's in series, the total surface area needed is **2900 m^2** or 0.3 ha per 1000 P.E. which represents an increase in footprint of 45%.

The slow sand filter (SSF), due to its operation and design simplicity, makes it an appropriate and most affordable technology for use as a TTU system. Most commercial SSF's are rectangular in shape and usually built with concrete or masonry materials to increase the longevity of the system (Fig 4.2) (Huisman and Wood 1974; Scholz, 2006). Using concrete makes the system robust and long lasting. Common materials used are concrete for the floor and brick, stone or mass concrete for the walls. Puddled clay can be used as a waterproof layer. Sloping walls, for an SSF, can be applied as a design concept if the capital cost for building materials is limited; however this design requires a larger area footprint (Huisman and Wood 1974).

One of the major operational costs is to require personnel to clean the system, where cleaning is done on a regular basis. Collins (1999) suggested that a 30 d filter run was optional, and cleaning of the filter should then occur. The cleaning is either done by scraping the top surface of the SSF (schmutzdecke) to prevent clogging of the system or the use of BIDIM® as an extra layer between and on top of the system. For easier cleaning purposes, materials like BIDIM® are recommended. This allows the personnel to remove the top BIDIM® layer, allow the algae to dry and then remove via shaking or scraping. The filter systems should also be covered on top to prevent leaf debris and insects from entering the system (Huisman and Wood, 1974). When integrating an SSF system with an IAPS, the SSF is required to be on a lower ground level than the IAPS to achieve successful passive flow under gravity. Therefore it is suggested to build an SSF system below ground level in order for the system to retain heat, allow easy access to the top of the SSF and provide support for the vertical walls

(Huisman and Wood, 1974).

In order to accommodate a flow rate of 75 $m^3.d^{-1}$, the construction of a SSF with a loading rate of between 0.1 and 0.32 m/h, as recommended by Tyagi *et al.* (2009), or the calculated 0.38 m/h (Chapter 2, Section 2.2.3) equation proposed by McDowall (2008) was used to determine the surface are required.

A = Q/ HLR

A = Area (m^2)
Q = flow rate ($m^3.d^{-1}$)
HLR (hydraulic loading rate) (m/h)

A = 75/ 0.38
A = **± 200 m^2 per SSF**

Total area: 2 × 200 m^2 = **400 m^2**

The dimensions of the SSF are usually rectangular in shape with a 2:1 length: breadth ratio. Therefore the following was calculated:

A = L × B

Therefore: 200 m^2 = 2: 1 length: breadth ratio

200 m^2 = 20 m length: 10 m breadth ratio

20 m length: 10 m breadth

Volume water = L × B × H

= (20) (10) (0.8)
= 160 m^3
= (2) (160)
= **320 m^3 (Both SSF's combined)**

Volume fine sand = L × B × H

$= (20)\ (10)\ (0.5)$

$= 100\ m^3$

$= (2)\ (100)$

= **<u>200 m³ (Both SSF's combined)</u>**

Volume gravel = L × B × H

$= (20)\ (10)\ (0.2)$

$= 40\ m^3$

$= (2)\ (40)$

= **<u>80 m³ (Both SSF's combined)</u>**

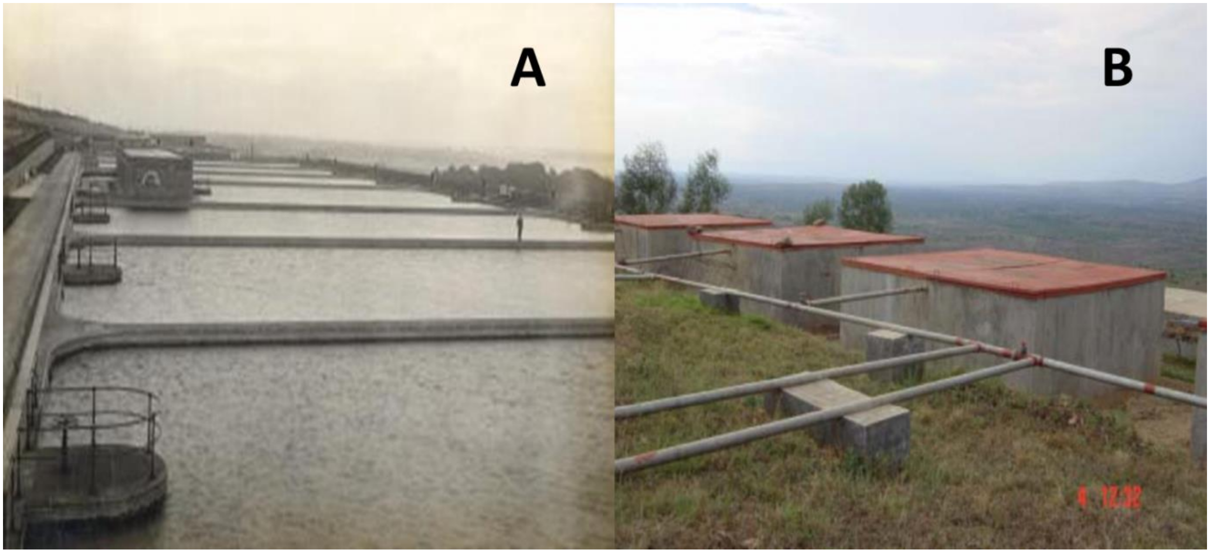

Figure 4.2: Commercial-scale slow sand filters: A) Multiple slow sand filters in Portsmouth, Hampshire, England in 1927. Source: Portsmouth Water (n.y.); B) Recent slow sand filters for a local community at the Nyabwishongwezi Water Treatment Plant, Umatara, Rwanda. Source: Thames Water, University of Surrey (2005). URL: http://www.sswm.info and URL: http://www.portsmouthwater.co.uk/

Therefore according to the results from the equation, the SFF systems need 200 m^2 of surface area each. But if added as a TTU system for the IAPS, then a combined surface area from the IAPS (2000 m^2) and both SSF's (400 m^2) will net to **<u>2400 m^2</u>** or 0.24 ha per 1000 P.E. which represents an increase in footprint of 20%.

Rock Filters (RF) are popular polishing systems and used globally because they are cheap, have no operational complexities and can be easily constructed from local materials (Fig 4.3)

(Davies-Colley, 2005). However land area costs depend on how many RF's are needed in series. To build a single RF system requires less land than the other two polishing systems, but if built in series (3 or more), more area would be required, thereby increasing the footprint of the RF.

One of the biggest advantages of the RF is the relatively low construction costs needed compared to other systems but this is also determined entirely on how many RF's need to be built in series (Middlebrooks *et al.,* 2005). There are more RF systems in series, compared to the SSF, therefore the construction/building costs are higher priced (Appendix B). Construction costs will primarily include personnel to design and build the system, construction materials as well as the mechanical equipment. The rocks needed for the system can be used from a local supply, where the cost would be set around R 800/ton.

Figure 4.3: A) Large-scale indoor rock filtration unit for the treatment of wastewater, City of Yakima, Washington, USA. B) Static trickling filter with wastewater being dispersed over volcanic rock media, using gravity flow in the Republic of Guatemala. URL: http://www.yakimawa.gov/ and URL: http://www.cep.unep.org

The RF requires little maintenance if the hydraulic loading rate is minimal. Infrequent clogging does occur, but it usually happens in the "first third of the RF". If the RF's did present a problem with solids accumulation, then the filter could quite easily be lifted and washed. That is the reason why a system like the RF should include at least two personnel. A RF system in Waiuku, New Zealand was successful in removing large quantities of algae did not have any clogging problems in their 12 year existence (Middlebrooks *et al.,* 2005).

The RF system needs to be constructed in a series to increase the retention time for pathogen removal. Therefore a common series of three with a carrying capacity of 75 $m^3.d^{-1}$ is to be designed. Using the formulae from chapter 2 and substituting for a 75 m^3 volume of wastewater we arrive at the following:

A = Q/ HLR

A = Area
Q = flow rate ($m^3.d^{-1}$)
HLR (hydraulic loading rate) (m/h)

A = 75/ 0.5
A = **<u>150 m^2 per RF</u>**

Therefore the area for all 3 RF's together in series is (150) × (3) = **<u>450 m^2</u>**

Rectangular dimensions are used (most of the commercial RF's are rectangular) to calculate the amount of materials needed to build the system:

A = L × B

Therefore: 150 m^2 = 1.5: 1 length: breadth ratio

150 m^2 = 15 m length: 10 m breadth ratio

<u>15 m length: 10 m breadth</u>

Volume water = L × B × H

= (15) (10) (0.1)
= 15 m^3
= (3) (15)
= **<u>45 m^3 (All RF's combined)</u>**

Volume gravel = L × B × H

= (15) (10) (0.8)
= 120 m^3

= (3) (120)

= **<u>360 m^3 (All RF's combined)</u>**

From the results obtained the RF needs 450 m^2 of surface area, with the addition of the IAPS surface area of 2000 m^2, the surface area required is **<u>2450 m^2</u>** or 0.245 ha per 1000 P.E. from both systems, an increase in footprint of 22.5%.

4.3 Discussion

There is a need to establish an appropriate tertiary treatment unit (TTU) for implementation with IAPS as a commercial technology and which compliments the low cost, environmental aspect of this wastewater treatment system.

The integrated algal pond system (IAPS) at the Belmont Valley WWTW was designed to receive 75 $m^3.d^{-1}$ of wastewater from the Makana Wastewater Municipal Plant. Initially, a MP series, SSF's and RF's as TTU's for the IAPS were designed, and based on flow and loading rates, constructed to receive the required volume of effluent for polishing. However in order for this system to be of commercial use, the TTU systems need to treat an effluent flow rate equivalent to that of the IAPS, hence 75 $m^3.d^{-1}$, thereby increasing the footprint.

The MP as a TTU system was efficient in terms of nutrient and pathogen removal, however it still was not as efficient as the SSF and RF systems (Table 4.1). More land area is also needed for an MP in series, which implicates higher cost for construction as well as increasing the footprint. Material costs were also calculated to be a lot higher than the other systems. Although these factors are inconvenient, labour cost would be the least as the MP's require very few personnel due to the low maintenance requirements (Appendix B).

Slow sand filtration is one of the more feasible option for wastewater treatment due to its low cost of installation, material costs and it capability to adapt to a broad variety of production systems (Garibaldi *et al.*, 2003; Slezak and Sims, 1984).

The SSF requires an area footprint of 2400 m^2, which is less than an MP (2900 m^2) and RF (2450 m^2) (Table 4.1). However, the area required is dependent on how many SSF's are needed in series. In this case, only two SSF's were needed, which is why land and material costs for the SSF were slightly less compared to the RF, which had three in series. If the SSF had added an extra unit, equivalent to the RF, land requirements and cost would be more than the RF's.

Slow sand filter systems entail minimal operating attention and therefore operational/maintenance costs are considered low and a few personnel (± 3 personnel) are needed. But, this system does require more personnel than the other TTU systems because of the maintenance requirements (clogging, cleaning etc.) (Appendix B) (Slezak and Sims, 1984; Spellman, 2008).

In terms of nutrient and pathogen removal, the SSF was more efficient than the MP but not as efficient as the RF. (Table 4.1). In the end, pathogen and nutrient removal play and important role for determining which TTU to construct and even though, the RF was more expensive than the SSF, it still produced a better quality effluent and therefore was a recommended choice (Appendix B).

Rock filtration is the most recommended option in comparison to the MP and SSF. In contrast to the MP, the RF requires less material and land costs but more operational cost due to cleaning of the system. Johnson *et al.* (2007) studied a range of polishing technologies for MP effluent and found that RF's had a "dramatic" cost advantage. Davies-Colley (2005) also mentioned that various kinds of filtering systems which include RF's are the "least expensive polishing option" and provide a smaller footprint in comparison to MP's.

The land area needed for the RF is marginally more (2450 m^2) than that of the SSF (2400 m^2) but this is due to 3 unit system in series, whereby the SSF only has 2 units (Table 4.1). In comparison to the RSF, the RF has similar characteristics (larger grain sizes and more porous space), and therefore a single system does require less land in general compared to a single SSF system because the filtration rate is so much higher (Logsdon and Ratzki, 2007). Halling-Sørensen (1993) suggested that filtration in series increases the depth in which the wastewater is allowed to flow, hence, a longer and therefore a more efficient filtration rate. This explains why the RF had a better quality final effluent compared to the SSF (Table 4.1). Building the RF system would be less expensive (individually), due to its sheer simplicity compared to other systems. However, the material costs of the RF are still more than the SSF (quantity of RF's in series), but once the system has been built, it will rely on minimal maintenance, unlike the SSF (Appendix B).

Table 4.1: The design parameters of the TTU's needed for the polishing the IAPS final effluent for discharge into the environment systems with an influent flow rate of 75 $m^3.d^{-1}$. Percentage removal rate of nutrients and pathogens from the IAPS final effluent of each TTU system.

PARAMETERS	MP	SSF	RF
IAPS	2000	2000	2000
Flow Rate (m^3/day)	75	75	75
No of units	4	3	4
Area needed (combined units) (m^2)	900	400	450
Total area footprint (IAPS included) (m^2)	**2900**	**2400**	**2450**
Nutrient removal efficiency (%)			
Ammonium-N	68	40	90
Phosphate	17	70	79
Nitrate-N	40	9	12
COD	17	38	33
Total suspended solids	26	28	0.8
Pathogen removal efficiency (%)			
Faecal coliforms	92	66	98
Total coliforms	94	61	97

4.4 Conclusion:

Since the post-treatment systems (MP, SSF and RF) play a vital role as an extra polishing step, they do however need to have the capability to support the flow rate of the IAPS effluent, which is 75 $m^3.d^{-1}$ to support 1000 P.E.

A total area footprint of the IAPS and a TTU system has resulted in the SSF having the smallest footprint (2400 m^2), followed by the RF (2450 m^2), then by the MP (2900 m^2). Constructing an MP is the most expensive due to it being the largest system and therefore needing a larger area to build and this will also contribute to its high construction costs. Yet once in operation, very little labour maintenance (the least out of all three systems) or electricity is needed. The SSF would require the least footprint compared to the other TTU systems but would require more maintenance, compared to the other TTU systems, due to constant cleaning of the system. Material costs for the SSF were also the least compared to the other systems. The RF would require a higher material and land cost compared to the SSF but this is all dependent on how many units are needed and the RF system has more units than that of the SSF. However, once the system is functioning, the maintenance costs are lower than the SSF.

In terms of nutrient and pathogen removal efficiency in a final effluent, the MP produced a final effluent of good quality, however it still did not measure the standards that the other TTU's produced and therefore not recommended as a TTU for choice. The SSF produced an effluent of good quality but on average the water quality was not as sufficient as what the RF produced. The RF is the most promising, where all parameters were in compliance with DWA's specific discharge standards with the exception of the TSS levels.
To conclude, the RF is the most suitable for commercial use due to having a small footprint, reasonably low construction costs and less maintenance required.

CHAPTER 5

General Discussion and Conclusion

5.1 Discussion

In South Africa, industrial and sewage pollution pose a major risk to human and environmental health due to the high concentration of toxic organic and inorganic molecules as well as waterborne pathogens. With the majority (80%) of South Africa's wastewater treatment systems not performing according to specification, more "environmentally friendly", "easy to deploy", low maintenance and robust wastewater treatment systems are required.

Integrated Algae Pond Systems (IAPS) are a derivation of the Oswald designed Algal Integrated Wastewater Pond Systems (AIWPS®) and combine the use of anaerobic and aerobic bio-processes to effect wastewater treatment. IAPS technology was introduced to South Africa in 1996 and a pilot plant was designed and commissioned at the Belmont Valley WWTW in Grahamstown. The system has been in continual use since being implemented and reclaims secondarily treated water according to its design specifications. These specifications resemble those of the AIWPS® Advanced Secondary Process developed by Oswald.

While the technology performed well and delivered a final effluent superior to most pond systems deployed in South Africa it was unable to meet The Department of Water Affairs General Standard for nutrient removal and effluent discharge. Multiple pond systems like the IAPS as a domestic wastewater treatment system does not always yield a final effluent acceptable for discharge to the environment. Generally the removal of wastewater organic solids is very efficient, however, the disinfection of the wastewater, algal solids removal and nutrient removal are inconsistent (Craggs *et al.*, 2012).

In fact, a recent report on the operation of hectare-scale high rate algae oxidation ponds (HRAOP) for enhanced wastewater treatment by Craggs *et al.* (2012) strongly advocated additional treatment of the outflow from algae settling pond (ASP) by polishing to meet specific discharge standards. It was recommended that the inclusion of one or a combination of maturation ponds (MP) and UV treatment by storage prior to discharge, or rock filtration

of the MP effluent, or direct UV treatment if insufficient land is available, and if funds are available, membrane filtration to achieve a high quality final effluent for re-use.
Clearly, there is therefore a need to establish an appropriate tertiary treatment unit (TTU) for implementation with IAPS as a technology and which compliments the low cost, environmental aspect of this wastewater treatment system. In addition, any TTU must allow for the final effluent to meet the standards as determined by the Department of Water Affairs (DWA) for discharge to the environment.

Three suitable post-treatment systems were investigated to determine their water polishing efficacy and suitability for incorporation into the IAPS process flow. An MP in series, two SSF's and a three RF's in series were configured in parallel to treat water after secondary treatment using an IAPS. According to Gerba and Pepper (2011), TTU systems are used to improve the microbial quality of a secondary treated wastewater plant so that the effluent can be re-used or discharged into the environment. TTU's are also capable of decreasing the nutrient content of treated water, and removing residual organics and pathogens.

From the results obtained, the RF was the most effective post IAPS treatment than either the MP or SSF in terms of cost efficacy, effluent quality and operational simplicity. It is also one of the best solutions to address wastewater polishing (Hamdan and Mara, 2013).

With regard to land availability, the SSF (400 m^2) required the least amount area footprint, due to having only two units, where the RF (450 m^2) required fairly more, having three units in place. However, in terms of nutrient and coliform removal, the RF was the most efficient, even though these systems only treated a fraction of the total IAPS final effluent. A MP series is an important additional polishing step in conventional wastewater treatment (Mara, 2005). Due to land requirements, and unless a MP series is already in position, SSF and/or RF might be preferable due to the smaller footprint and ease of operation. Therefore it is recommended that either the SSF and /or the RF should be incorporated as part of the design and process flow of the IAPS as a commercial treatment for municipal wastewater.

Nutrient removal was effective for the TTU's tested in the present study. Ammonium-N and phosphate-P were considerably low in the SSF final effluent compared to the final effluent of the IAPS and well below the DWA recommendations for discharge. Ammonium-N and phosphate-P were reduced 40% and 70% respectively. Nitrate-N however, was only reduced by 9%. This slight decrease may be due to the sand (river sand) containing adsorbed fertilizer from agricultural runoff, which leads to an increase of nitrate concentrations (Gormly and Spalding, 1979). In final effluent from the MP series, ammonium-N, phosphate-P and nitrate-N were reduced 68%, 17% and 40% respectively. MP's are able to achieve these reductions in nutrients as algae present, effectively absorb the nitrogen and phosphorus while ammonia is usually lost through volatilization due to the pH increase (Camargo Valero and Mara, 2007). The RF system is a promising system for nutrient removal in comparison to conventional systems (Hamdan and Mara, 2013). The RF excelled in nutrient removal and resulted as the most efficient TTU, in terms of ammonium-N (90%), phosphate-P (79%) removal, in comparison to the MP and SSF. The nitrate-N levels were still below DWA's recommendations for discharge, however, there was only a 12% reduction, indicating the occurrence of nitrification, where the ammonium-N will decrease, which in effect, increases the nitrate-N levels (Beutel, 2001). Hamdan and Mara (2013) had stated that the removal of nitrogen in RF systems was not effective, due to the system rapidly becoming anoxic throughout the filtration period.

The filtered chemical oxygen demand (COD) in the IAPS final effluent was above DWA's recommendations for discharge. This may be due in part, to the system not being effective at removing suspended algae (Meiring and Oellermann, 1995). Algae can increase the COD of the water due to excretion of small organic molecules produced by photosynthesis (Wang *et al.*, 2010) and metabolism in general, and the characterization of these is the subject of further investigation.

The COD was reduced by all TTU's namely RF's, MP's and SSF's presumably by a combination of physical and biological processes. In SSF, solute adheres to the sand particles when wastewater is filtered forming a biofilm or 'schmutzdecke' consisting of algae, bacteria, protozoa, rotifers as well as other microorganisms (Lwesya and Li, 2010). The schmutzdecke plays a vital role in absorbing pollutants that may contribute to COD by breakdown of these organic compounds preventing ingress into the filter bed (Xiangsheng *et al.*, 2010). It has been reported that 75-80% of COD is removed in the first 25-30 cm of sand, which illustrates the fundamental impact of the 'schmutzdecke' on nutrient removal (Lwesya and Li, 2010; McNair *et al.*, 1987). Tyagi *et al.* (2009) showed that SSF can reduce TSS by up to 89%. The COD in the final effluent of the MP series fluctuated between the DWA limit for discharge of treated wastewater. The increase in TSS (algae), throughout the testing period, may have caused the MP series to only reduce COD by 17%. However, the COD reduction is presumably by a combination of nutrient removal and photooxidation. Ultra violet radiation has for example been shown to reduce the COD of industrial wastewater by as much as 60-70 % (Craggs, 2005; Chen *et al.,* 1997). High TSS levels in the RF contradict to why the COD was within DWA's specified range. The RF only had a 0.8 % removal of TSS and therefore not within standard, whereas the COD had a 33% removal. Concern about the constant malfunctioning, which allowed the wastewater to be stagnant, was a key factor to why the TSS was so high. Total suspended solids (TSS) were reduced for both effluent streams indicating that MP's and SSF were efficient at reducing suspended solids. However, these TSS levels were still not within DWA recommendations for discharge. This may be of concern as high TSS levels deplete the rate of photosynthesis and therefore may have an effect on the biotic communities when the effluent is released back into the environment (DWAF, 1996).

Effluent pH from the MP series did not comply with the DWA standard which is expected as algae in this TTU alkalinize the water (Griffiths, 2010). This aspect aside, the physicochemical properties (pH, DO and EC) of the final effluent from the MP series, SSF and RF complied with General Limit Values throughout the period of analysis.

Faecal coliforms (FC) and total coliforms (TC) were very high in final IAPS effluent between weeks 3 and 5 due largely to operating issues. Nevertheless, there was a significant reduction in FC and TC by these TTU's after necessary reparation and as might be expected. In SSF, no algae or solid material should pass through the filter and typically these systems reduce FC by 99% and Streptococci by 99% (Tyagi *et al.,* 2009).

There are many other types of tertiary and quaternary treatment units which could have been incorporated into the IAPS system as an extra polishing unit. For example: horizontal and vertical flow wetland systems, duckweed systems, chlorination, ozonation and UV radiation (chlorination, ozonation and UV radiation have already been discussed in chapter 1). Wetlands have proved to be an effective technological system for wastewater treatment (Kivaisi, 2001). Constructed wetlands are seen to be low-cost methods in trying to improve the water quality for domestic wastewater, especially in poverty stricken areas. Wetlands are a much cheaper resource (than conventional systems) mainly because it's a natural treatment for water purification and less electricity is consumed (Ko *et al.,* 2004). The duckweed system is a reliable tertiary treatment unit in terms of BOD and TSS removal but these systems require a large land area and the ceasing of duckweed growth seems to occur during the winter months (Bonomo *et al.*, 1997).

Algae play a fundamental role within the IAPS system. The pH in the HRAOP is known to increase due to algal photosynthesis and therefore kill all *E. coli*. The HRAOP system also increases the DO levels caused by photosynthesis of the algae which converts sunlight, carbon dioxide and water into oxygen (Wells, 2005). However, the IAPS and the tertiary treatment units all produced final effluents with high TSS levels (which contributes to the high COD levels). It is unclear to why the levels were so high in the IAPS, but a reason may be due to the constant malfunctioning of the systems, which allows the water to become stagnant. Since one algal settling pond (ASP) is used to decrease the algae content in the IAPS final effluent, maybe multiple ASP's in series could be constructed and observed to see whether the algae content will decrease in the final effluent.

High TSS levels somewhat contradict studies shown in other research in which tertiary treatment units, in particular SSF's and RF's, are known to reduce TSS levels. Filtering media of the SSF and RF may have been too large therefore allowing minute extracellular polysaccharides to filter through each system. Reasons why the MP had such a high TSS levels was probably due to the short-circuiting of the system. The baffle system tends to prevent short-circuiting but very often, there are minor gaps (especially on the sides of the baffle system) which allow minute algae cells to enter and hence, increase the TSS levels in the final effluent. Therefore, these are all issues which need to be considered and somewhat prevented in order to have a system which provides a good quality effluent for environmental discharge.

5.2 Conclusion

The MP's, SSF's and RF's are all effective tertiary treatment systems for the IAPS as a wastewater remediation step. The three post-treatments complied with the majority of the parameters according to DWA's standards for wastewater discharge into the environment. The SSF is seen as a very promising post treatment option and classified as a high class filtrate and thoroughly used in the potable water industry but even though it had the cheapest cost, it still lacked performance with regard to pathogen and TSS removal. The MP's have also proved to be an important additional polishing step to water cleansing (Mara, 2005), however it had the highest footprint and material cost compared to the other systems. With all the results obtained, the RF was seen to be the most promising due to its small footprint, cost efficacy, effluent quality and operational simplicity. Since many countries live in water stressed areas, it must be acknowledged that these countries need to implement more sustainable solutions in treating and recycling water for environmental discharge (Sowers *et al.,* 2011) and the IAPS system with the use of a RF system in series has been found to be reliable and effective in this regard.

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Appendices

Appendix A

Table A1: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Ammonium-N

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	103.5	3.33871	2.052472
MP	30	32.32	1.077333	1.547482
SSF	30	60.74	2.024667	1.647412
RF	18	5.7	0.316667	0.073471

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	129.8462045	3	43.28207	29.23052	8.06E-14	2.691133
Within Groups	155.4750817	105	1.480715			
Total	285.3212862	108				

Table A2: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Phosphate-N

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	262.4335	8.465598	17.35618
MP	30	209.0707	6.969022	29.02414
SSF	30	76.99614	2.566538	6.182083
RF	18	31.55419	1.753011	1.995457

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	844.7843964	3	281.5948	18.76597	8.06E-10	2.691133
Within Groups	1575.5886	105	15.00561			
Total	2420.372996	108				

Table A3: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Nitrate-N

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	232.6034	7.503335	31.18167
MP	30	135.0365	4.501217	8.728177
SSF	30	204.4607	6.815356	30.47889
RF	18	118.6861	6.593674	20.97824

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	151.3241	3	50.44136	2.180386	0.094706	2.691133
Within Groups	2429.085	105	23.13414			
Total	2580.409	108				

Table A4: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

$COD_{filtered}$

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	2940.833	94.86559	300.0045
MP	30	2341.667	78.05556	531.5773
SSF	30	1762.5	58.75	173.9823
RF	18	1135	63.05556	171.732

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	23003.68716	3	7667.896	24.86439	3.11E-12	2.691133
Within Groups	32380.8057	105	308.3886			
Total	55384.49286	108				

Table A5: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

pH

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	272.06	8.776129	0.855458
MP	30	286.4	9.546667	0.702395
SSF	30	248.73	8.291	0.478837
RF	18	154.03	8.557222	0.520904

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	25.53893	3	8.512977	12.99694	2.77E-07	2.691133
Within Groups	68.77483	105	0.654998			
Total	94.31377	108				

Table A6: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Faecal Coliforms

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	26	240200	9238.462	2.18E+08
MP	26	18670	718.0769	2497519
SSF	26	80730	3105	81603490
RF	18	2900	161.1111	113986.9

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	1.26E+09	3	4.21E+08	5.122457	0.00254	2.703594
Within Groups	7.55E+09	92	82101136			
Total	8.81E+09	95				

Table A7: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Total Coliforms

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	2106500	67951.61	2.03E+10
MP	30	125040	4168	80558712
SSF	30	796717	26557.23	7.45E+09
RF	18	38800	2155.556	9168897

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	7.86E+10	3	2.62E+10	3.327887	0.022475	2.691133
Within Groups	8.27E+11	105	7.87E+09			
Total	9.05E+11	108				

Table A8: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Dissolved Oxygen

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
IAPS	31	200.82	6.478065	12.03312
MP	30	351.05	11.70167	20.74302
SSF	30	182.04	6.068	6.257803
RF	18	222.15	12.34167	3.161262

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	869.3285144	3	289.7762	25.40285	1.95E-12	2.691133
Within Groups	1197.759031	105	11.40723			
Total	2067.087545	108				

Table A9: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Total Suspended Solids

SUMMARY

Groups	Count	Sum	Average	Variance
IAPS	31	1906	61.48387	1421.991
MP	30	1362	45.4	995.2828
SSF	30	1325	44.16667	1211.868
RF	18	1098	61	1744.588

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	7389.038	3	2463.013	1.897056	0.134601	2.691133
Within Groups	136325.1	105	1298.334			
Total	143714.1	108				

Table A20: Summary of a 1 way analyses of variance (ANOVA) to test the difference between all the treatment units from February 2013 to November 2013.

Electrical Conductivity

SUMMARY

Groups	Count	Sum	Average	Variance
IAPS	30	3583	119.4333	134.8057
MP	30	3522	117.4	22.73103
SSF	30	3527	117.5667	140.1161
RF	18	1869	103.8333	131.7941

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	3143.816667	3	1047.939	10.02403	7.36E-06	2.691979
Within Groups	10872.43333	104	104.5426			
Total	14016.25	107				

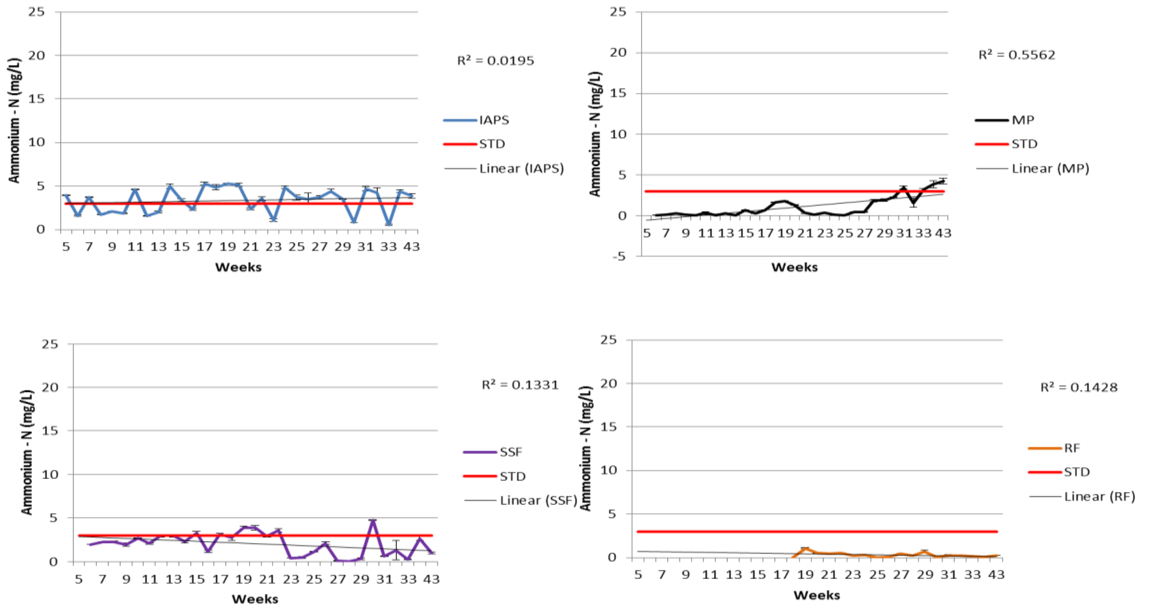

Figure A1: Measurement of Ammonium-N in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

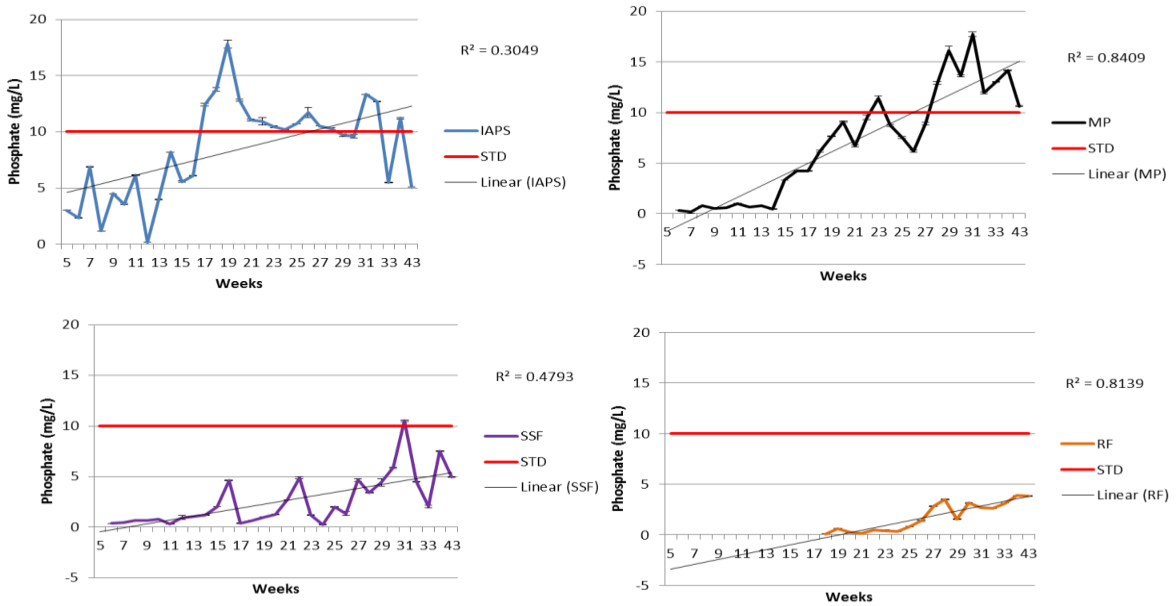

Figure A2: Measurement of Phosphate-P in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

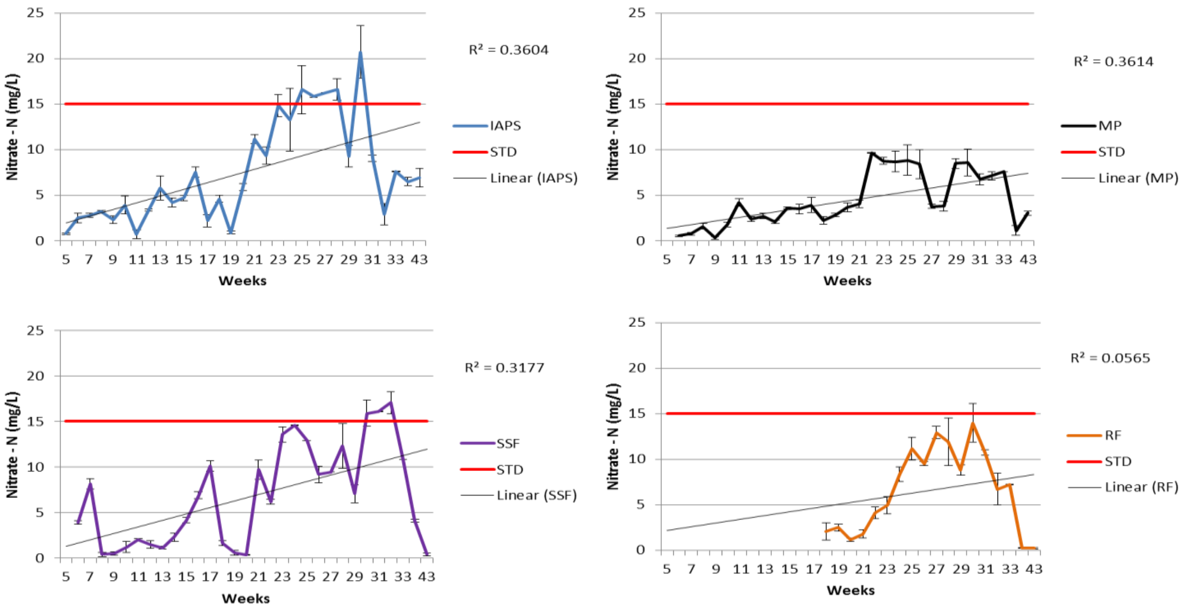

Figure A3: Measurement of Nitrate-N in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

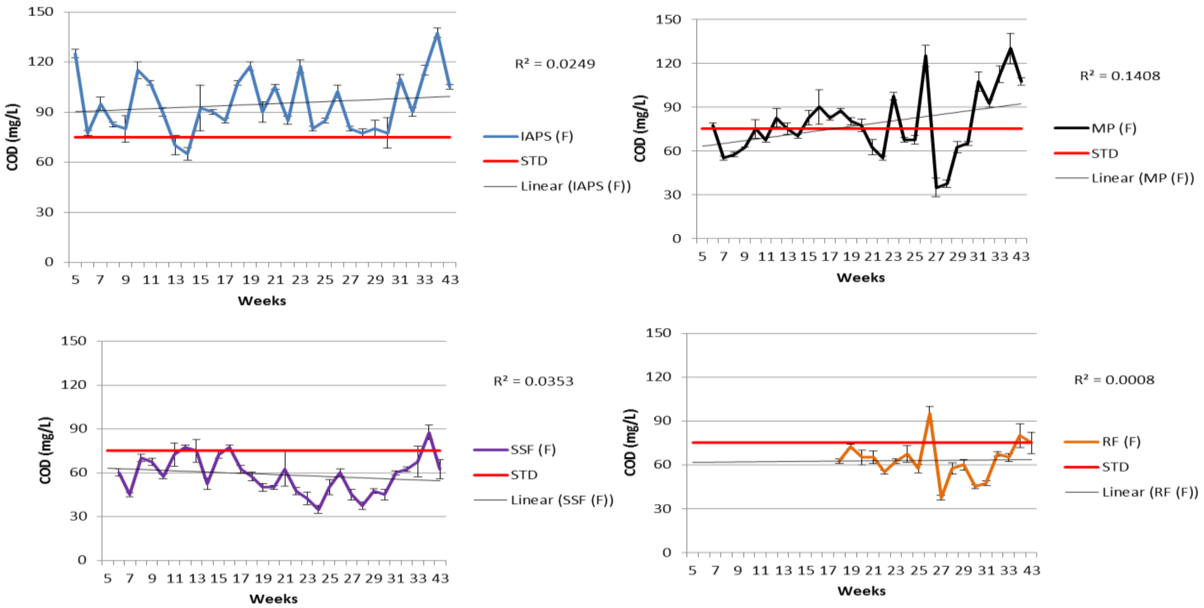

Figure A4: Measurement of COD$_{filtered}$ in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

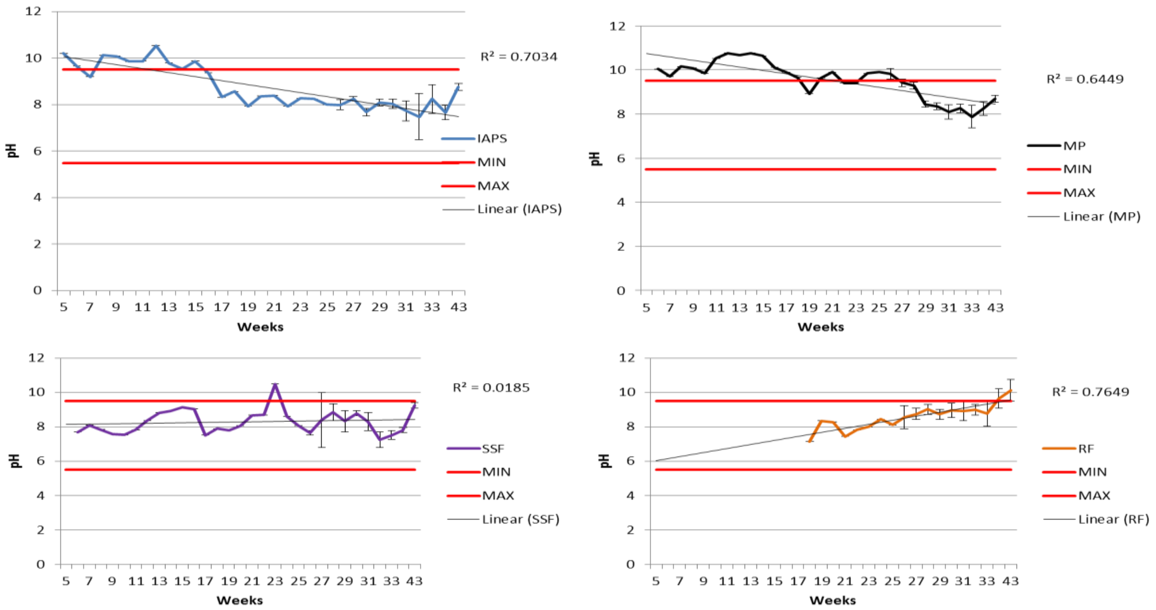

Figure A5: Measurement of pH in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

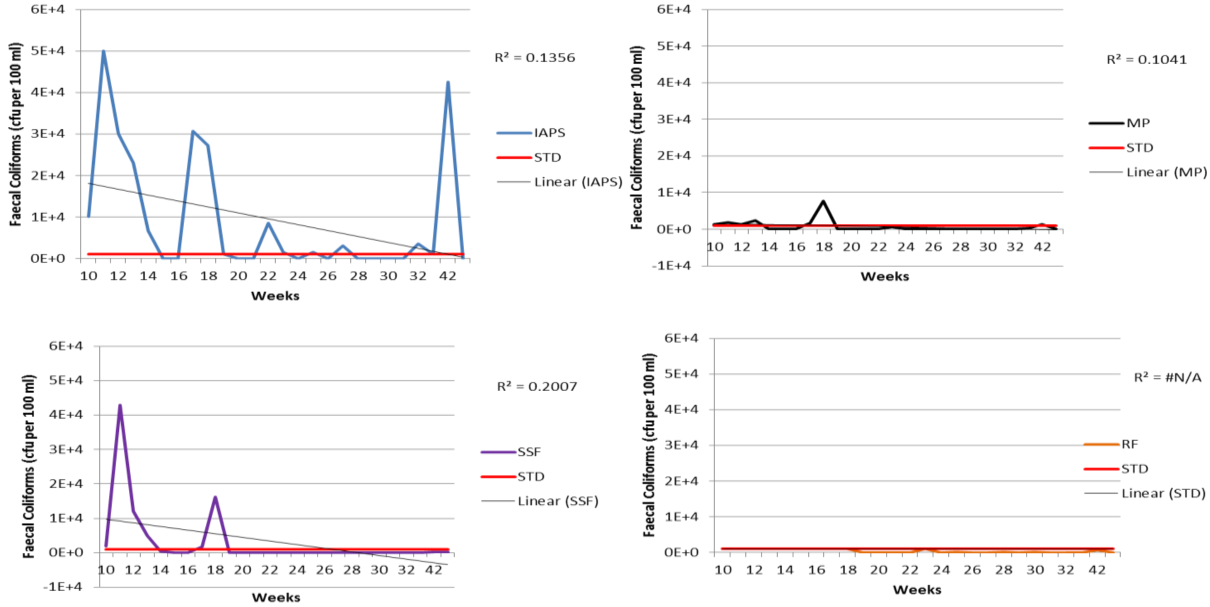

Figure A6: Measurement of Faecal coliforms in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

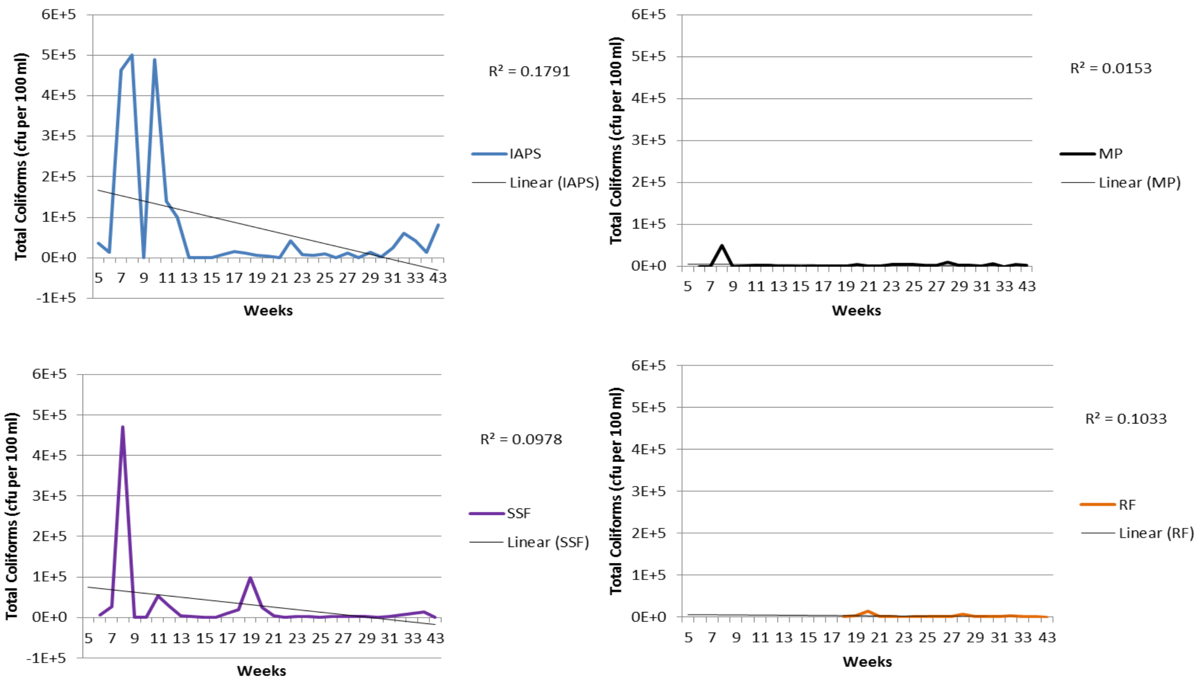

Figure A7: Measurement of Faecal coliforms in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

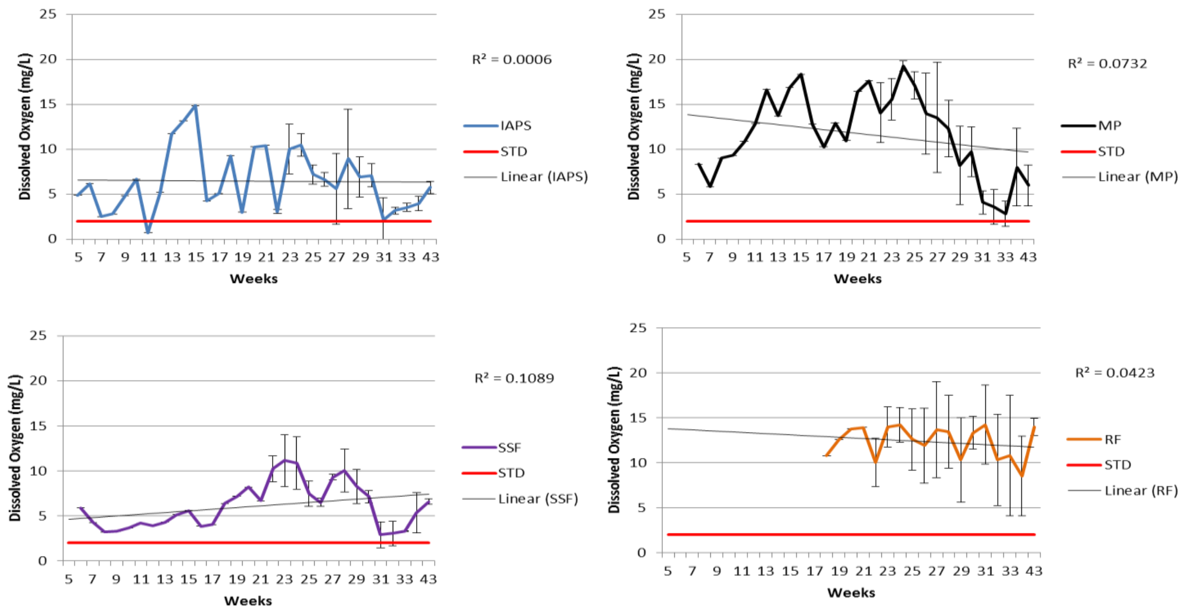

Figure A8: Measurement of Total coliforms in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

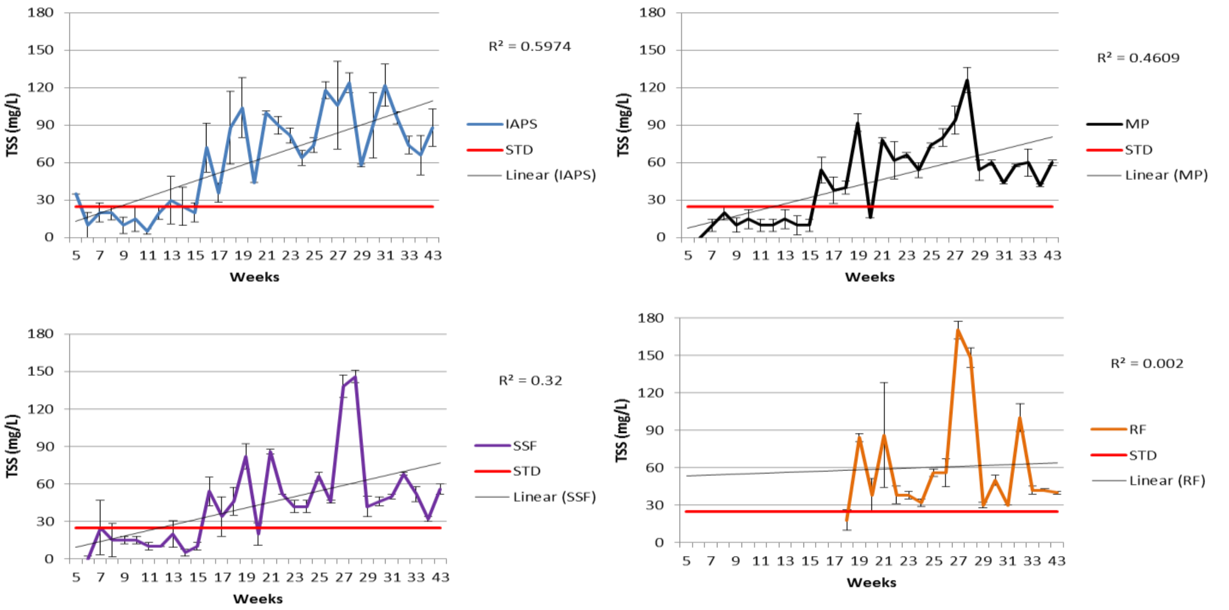

Figure A9: Measurement of Total suspended solids in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

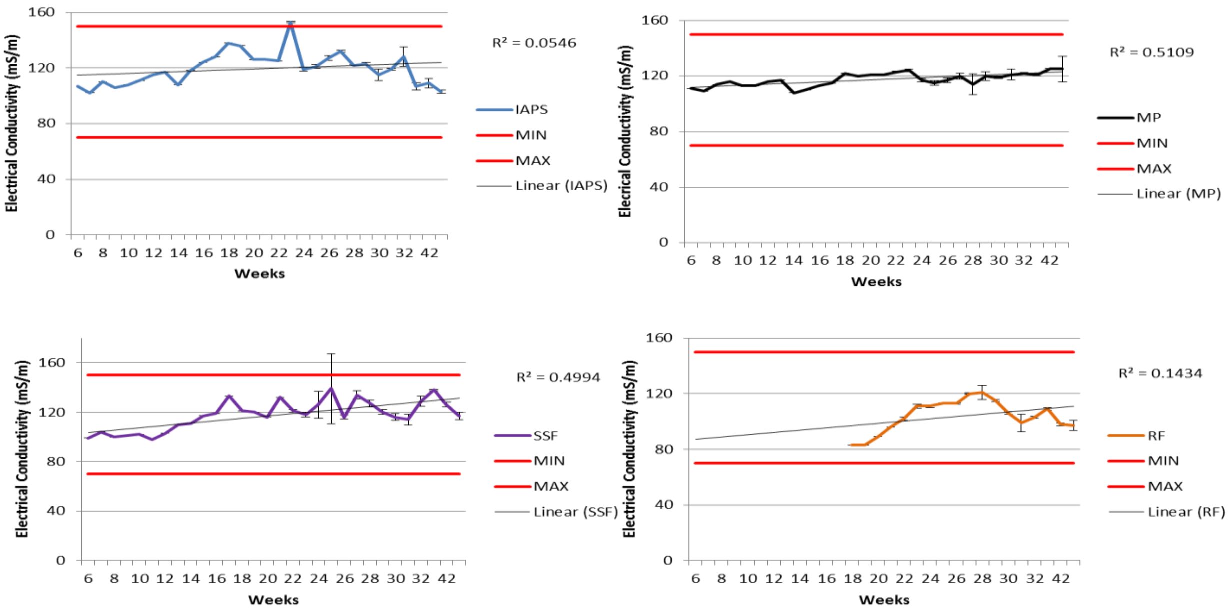

Figure A10: Measurement of Electrical conductivity in the final effluent by the IAPS and the other tertiary treatments using either a maturation pond, slow sand filtration or rock filtration. MP = Maturation Pond Series; SSF = Slow Sand Filtration; RF= Rock Filtration; STD = Standard (DWA). A linear regression line was calculated, which determined the R-squared value.

Appendix B

Table B1: Weather conditions (temperature, precipitation and wind speed) from January 2013 to December 2013. (Weather History of Grahamstown, South Africa, 2013).

	Temperature (°C)			Precipitation (mm)			Wind Speed (km/h)		
	Max	**Mean**	**Min**	**Max**	**Mean**	**Min**	**Max**	**Mean**	**Min**
January	33	**18**	9	6	**0.4**	0	48	**15**	4
February	36	**17**	8	8	**0.5**	0	43	**14**	0
March	36	**18**	9	20	**1.5**	0	41	**14**	0
April	31	**16**	6	5	**0.3**	0	32	**12**	0
May	30	**13**	3	6	**0.4**	0	39	**13**	0
June	25	**11**	1	1	**0.1**	0	54	**18**	0
July	25	**11**	1	3	**0.2**	0	52	**17**	0
August	31	**11**	2	4	**0.4**	0	46	**18**	4
September	30	**13**	3	0	**0**	0	50	**16**	4
October	34	**15**	4	30	**2.7**	0	37	**14**	0
November	36	**17**	8	35	**2.4**	0	41	**16**	0
December	31	**18**	7	6	**0.8**	0	39	**15**	0

Table B2: The various basic costs involved for developing the TTU structures (MP, SSF and RF) for commercial use. The costs include material costs, land costs and labour costs.

	Material cost		Footprint (land) cost		Labour maintenance/operational cost (monthly)		TOTAL
MP	Plastic liner @ R65.00/m^2 (× 2766 m^2) (× 3)	R 539 370	Plot @ R500 000/1000 m^2 (× 2766 m^2) (× 1)		Personnel @ R12.00/hr (× 8 hrs) (× 30 d) (× 1 personnel)		
	PVC piping @ R150.00/6m (× 3)	R 450					
	TOTAL	**R 539 820**		**R 1 383 000**		**R 2 880**	**R 1 925 700**
SSF	Bricks @ R2.50/brick (× 5376 bricks) (× 2)	R 26 880	Plot @ R500 000/1000 m^2 (× 2266 m^2) (× 1)	-	Personnel @ R12.00/hr (× 8 hrs) (× 30 d) (× 3 personnel)	-	-
	Gravel @ R800/ton (× 54 tons) (× 2)	R 86 400		-		-	-
	Sand @ R100.00/ton (× 7 tons) (× 2)	R 1 400		-		-	-
	Cement @ R70.00/50 kg bag (× 27 bags) (× 2)	R 3 780		-		-	-
	Bidim @ R11.00/m^2 (200 m^2) (× 4)	R 8 800		-		-	-
	PVC piping @ R150.00/6m (× 2)	R 300		-		-	-
	TOTAL	**R 127 560**		**R 1 133 000**		**R 8 640**	**R 1 269 200**
RF	Bricks @ R2.50/brick (× 3143 bricks) (× 3)	R 23 573	Plot @ R500 000/1000 m^2 (× 2316 m^2) (× 1)	-	Personnel @ R12.00/hr (× 8 hrs) (× 30 d) (× 2 personnel)	-	-
	Gravel @ R800/ton (× 166 tons) (× 3)	R 398 400		-		-	-
	Sand @ R100.00/ton (× 4 tons) (× 3)	R 1 200		-		-	-
	Cement @ R70.00/50 kg bag (× 16 bags) (× 3)	R 3 360		-		-	-
	PVC piping @ R150.00/6m (× 3)	R 450		-		-	-
	TOTAL	**R 426 983**		**R 1 158 000**		**R 5 760**	**R 1 590 743**

Table B3: IAPS malfunctioning during the testing period as well as municipal electrical cuts and changing of the slow sand filter. Different colour codes represent the different complications or changes of the system from week 5 to week 43 of 2013. HRAOP = high rate algal oxygenated pond.

IAPS Malfunctioning Operations																																							
Week	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43
Misplaced pump to HRAP B																																							
Facultative Pond (anaerobic digestor)																																							
Paddlewheel A																																							
Paddlewheel B																																							
Splitter box																																							
Settling Ponds																																							
Change in SSF																																							
Electricity Cuts																																							

Table B4: Average measurements recorded of each component (Ammonium, Phosphate, Nitrate, COD, pH, Dissolved Oxygen, EC, FC and Total Coliforms) in the different treatment systems over a 43 week period (with the exception of the FC count (33 week period) and the EC (42 week period).

	Ammonium	Phosphate	Nitrate	COD	TSS	pH	Dissolved Oxygen	Electrical Conductivity	Faecal Coliforms	Total Coliforms
Measurement	mg. L^{-1}	mg. L^{-1}	mg. L^{-1}	mg. L^{-1}	mg. L^{-1}		mg. L^{-1}	mS.m^{-1}	cfu per 100 ml	cfu per 100 ml
Standard	**3**	**10**	**15**	**75**	**25**	**5.5 - 7.5**	**>2**	**70 -150**	**1000**	
IAPS	3.34	8.43	7.5	94.8	62	8.78	6.48	119.43	9238	67952
Maturation Pond	1.08	6.97	4.5	78.3	45	9.55	11.7	117.4	718	4168
Slow Sand Filter	2.02	2.57	6.8	58.7	44	8.29	6.07	117.57	3105	26557
Rock Filter	0.32	1.75	6.59	63.2	61	8.56	12.34	103.83	161	2156

Table B5: A table showing different COD wavelength results with regard to the different concentrations. Measurements were done in triplicate and the average was then calculated.

Concentrations	1	2	3	Average
0	0	0	0	0
200	0.071	0.052	0.117	0.08
400	0.16	0.17	0.175	0.168333
600	0.25	0.225	0.226	0.233667
800	0.322	0.32	0.34	0.327333

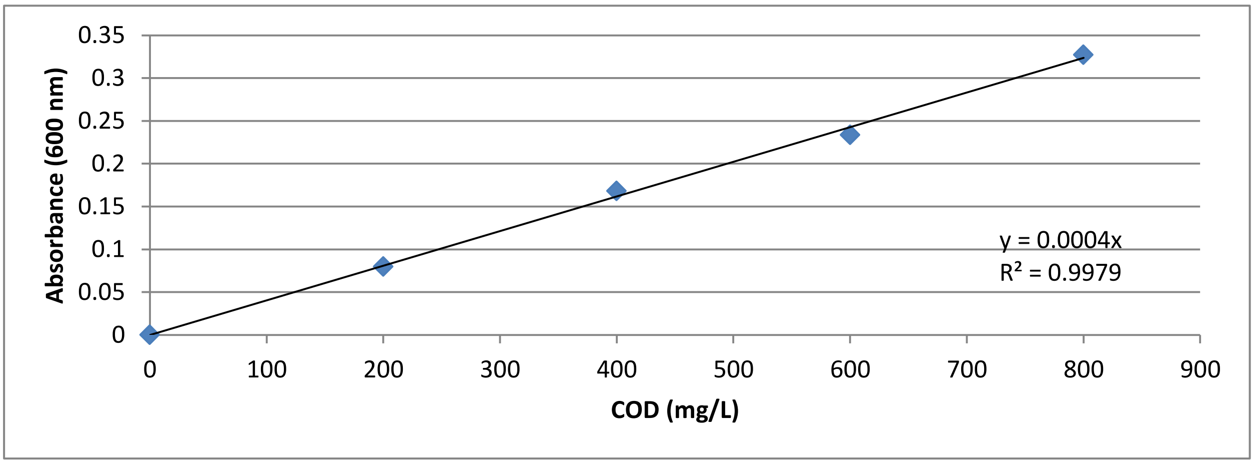

Figure B1: Graph illustrating the increasing concentrations of COD at a wavelength of 600 nm, the curve was used to determine unknown COD concentrations within the samples.

Table B6: A table showing different Phosphate-P wavelength results with regard to the different concentrations. Measurements were done in triplicate and the average was then calculated.

Concentration	1	2	3	Average
0	0	0	0	0
1	0.143	0.144	0.149	0.145333
2	0.299	0.3	0.31	0.303
3	0.505	0.447	0.448	0.466667
4	0.561	0.563	0.564	0.562667

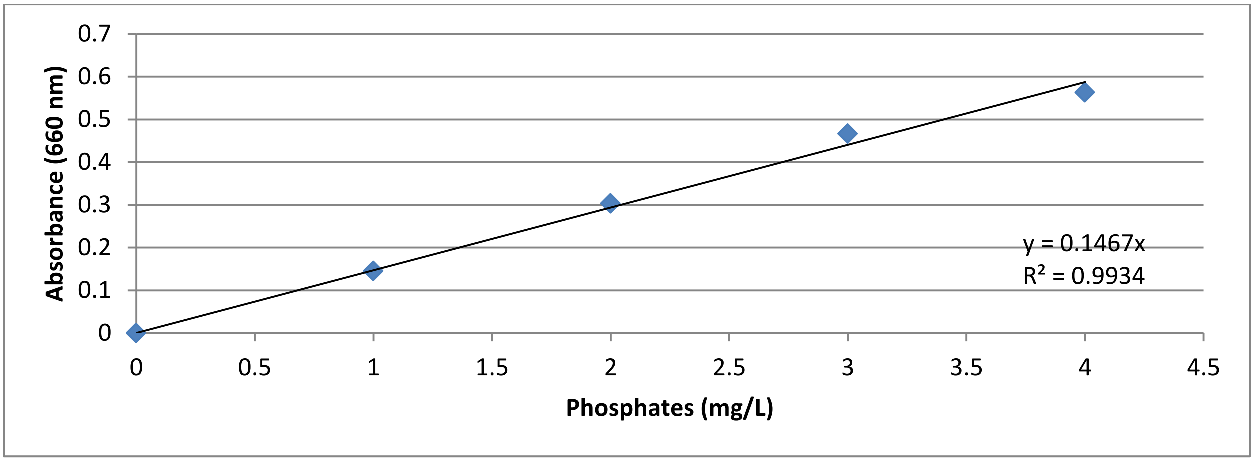

Figure B2: Graph illustrating the increasing concentrations of Phosphate-P at a wavelength of 660 nm, the curve was used to determine unknown Phosphate-P concentrations within the samples.

Table B7: A table showing different Nitrates-N wavelength results with regard to the different concentrations. Measurements were done in triplicate and the average was then calculated.

Concentrations	1	2	3	Average
0	0	0	0	0
5	0.169	0.208	0.191	0.189333
10	0.456	0.468	0.457	0.460333
15	0.564	0.672	0.655	0.630333
20	0.756	0.9	0.715	0.790333

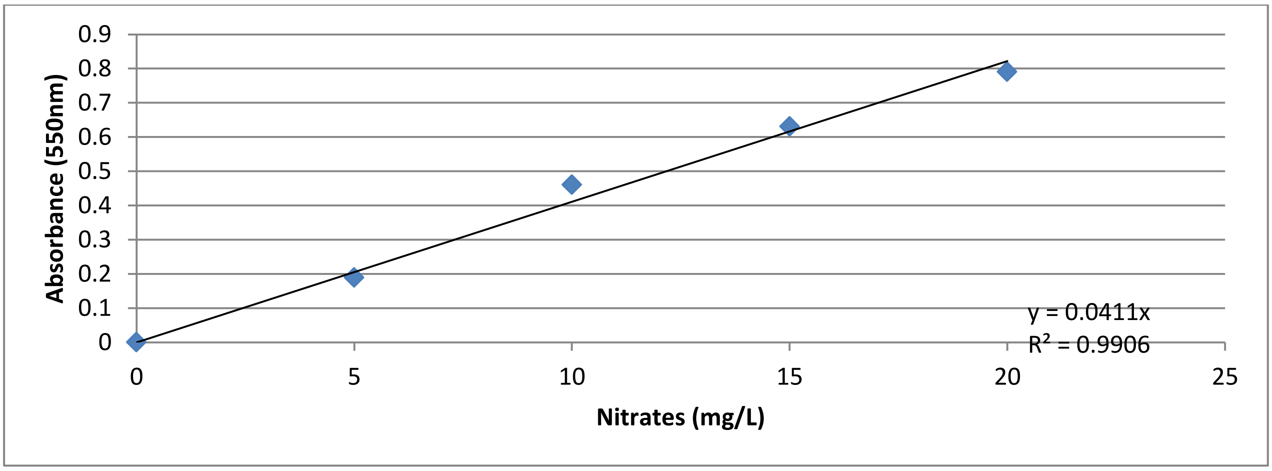

Figure B3: Graph illustrating the increasing concentrations of Nitrates-N at a wavelength of 550 nm, the curve was used to determine unknown Nitrate-N concentrations within the samples.

Table B8: A table showing different Ammonium-N wavelength results with regard to the different concentrations. Measurements were done in triplicate and the average was then calculated.

Concentration	1	2	3	Average
0	0	0	0	0
0.5	0.294	0.305	0.313	0.304
1	0.606	0.593	0.533	0.577333
2	1.152	1.097	1.165	1.138
3	1.583	1.638	1.582	1.601

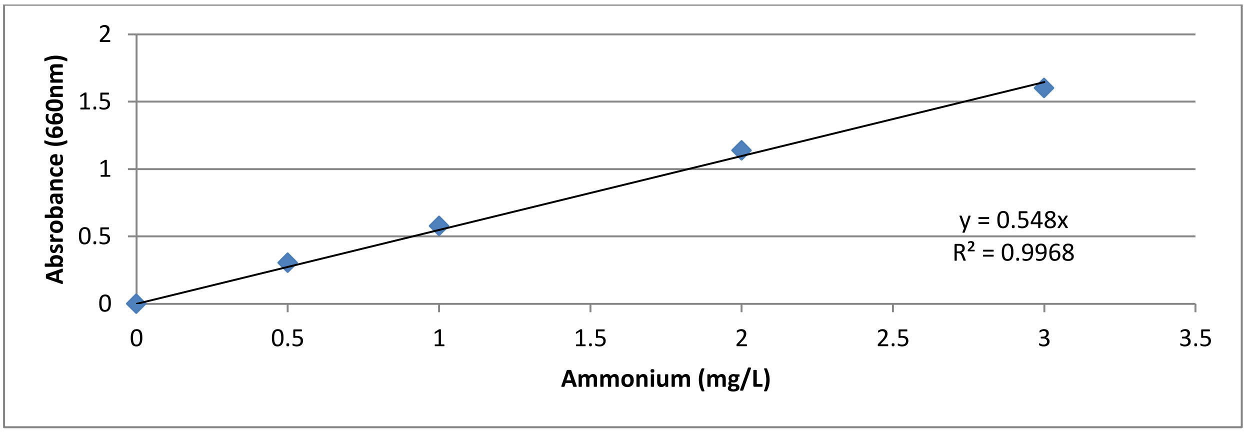

Figure B4: Graph illustrating the increasing concentrations of Ammonium-N at a wavelength of 660 nm, the curve was used to determine unknown Ammonium-N concentrations within the samples.