EVALUATION OF INTEGRATED ALGAE POND SYSTEMS FOR MUNICIPAL WASTEWATER TREATMENT

The Belmont Valley WWTW Pilot-Scale IAPS Case Study

AK Cowan, PM Mambo, DK Westensee and DS Render

WATER RESEARCH COMMISSION

TT 649/15

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Report to the
Water Research Commission

by

Institute for Environmental Biotechnology, Rhodes University (EBRU)
Rhodes University
Grahamstown

WRC Report No. TT 649/15

January 2016

Obtainable from

Water research Commission
Private Bag X03
Gezina, 0031

orders@wrc.org.za or download from www.wrc.org.za

The publication of this report emanates from a project entitled *Evaluation of Integrated Algae Pond Systems for Municipal Wastewater Treatment* (WRC Project No. K5/2123).

ISBN 978-1-4312-0732-9
Printed in the Republic of South Africa

Executive Summary

Integrated algae pond systems (IAPS) are a derivation of the Algal Integrated Wastewater Pond Systems (AIWPS®). Technology transfer into South Africa was facilitated by the Water Research Commission (WRC) and occurred in 1996 and a pilot plant was designed and commissioned at the Belmont Valley Wastewater Treatment Works (WWTW) in Grahamstown.

This report presents a reappraisal of IAPS as a technology for the treatment of municipal wastewater. Wastewater treatment processes currently used by South African municipalities include waste stabilization ponds (WS) or oxidation ponds (OP), activated sludge (AS), and bio-filtration (BF) which together comprises 92% of all technologies currently in use. Waste stabilization ponds are prolific; for the most part micro-sized (i.e. treat <0.5 $M\ell.d^{-1}$), and are ideally suited to transformation into IAPS to increase capacity without associated economic, energy and environmental costs. In view of the potential of this technology for deployment in South Africa, this report focusses on the IAPS bioprocess, design, component processes and operation, compliance, incorporation of tertiary treatment components, factors affecting the technology, and downstream valorization of the end products. Where necessary, aspects of IAPS technology that impact greenhouse gas (GHG) emissions and climate change have been addressed by life cycle assessment (LCA). Taken together, it is concluded that;

- IAPS is a contemporary wastewater treatment technology that is being intensively studied worldwide and at the water-energy-food nexus for CO_2 sequestration and to derive possible substitutes for fossil fuels;
- Modelling of the kinetic parameters of the Belmont Valley WWTW pilot-scale IAPS advanced facultative pond (AFP)-coupled in-pond digester (IPD) and high rate algae oxidation pond (HRAOP) components confirmed that both organic and hydraulic loading was commensurate with the original design specifications for a 500 person equivalent (PE) system;
- IAPS-treated water complies with the general limit values for either irrigation or discharge into a water resource that is not a listed water resource for volumes up to 2 Mℓ of treated wastewater on any given day; parameters including chemical oxygen demand (COD), total suspended solids (TSS), pH, dissolved oxygen (DO), electrical conductivity (EC), and N and P values were within the general limit after tertiary treatment by either a

maturation pond series (MPS), slow sand filtration (SSF) or controlled rock filtration (CRF); and, there is no faecal sludge handling;

- Large gaps in terms of technology status, design and process operation, and cost of construction exist that can only be addressed following implementation of full-scale commercial systems;
- LCA modelling, to map both energy flows and greenhouse gas (GHG) emissions of the Belmont Valley WWTW pilot-scale IAPS treating municipal sewage, revealed that an equivalent commercial system would yield 0.16 tonnes $CO_2 \cdot M\ell^{-1}$ of wastewater treated indicating a technology with an ability to mitigate climate change;
- Products from the 500 PE Belmont Valley WWTW pilot-scale IAPS treating municipal wastewater include water for re-cycle and re-use (~28 $M\ell \cdot y^{-1}$), methane-rich biogas (~1880 $kgCH_4 \cdot y^{-1}$equivalent to 26 MW or, ~55 $kWh.PE^{-1}.y^{-1}$), and biomass (>3 tonnes $DW \cdot y^{-1}$).

For IAPS to be successfully utilised as a municipal wastewater treatment technology, the roles of customer municipalities, technology providers, and consulting engineers have been integrated into an implementation strategy based on location, problem/need, engagement, and agreement to achieve a zero-waste solution and facilitate re-cycle and re-use.

Acknowledgements

The project team is grateful to the WRC for both technical and financial support of this research study.

Mr Dirk Westensee (MSc student) acknowledges receipt of a partial bursary from the National Research Foundation grant (IFR1202220169) awarded to Professor A.K. Cowan. Ms Prudence Mambo (PhD student) acknowledges financial support from the WRC Project (No. K5/2123) awarded to Prof A. Keith Cowan. Ms Zanele Mlambo (MSc student) was supported by a National research Foundation postgraduate scholarship. Rhodes University is acknowledged for providing bridging finance and additional funding via a grant to the Institute for Environmental Biotechnology (EBRU).

The technical and laboratory support from Mr Andile Magaba, Mr Olwethu Baba, Ms Nomaindia Toto, and Ms Michelle Isaacs is also gratefully acknowledged.

Technical input and consultation from the following are acknowledged, with thanks;

Frans Horjus	Royal Haskoning-DHV (previously DHV)
Paul Gaydon	WEC Projects (previously DHV)
Richard Laubscher	Institute for Environmental Biotechnology (EBRU)
Patricia Harvey	University of Greenwich, United Kingdom
Blahoslav Marsalek	Masarykova Univerzita, Brno, Czech Republic
Mohammad Abdel-Hamid	University of Mansoura, Mansoura, Egypt
Wageed Kamish	Stellenbosch University
Randall Winnaar	Element Consulting Engineers

The input provided by the reference group for this project is appreciated. In particular, we would like to thank Dr Valerie Naidoo for her role in supporting and championing this project. Reference group members include;

Bernelle Verster	University of Cape Town
Gary Brown	Tecroveer, Johannesburg
Sheena Kumari Santosh	Durban University of Technology
Kerneels Esterhuyse	Water and Sanitation Division, City of Tshwane
Clifford Jones	Rhodes University
Chris Buckley	University of KwaZulu-Natal
Jerrard Muller	JM & Associates,

Administrative support from Bennie Mokgonyana, Charmaine Khanyile and Shirley Machelesi of the WRC, and Patricia Jacob and Gila Eustace of Rhodes University, is much appreciated.

Capacity Building and Publications

Student Training

Zuma PZ, Exploring the fertiliser potential of biosolids from integrated algae wastewater treatment systems, MSc Rhodes University, 2013.

Westensee DK, Post-treatment technologies for integrated algal pond systems, MSc Rhodes University, 2014.

Mambo MP, Integrated algae pond systems for the treatment of municipal wastewater, PhD Rhodes University, 2015.

Operator Training

Baba O, IAPS operation and maintenance.

Research Publications and Conference Presentations

Cowan AK and Mlambo PZ (2015) Algae-based wastewater treatment: Opportunities in food production and biosystems engineering. In Petrotos K and Leontopoulos S, (eds.) Proceedings of 2nd International Conference on Food and Biosystems Engineering, Vol. **2** 50-55.

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Table of Contents

List of Figures

List of Tables

Abbreviations

AD	Anaerobic digester
ADB	Algae drying bed
AFP	Advanced facultative pond
AIWPS®	Advanced integrated wastewater pond system
AP	Aerobic pond
API	American Petroleum Institute
AS	Activated sludge
ASP	Algae settling pond
BF	Biofiltration
BOD	Biological oxygen demand
CCM	Carbon dioxide concentrating mechanism
COD	Chemical oxygen demand
CO_2e	Carbon dioxide equivalents
CRF	Controlled rock filtration
DAF	Dissolved air flotation
DMS	Dimethyl sulphide
DMSP	Dimethylsulfoniopropionate
DO	Dissolved oxygen
DOC	Dissolved organic carbon
DOM	Dissolved organic matter
DPW	Department of Public Works
DW	Dry weight
DWS	Department of Water and Sanitation
EC	Electrical conductivity
GHG	Greenhouse gas
GWP	Global warming potential
HRAOP	High rate algae oxidation pond
HRT	Hydraulic retention time
IAPS	Integrated algae pond system
IPD	In-pond digester
ISO	International Standards Organization
LCA	Life cycle assessment
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
MOB	Methane oxidising bacteria
PAR	Photosynthetic active radiation
PCD	Programmed cell death
PE	Person equivalents
POC	Particulate organic carbon
SDM	Secchi disk measurement
STP	Sewage treatment plant
SSF	Slow sand filtration
TKN	Total Kjeldahl nitrogen
TOC	Total organic carbon
TSS	Total suspended solids
TTU	Tertiary treatment unit
TVSS	Total volatile suspended solids
WRC	Water Research Commission
WSP	Waste stabilization pond
WWT	Wastewater treatment
WWTW	Wastewater treatment works

Preamble

Waste treatment technologies should be sustainable, support peri-urban primary industry such as agriculture, prevent exploitation of water reserves and other resources, and enhance the quality of life of communities (Wang et al., 2012). Any implemented technology must ideally be biologically/mechanically rigorous, ecologically sound and environmentally friendly (Golueke and Oswald, 1963, Oswald, 1991, Oswald, 1995) and the wastewater treatment works (WWTW) solid, able to withstand the elements, and require minimal maintenance over extended periods (Wallis et al., 2008, González et al., 2012). Thus, for a chosen technology to be considered sustainable its use should, over the medium to long-term, lower overall costs without sacrificing reliability and efficiency (Katukiza et al., 2012).

Rhodes University, together with funds from the Water Research Commission (WRC), constructed an Oswald-designed version of the AIWPS®, for local conditions, called the Integrated Algae Pond System (IAPS). This pilot-scale IAPS is located at the Belmont Valley WWTW. The system has been in continuous operation treating 75 $k\ell.d^{-1}$ municipal sewage and since commissioning in 1995 has been the subject of several WRC-funded projects. Despite being a passive process that functions virtually in perpetuity without need for sludge handling, IAPS remains demonstration-stage technology. The reasons for the status quo are unclear but in part due to; ignorance about the technology, perception that the treated water does not comply with standards set by the Department of Water and Sanitation (DWS), a perceived skills shortage, and an apparent lack of will to address wastewater management issues due mainly to the high costs of infrastructure repair and upgrade.

Correct implementation and management of IAPS is required to deliver clean water for recycle and reuse, energy, and biomass for valorization. Even so, and as with any near market ready technology there is the element of risk and/or failure to comply. The present work emphasizes the merits of IAPS and addresses questions and concerns to facilitate informed decision-making. In particular, attention is directed to the fundamental question of compliance. A review of IAPS as a bioprocess technology is presented to elaborate the state of the art. Re-examination of the design parameters, process configuration, treatment capacity and efficiency, and water quality is also described. Data on effluent quality, potential biogas output, sustainability, biomass yield, and a life cycle assessment of the sustainability credentials of IAPS is provided. Models arising from this data are used in a gap analysis to explore design parameters, beneficiation potential and versatility, ease of installation and operation and, where necessary, remedial capability.

1. Introduction

Domestic wastewater is an anthropogenically contaminated water body or stream which varies significantly depending on its origin and reaction to environmental influences chiefly rainfall and evaporation (Adewumi et al., 2010). Rainfall dilutes the effluent and evaporation has a concentrating effect (Adewumi et al., 2010, Ahmad et al., 2011). Origins of wastewater may be inclusive of but not limited to households, industry and agriculture (Bdour et al., 2009) and its source directly impacts its composition. However, factors such as social behaviour, economics, type and number of industries, area, climate, water consumption and the type and condition of the sewer system all contribute significantly to wastewater composition (Sonune et al., 2004, Su et al., 2012, Travis et al., 2012). Domestic wastewater may contain contaminants such as plastics, rags, plant debris, pathogenic bacteria, fats, greases, nitrates, phosphates, heavy metals, and other potentially hazardous compounds (Sonune et al., 2004, Ansa et al., 2012). Unless removed or rendered harmless in the waste-water treatment (WWT) process these can adversely affect the environment (Table 1). Thus, any remedial process must achieve an appropriate concentration of minerals and nutrients to avoid any acute or gradual influx into the environment of xenobiotics and toxic compounds (Lettinga, 1996; Debelius et al., 2009; Sekomo et al., 2012). The South African government, through the Department of Water and Sanitation (DWS) has therefore mandated the remediation of all effluent streams prior to discharge to the environment and the prescribed standards are presented in Table 2 (DWA, 2013). Discharge standards ensure that effluent streams released by municipalities (and industries) comply and will not be detrimental and/or damaging to the receiving environment.

Table 1 Contaminants commonly found in untreated domestic wastewater.

Contaminant	Effect on environment	References
Nutrients: nitrates, ammonium, phosphates	Proliferation of algae and nutrient depletion of still water bodies. Decomposition of algae utilizes available oxygen and eliminates photosynthesis by other plants in the system. Increased anaerobicity of the system and substantial decline in biodiversity.	Craggs et al.,1996a, Craggs et al.,1996b Craggs et al.,1995
Metals: copper, aluminium, cadmium, cobalt, iron, lead, manganese, nickel and zinc	Metals interfere with normal cell function to become toxic and damage the nervous system as well as vital organs of higher animals to exacerbate Alzheimer's and Parkinson's disease, muscular dystrophy, and multiple sclerosis	Debelius et al., 2009 Sekomo et al., 2012 Wilde et al., 1993
Chemical Oxygen Demand (COD)	COD increases risk of anaerobicity and reduces biodiversity in affected water systems	Craggs et al., 2004
Pathogenic bacteria	Approximately 2.4 billion people did not have commensurate access to sanitation and potable water which resulted in 1.7 million preventable deaths in 2002 due to cholera, typhoid and dysentry	Ansa et al., 2012 Mwabi et al., 2011 Koivunen et al., 2003

Innovation and advancement in the sector have proliferated wastewater treatment works (WWTW) and new process technologies are regularly made available as strategies to improve the management and remediation of wastewater (Bdour et al., 2009). Even so, management of WWT and control of final effluent quality/discharge is complex and some of the associated challenges include land, capacity, operations, maintenance and repair, technology developments, climate change, water course accessibility, and sustainability (Muga et al., 2008, Gravelet-Blondin et al., 1997). These coupled with available financial resources directly impact wastewater infrastructure by influencing design, construction, operation, inspection, maintenance, and the overall efficiency of the WWTW (Korf et al., 1996). Since WWT is not a free market enterprise in South Africa, acceptable “off-the-shelf” process technologies are viewed by many as those that are either already optimized or can be immediately optimized without consideration of additional energy and monetary costs.

Table 2 Current standards for disposal of treated wastewater into a water resource that is not a listed water resource and to irrigation of any land up to 2 Mℓ (DWA, 2013).

Variable	Discharge standard	Irrigation standard
pH	5.5-9.5	5.5-9.5
Electrical Conductivity ($mS{\cdot}m^{-1}$)	150	150
Suspended Solids ($mg{\cdot}ℓ^{-1}$)	25	25
Cloride as free Chlorine ($mg{\cdot}ℓ^{-1}$)	0.25	0.25
Fluoride ($mg{\cdot}ℓ^{-1}$)	75	1
Soap, Oil and Grease ($mg{\cdot}ℓ^{-1}$)	2.5	2.5
Chemical Oxygen Demand ($mg{\cdot}ℓ^{-1}$)	75	75
Faecal coliforms (cfu per 100 mℓ)	1 000	1 000
Ammonia ionised and unionised as nitrogen ($mg{\cdot}ℓ^{-1}$)	6	3
Nitrate/Nitrite ($mg{\cdot}ℓ^{-1}$)	15	15
Ortho-phosphate as phosphorus ($mg{\cdot}ℓ^{-1}$)	10	10

Wastewater treatment technologies currently deployed in South Africa include waste stabilization ponds (WS) or oxidation ponds (OP), activated sludge (AS), bio-filtration (BF), biological nutrient removal (BNR), constructed wetlands (CW), and more (Adewumi et al., 2010, Oller et al., 2011). A brief description of other globally available WWT technologies is presented in Table 3.

The Green Drop Report (DWS, 2012) indicates that in South Africa, 156 municipalities provide wastewater services via a network of 821 collector and treatment systems with total design capacity of 6 614 $Mℓ{\cdot}d^{-1}$ and actual received flow of 5 258 $Mℓ{\cdot}d^{-1}$, leaving a spare capacity of 1 356 $Mℓ{\cdot}d^{-1}$. In 2009, it was estimated that the operational cost of wastewater treatment then, exceeded R 3.5 $billion{\cdot}y^{-1}$ (DWS, 2009). Distribution of these WWTW according to size shows distinct differences between the nine provinces specifically:

- Gauteng province has a relatively high number of medium (defined as WWTW with a design capacity in the range 2-10 $M\ell \cdot d^{-1}$) and large (defined as WWTW with a design capacity in the range 10-25 $M\ell \cdot d^{-1}$) WWTWs, with fewer micro (defined as WWTW with a design capacity <0.5 $M\ell \cdot d^{-1}$) and small size (defined as WWTW with a design capacity in the range 0.5-2 $M\ell \cdot d^{-1}$) plants;
- Eastern Cape, Northern Cape, Mpumalanga and Limpopo provinces mainly have micro size and small size plants;
- North West, KwaZulu-Natal and Free State provinces have a wider spread of WWTW across all the plant size categories; and
- The Western Cape Province has a spread of WWTW sizes similar to the national situation (Fig. 1).

Thus, >80% of municipal WWTWs treat less than 10 $M\ell \cdot d^{-1}$ and more than 50% of all WWTWs are micro sized while the preferred technologies are WSP and AS at 41 and 35% respectively. Due to a paucity of information it is not possible to determine what proportion of the estimated total flow is treated by each technology. Suffice it to say, together with bio-filtration at 16%, the range of WWT technologies commissioned by municipalities in South Africa is particularly narrow but distinctly biological.

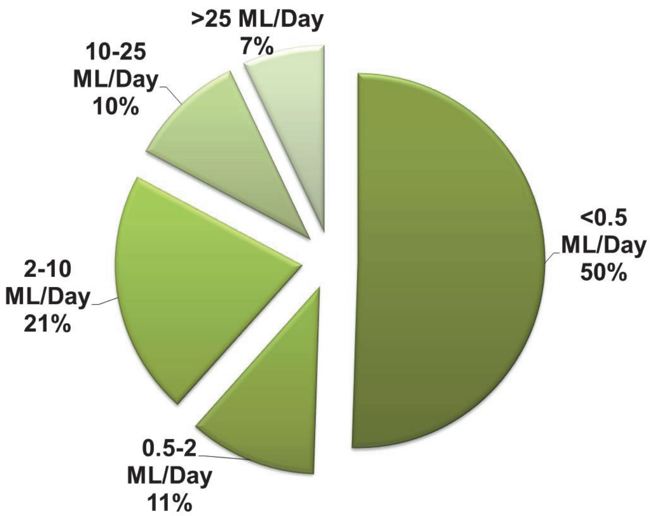

Figure 1 Size distribution of municipal wastewater treatment works in South Africa (DWS, 2009).

Biological remediation of wastewater has for many years generally been favoured over conventional treatment techniques even in light of the major limitation which is sensitivity to toxic components (Korf et al., 1996). Contemporary evaluation would seem to share this opinion and is based largely on the costs involved in the construction and maintenance of

Table 3 Some examples of available wastewater treatment technologies.

Technology	Brief description	Reference
Advanced oxidation	Ozonation, hydrogen peroxide or UV light to remediate wastewater containing biologically toxic and/or recalcitrant compounds, e.g. aromatics, pesticides, petroleum, and volatile organic compounds.	Oller et al., 2011 Cha et al., 1997
Aerobic granular reactor	Microbial biomass supported on fast settling granules to efficiently remove COD, nitrogen and phosphorous in a discontinuous system.	Bassin et al., 2012 Quan et al., 2012
Anaerobic filter	Anaerobic microorganisms attached to filter media under anaerobic conditions to remediate wastewater.	Hamdi & Garcia, 1991
American Petroleum Institute (API) oil water separators	Utilization of the specific gravity difference between oil and wastewater to separate oil from wastewater.	Punnaruttanakun et al., 2003 Deng et al., 2002
Anaerobic lagoon	Degradation of animal wastewater in a manmade water body by microorganisms.	Wu et al., 2011 Safley et al., 1992
Constructed wetland	Artificial marsh built to act as a biofilter capable of removing sediments and contaminants such as heavy metals from wastewater.	Travis et al., 2012 Kaseva, 2004 Kivaisi, 2001
Dark fermentation	Anaerobic biohydrogen production through the degradation of contaminants found in industrial and domestic wastewater.	Kargi et al., 2012 Ozmihci et al., 2011
Dissolved air flotation	Pressurized air is introduced to wastewater. When the pressure is released air rises while adhering to solids and this effectively removes wastes from suspension.	Edzwald, 1995
Desalination	Removal of salts and minerals (filtration, chemical precipitation, etc.) from saline water to generate water suitable for irrigation or following the relevant further remediation drinking water.	Bdour et al., 2009 Urkiaga et al., 2006
Electrocoagulation	Application of an electrical pulse in order to charge the surface charge of particles in wastewater to cause its aggregation and consequent sedimentation thereby remediating the water.	Feng et al., 2007 Holt et al., 2005
Electrolysis	Introduction of an electric current to wastewater that results in its separation into its constituents, these subsequently settle out of suspension and generate a clean effluent.	Cheng et al., 2007
Forward osmosis	Application of an osmotic potential through a semi permeable membrane that results in the separation of solutes from wastewater, rendering it clean.	Wang et al., 2010 Yangali-Quintanilla et al., 2011
Reverse Osmosis	Employment of pressure on a selective membrane that allows the solvent to pass through freely but retains solutes and other debris on the pressurized side of the membrane	Yangali-Quintanilla et al., 2011
Rotating biological contactor	Closely spaced parallel discs with a biofilm layer are introduced to wastewater the microorganisms in the biofilm take up the nutrients while degrading any organic compounds in the wastewater. This process remediates the water.	Buchanan et al., 1994 Teixeira et al., 2001
Septic tank	An anaerobic digester that may be directly linked to a household. It results in the degradation of organic compounds and the remediation of wastewater which then seeps into the environment.	Moussavi et al., 2010 Withers et al., 2011
Sequencing batch reactor (aerobic or anaerobic)	Bacteria are used to remediate wastewater, oxygen is pumped into the first reactor aiding in the complete aerobic breakdown of the components of the wastewater. The effluent generated is then channelled into a second reactor where any suspended solids are allowed to settle out of suspension.	Bassin et al., 2012 Liu et al., 2005 Rodrigues et al., 2003
Submerged aerated filters	Employment of an upflow fixed biofilm reactor with a coarse medium that does not require backwashing. Nitrification is utilized to remediate wastewater.	Khoshfetrat et al., 2011
Ultrafiltration	Use of hydrostatic pressure through a semi permeable membrane that results in the separation of solutes from the solvent. It is one of a variety of membrane technologies which vary depending on size of compound being filtered, i.e. nanofiltration, microfiltration and gas separation.	Cha et al.,1997 Cherkasov et al., 1995 Hamza et al., 1997
Upflow anaerobic digester	Employment of gravity within an anaerobic digestion to settle organic solids out of suspension for digestion and retention in the reactor, while the clean effluent is channelled out of the reactor.	Parawira et al., 2005 Hansen et al., 1992
Wet oxidation/ Zimpro Wet Air Oxidation	Superheated air is used to oxidize components in wastewater for degradation by conventional wastewater treatment systems.	Cha et al., 1997 Lei et al., 1998 Sun et al., 2008

biological treatment facilities (Cisneros et al., 2011). Toxicity, while a potential hazard to the microbial biocatalysts used in wastewater treatment, may be attributed to content and composition and factors such as shifts in pH and temperature (Muga et al., 2008; Chan et al., 2009). Typically, it is maintenance of optimum biocatalyst activity that completely degrades organic pollutants (Gori et al., 2011; Mo et al., 2012; Daelman et al., 2012) and effects mineral and nutrient removal to yield a treated effluent that can be discharged regardless of shock loads (Gori et al., 2011, Rodriguez-Garcia et al., 2012). Gaseous bi-products emitted as a consequence of remediation particularly from waste stabilization ponds include CO_2, CH_4, fluorinated gases and nitrous oxide (Strutt et al., 2008).

Due to the nature and construction of ponds, large emissions of CO_2 and CH_4 from the surface area can be expected and CH_4 production rates of 0.17 kg $CH_4 \cdot kg^{-1}$ biological oxygen demand $(BOD)_{waste}$ were found in models of anaerobic ponds fed with municipal sewage (DeGarie et al., 2000, van der Steen et al., 2003). Furthermore, about 3.3 kℓ per person equivalents $(PE)^{-1} \cdot y^{-1}$ of CH_4 is produced by full scale ponds based on a daily sewage production of 100 g chemical oxygen demand $(COD) \cdot PE^{-1} \cdot d^{-1}$. While CO_2 is the best studied and most known greenhouse gas (GHG) CH_4, which constitutes up to 75 % of the total gas emitted during anaerobic wastewater treatment, is 25 times more potent (Forster et al., 2007, Daelman et al., 2012). Similarly nitrous oxide, while emitted in relatively low amounts, is 300 times more damaging than CO_2 (Daelman et al., 2012; Strutt et al., 2008). Although, harvesting and/or recycle of these gases can potentially avert any detrimental impact (Oswald, 1995; Green et al., 1995a), it is now understood that methane oxidizing bacteria (MOB), a relatively common group of bacteria capable of utilizing CH_4 as their sole carbon and energy source if sufficient O_2 is present, utilize in-pond algae-derived O_2 to consume much of the emitted CH_4 before it reaches the atmosphere (van der Ha et al., 2011). Even so, only part of the CH_4 produced by methanogens in the anaerobic sludge blanket is consumed by MOB before it reaches the atmosphere and elementary extrapolation of measured pond emissions still show a total loss of about 3000 kℓ$\cdot CH_4 \cdot y^{-1}$, equalling a yearly contribution of 55 tonnes CO_2-equivalents$\cdot y^{-1}$ or an emission of 0.98 kg $CH_4 \cdot y^{-1} \cdot PE^{-1}$.

Despite concessions to global warming, wastewater treatment by biological means remains vital for reclaiming this essential resource to continuously replenish environmental reserves and mitigate exploitation of untapped water sources (Gravelet-Blondin et al., 1997; Showers, 2002). Unfortunately, current WWTW infrastructure in South Africa is unable to cope with the sheer volumes generated by urban areas and it is estimated that >80% are either in disrepair, underperform or are overloaded (DWS, 2010). Population growth and immigration are seen as major contributors (Korf et al., 1996; Van Koppen, 2003; Showers, 2002). Furthermore, due to limited resources waste management including wastewater treatment has been neglected in favour of other priorities (e.g. health, housing, education) preventing acquisition of new infrastructure and provision of associated municipal services (Cornel et al., 2011; Wang et al., 2012). As a consequence, municipalities in many southern African countries have little choice but to continue to discharge partially and untreated wastewater to the environment (Wang et al., 2012).

The primary goal of the National Waste Management Strategy (NWMS) is to achieve the objectives of the Waste Act (Republic of South Africa, *Waste Act 2008*; Republic of South Africa, *National Environmental Management: Waste Amendment Act 2014*), which are: 1, minimizing pollution, environmental degradation and the consumption of natural resources, 2, implementing the waste hierarchy, 3, balancing the need for ecologically sustainable development with economic and social development, and 4, promoting universal and affordable waste services. Framed within the context of the overall goals, approach and regulatory model of the NWMS, introduction of WWT technologies requires demonstration of proficiency, education, and increased awareness amongst all stakeholders including the public at large, the three spheres of government, and the private sector. Statutory, para-statutory and non-governmental organizations and citizens share a common concern regarding the national crisis relating to small and medium municipal WWTW in South Africa, many of which are currently in a state of disrepair and are blamed for disease outbreak and infant mortality (Green Drop Report 2009, 2012). A skills shortage, apparent lack of will to address these issues due mainly to the high costs of infrastructure repair and upgrade, and poor technology choices have not helped the situation. While any and all exposure and attention to this problem is real, there is the risk that efforts to mitigate the crisis will sow seeds for a new one through inappropriate or unsustainable technology choices. Population growth, migration, financial challenges at local government level, water shortages in many areas, the shortage and the cost of skilled personnel and the cost of electricity, among others, all impact decisions of choice. There are alternative technologies to the skills-intensive and widely accepted activated sludge process. This includes algal ponding systems (Horjus et al., 2010). But are these being adapted for adoption in a changing South African scenario? (Laxton, 2010; Mambo et al., 2013).

2. IAPS as a Bioprocess for Municipal Sewage Treatment

Sewage treatment typically comprises five distinct phases. Primary treatment involves removal of suspended solids. Removal of dissolved biodegradable organic matter is a secondary treatment that reduces BOD to a level sufficient to prevent oxygen depletion of the receiving water body. Nitrogen and phosphorus are removed by tertiary treatment to minimize growth of algae and other aquatic plants. Removal of refractory organic compounds is achieved by quaternary treatment while quinary treatment removes dissolved organics and salts including heavy metals. Successful waste treatment technologies should be sustainable, support peri-urban primary industry such as agriculture, prevent exploitation of water reserves and other resources, and enhance the quality of life of the community (Wang et al., 2012). Wastewater treatment must also be biologically/ mechanically rigorous, ecologically sound and environmentally friendly (Golueke and Oswald, 1963; Oswald, 1991; 1995) and the WWTW solid, able to withstand the elements and require minimal maintenance over an extended period of time (Wallis et al., 2008; González et al., 2012). Thus, for an implemented technology to be considered a sustainable process its use should over the medium to long-term, lower the overall cost without sacrificing reliability and efficiency (Katukiza et al., 2012).

Waste stabilization ponds (WSP) are a technology used prolifically by municipalities in South Africa. As stated by Oswald (1995) "*The greatest advantages of ponds are their simplicity, economy, and reliability; their greatest drawbacks are their high land use, their potential for odour, and their tendency to eutrophy and fill in with sludge and to become less effective with age.*" Research to maintain the advantages of WSPs while mitigating the drawbacks resulted in the innovation known as the AIWPS® (Oswald et al., 1957), which is still utilized globally for the remediation of domestic wastewater (Oswald, 1995; Green et al., 1995b; Craggs et al., 1996b; Craggs, 2005; Park et al., 2011b). Examples of this technology are depicted in Figure 2.

The term AIWPS® is registered and used primarily in the U.S.A. to describe IAPS wastewater treatment technology.

In developing the IAPS concept, focus was initially on the symbiotic relationship between algae and bacteria in wastewater treatment (Oswald et al., 1955). Later, the term photosynthetic oxygenation was coined (Oswald et al., 1957) and used to describe the aeration effect caused by algae (Ludwig et al., 1951; 1952; Oswald et al., 1953b; 1955). By

1957, Oswald had established the high rate algae oxidation pond (HRAOP). This algae-containing raceway amalgamated wastewater remediation via biological oxygenation and nutrient removal, and led eventually to the fully developed bioprocess system (Oswald, 1991; 1995). Based on results from studies over extended periods it was concluded that when properly designed, IAPS were economical, effective, attractive, and problem free (Green et al., 1995; Oswald, 1995).

Figure 2 Examples of IAPS implemented in California, USA. A) Delhi, California: design flow = 3.2 Mℓ·d^{-1}, wet surface area = 8 ha, total area = 16 ha; B) Hilmar, California: design flow = 2.5 Mℓ·d^{-1}; wet surface area = 7 ha; total area = 15 ha; C) St Helena, California: design flow unknown.

As shown in Figure 3, there are several versions of AIWPS®/IAPS and these have been categorized into first, second and third generation processes depending on the quality of the final effluent (Green, 1996). First generation systems remediate domestic wastewater to a standard suitable for discharge to the environment whereas second and third generation systems are equipped for reclamation of water, and for the harvest of methane, and in some cases harvest of algae biomass (Green, 1996).

The various IAPS systems rely on the combined activity of anaerobic digestion, photosynthetic oxygenation by algae, and microbial oxidation to achieve wastewater treatment (Oswald, 1995; Craggs et al., 1996b; Downing et al., 2002). Thus, the natural functionality of anaerobic, facultative and aerobic microorganisms is exploited by process designs that typically include: an in-pond digester (IPD), an advanced facultative pond (AFP), HRAOPs, algae settling ponds (ASP) and maturation ponds (MP). A high quality tertiary treated water is obtained following filtration and UV sterilization. As stated by Oswald (1990), "*when properly designed in appropriate locations, the systems virtually eliminate sludge disposal, minimize power use, require less land than conventional ponds, and are much more reliable and economical than mechanical systems of equal capacity*". In brief, IAPS technology represents an innovative re-design in which the low-cost reactors from waste stabilisation and oxidation ponds have been incorporated into a single system. No sludge management is required and the time in which sludge residues accumulate to

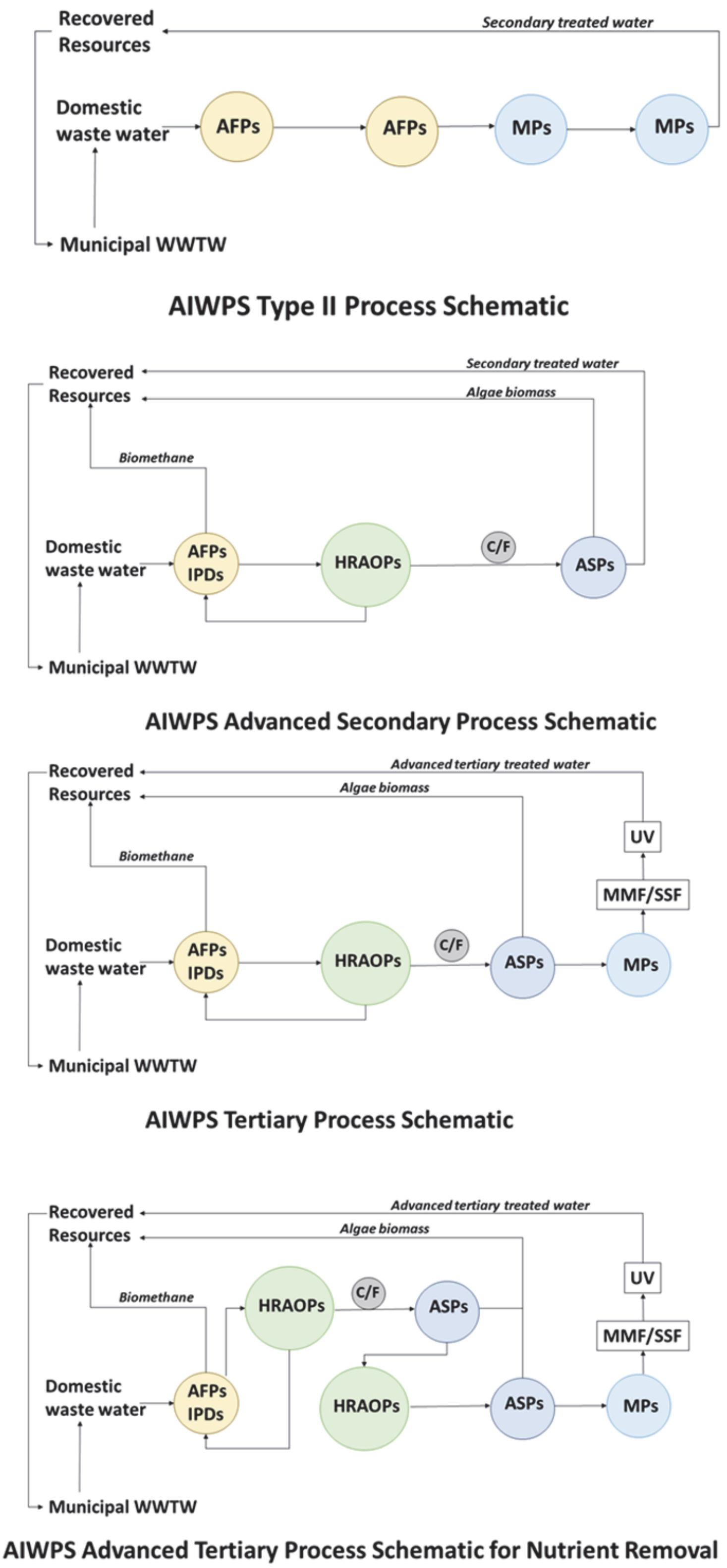

Figure 3 Schematics of IAPS process flows and operating configurations for nutrient, energy and water recovery. AFP = advanced facultative pond; IPD = in-pond digester; HRAOP = high rate algae oxidation pond; C/F = coagulation/flocculation; ASP = algae settling pond; MP = maturation pond; MMF = multimedia filtration; UV = ultraviolet light.

require removal and disposal is of the order of decades. The conversion of organic solids to methane, nitrogen gas and carbon dioxide via methane fermentation, and the assimilation of nutrients, and organic and inorganic carbon into algae biomass via photosynthesis, provide the basis for primary, secondary, and tertiary treatment (Green et al., 1995a).

2.1 IAPS as a Global Wastewater Treatment Technology

During the past six decades IAPS have been successfully implemented in Australia, Belgium, Brazil, Canada, China, Egypt, Ethiopia, France, Germany, India, Ireland, Israel, Italy, Kuwait, Mexico, Morocco, New Zealand, The Netherlands, Philippines, Portugal, Scotland, Singapore, South Africa, Spain, Switzerland, Thailand, U.S.A., Vietnam and Zimbabwe (Oswald, 1995). More recent deployments of this bioprocess technology are presented in Table 4.

Global interest in IAPS is primarily as a 'green' technology in which HRAOPs are considered algae production units with the potential to address imperative issues such as global warming and climate change (Wallis et al., 2008; Brune et al., 2009).

Algae biomass generated during wastewater treatment, i.e. the standing biomass, represents a carbon sink and thus mitigates the negative effect of CO_2 by photosynthetic sequestration of this greenhouse gas (Green et al., 1995). Microalgae mass culture can also be used to biofix power plant flue gas and other concentrated CO_2 sources into biomass that can then be used to produce renewable fuels such as methane, ethanol, biodiesel, oils and hydrogen and other fossil-fuel sparing products and processes (Dalrymple et al., 2013). Thus, as a product of wastewater treatment, microalgae generated in HRAOPs may be used to justify the use of IAPS as a sustainable and environmentally friendly technology (Oswald, 1995) and to mitigate emissions of fossil CO_2 and other greenhouse gases. Also, application to horticulture has shown that the algal biomass produced in the HRAOPs provides a fertiliser and/or soil amendment function in high-value horticulture which is at least equivalent to, if not better than, commercial chemical fertilisers (Cowan and Mlambo, 2015). This input to organic farming emphasises the versatility of this wastewater treatment technology in which the disinfected treated water is recovered and recycled and the biomass valorised to commodity products. In addition to municipal sewage, brewery effluent, food processing waste, industrial effluent, and abattoir waste have successfully been remediated by IAPS (Van Hille et al., 1999; Rose et al., 1996; Boshoff et al., 2004).

Table 4 Examples of IAPS for wastewater treatment.

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
Belgium	Temperate	Piggery	HRAOPs		Discharge to environment	1982	De Pauw & Vaerenbergh, 1983
Brazil	Tropical-Temperate	Domestic	HRAOPs		Discharge to environment	1983	Kawai et al., 1984
Canada	Boreal	Piggery	HRAOPs		Discharge to environment	1989	Buelna et al., 1990
China	Monsoon	Domestic	HRAOPs		Discharge to environment	2002	Chen et al., 2003
Egypt	Desert	Domestic	HRAOPs		Discharged to environment	1990	El-Gohary et al., 1991
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., 2004
France	Temperate	Domestic	HRAOPs			1997	Bahlaoui et al., 1997
Germany	Temperate	Domestic	HRAOPs		Discharge to environment	1986	Grobbelaar et al., 1988
India	Tropical	Domestic	AFPs, HRAOP, ASP, MP	98 % BOD, 92 % SS, 91 % nitrogen 96 % E. Coli removal	Discharge to environment	1986	Mahadevaswamy & Venkatamaran, 1986
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
Jordan	Desert	Domestic	Two AFPs and four MPs	85 % BOD removal	Discharge to environment	1985	Al-Salem, 1987
Morocco	Mediterranean	Domestic	IAPS		Discharge to environment	2003	El Hamouri et al., 2003
The Netherlands	Moderate maritime	Domestic	IAPS		Discharge to environment	1988	Kroon et al., 1989
New Zealand	Temperate	Domestic	Two HRAOPs	100 % faecal coliform disinfection	Nutrient rich biomass as fertilizer	2007	Craggs et al., 2003
New Zealand	Temperate	Domestic	Four adjoining HRAOPs. solids removal pre-treatment and fertilizer harvest	91 % BOD 67 % ammonium 24 % phosphate 99 % coliform removal pH 9.3	Discharge to environment	2010	Broekhuizen et al., 2012
Singapore	Tropical	Piggery	HRAOPs			1997	Taiganides, 1997
Spain	Mediterranean	Domestic	Two HRAOPs		Discharge to environment	2006	Garcia et al., 2006
Sweden	Cold	Domestic	AFP, HRAOPs, MP	97 % BOD 64 % phosphate 90 % nitrogen removal	Discharge to environment	2003	Grondlund et al., 2004
United States of America	Mediterranean	Domestic	AFP, HRAOP, ASP, then DAF prior to introduction into a reverse osmosis plant	82 % BOD, 99 % nitrogen, 98 % TSS removal Membrane fouling after 100 days	Discharge to environment	1959	Downing et al., 2002
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	Discharge to environment	1959	Oswald and Golueke, 1960

				99.999% coliform removal			
United States of America	Mediterranean	Domestic	AFP, HRAOP, ASP, MP	97 % BOD, 93 % COD, 90 % nitrogen 64 % phosphate removal	Discharge to environment	1959	Oswald and Golueke, 1960
United States of America	Mediterranean	Domestic	AFP, HRAOP ASP	96 % BOD, 42 % TVSS 99.999 % E. Coli removal	Discharge to environment	1959	Oswald and Golueke, 1960

2.2 Design and Operation of IAPS

Primary treatment takes place in an AFP which houses the IPD. The IPD is the point of entry of raw wastewater and is responsible for the anaerobic decomposition of organic matter (Oswald, 1995). A coupled IPD/AFP promotes the deposition of organic material from suspension to facilitate decomposition at the base of the pond (Oswald, 1996). After 30 years of operation in the United States of America, sludge removal from fermentation pits has yet to be conducted (Green et al., 1996; Daelman et al., 2012; Katukiza et al., 2012).

The AFP is designed to reduce the BOD significantly and buffer the effluent prior to transfer to HRAOPs while the aerobic surface layer of the pond neutralizes odour causing compounds, e.g. hydrogen sulphide (Lettinga, 1996; Oswald, 1995; Green et al., 1996; (Muga and Mihelcic, 2008).

Secondary (and tertiary) treatment is carried out in HRAOPs operated in series in which nutrients are extracted by a rapidly growing naturally-occurring algae biomass. Algae photosynthesis supplies aerobic heterotrophic bacteria directly with oxygen while the bacteria in turn oxidise recalcitrant material to increase the nutrient load in solution. It is the assimilation of waterborne nutrients such as nitrate, ammonium and phosphates together with photosynthetic carbon reduction that drives algae growth and development. Typically, HRAOPs are 0.1-0.5 m deep and the entire water column is oxygenated by both algae photosynthesis and paddlewheel mixing. Paddlewheels pump water at a specific linear velocity and the action of the paddles on the water surface causes sufficient turbulence to allow for the introduction of oxygen, and CO_2, from the outside air. Total oxygenation capacity of the pond and the installed power of the paddlewheel give an oxygenation efficiency of 15 $kgO_2 \cdot kWh^{-1}$, which is a factor of 10 better than most mechanical aerators (Oswald, 1988; 1990). Thus, the oxygenation capacity of HRAOPs can be contrasted with mechanical aerators which rarely transfer oxygen from air to water at more than one kg·kWh^{-1} indicating that photosynthetic oxygenation is 10-100 times more efficient as all energy is solar derived.

Although CO_2 availability within wastewater treatment HRAOPs depends primarily on the heterotrophic oxidation of organic compounds by bacteria (Weissman and Goebel, 1987; Oswald, 1988; Craggs, 2005), domestic sewage typically contains insufficient carbon to fully support optimal algal production (3-7 C:N ratio in sewage versus 6-15 C:N in algal biomass) (Benemann, 2003). Recently it was shown that addition of CO_2 to wastewater HRAOPs enhanced algal productivity by at least 30% (Park et al., 2011b) and reduced nitrogen loss by ammonia volatilization to provide more nitrogen for recovery by assimilation into biomass (Park and Craggs, 2011). Even so, unicellular green algae and cyanobacteria cultivated in ambient air levels of CO_2 develop a dissolved inorganic carbon concentrating mechanism (also called a CO_2 concentrating mechanism or CCM) which is suppressed when cultured at elevated CO_2 and inhibited by O_2 (Ghoshal and Goyal, 2001). Similarly, denitrification and dissimilation, which converts nitrate to nitrogen gas, only occur in the absence of oxygen (Mitchell, 1974). In short, oxygen decreases the denitrification rate even if denitrifiers possess aerobic denitrification ability (Patureau et al., 1996). It might therefore be expected that the very high oxygenation capacity of HRAOPs would limit both growth of algae and denitrification. Whereas higher dissolved oxygen favours nitrification, denitrification (and nitrification) rates increase with increasing temperature and the diel (i.e. during the adjoining dark period) loss of nitrogen via denitrification for algae ponds appears to be 15-25% of total influent nitrogen (Zimmo et al., 2004).

For growth, the mechanism by which inorganic carbon species are taken up by algae involves the light-induced drawdown of inorganic carbon by photosynthetic carbon reduction which maintains a concentration gradient between the external medium and the active site of the primary photosynthetic enzyme, ribulose bisphosphate carboxylase oxygenase (Raven and Hurd, 2012). CO_2 reacts in water and equilibrium is established between CO_2 and carbonic acid (H_2CO_3). The conversion of CO_2 to H_2CO_3 is kinetically slow and at equilibrium only a fraction of CO_2 exists as H_2CO_3 with most remaining as solvated molecular CO_2. Carbonic acid dissociates in water in two steps to produce carbonate anions as follows: $H_2CO_3 + H_2O \leftrightarrow H_3O^+ + HCO_3^-$ (pK_{a1} at 25 °C = 6.37) and; $HCO_3^- + H_2O \leftrightarrow H_3O^+ + CO_3^{2-}$ (pK_{a2} at 25 °C = 10.25). It is the formation of carbonate ions and their interaction with cations that leads to deposition of insoluble metal carbonates (e.g. $CaCO_3$; $MgCO_3$) and which provides an additional driving force (Lide, 2006). Consequently, net photosynthetic rate of an algae pond at optimal depth (0.3 m) and under optimum light and temperature is almost always constant at approximately 10 t(C) $ha^{-1} \cdot y^{-1}$ over the course of a day because any increase in cell density, or decrease in photosynthetically active radiation, proportionately reduces the optimum pond depth and *vice versa* (Grobbelaar, 2007; Ritchie and Larkum, 2013). Even so, continual gravitation of effluent from the first HRAOP via ASPs to the second HRAOP removes some of the accumulated algae biomass (and residual bacteria) to mitigate substantive changes in optimum pond depth thereby increasing nutrient abstraction efficiency (Oswald, 1995). In addition, sustained algae photosynthetic activity coupled with nitrification and nitrate consumption leads to an increase in medium pH. Most of the energy for nitrate assimilation arises from photosynthesis, photosynthesis is also reported to be responsible for light regulation of nitrate reductase gene expression and activity (Lillo et al., 1996; Oswald et al., 2001), and linear electron flow and generation of reducing equivalents are promoted by photosystem 1 (PS l) light absorption which is believed to facilitate reduction of assimilated nitrate alongside CO_2 (Sherameti et al., 2002). Thus, as long as nitrate is abstracted, reduced to ammonium and the ammonium assimilated into amino nitrogen a 1:1 alkalinisation in relation to nitrate consumption is maintained (Ullrich and Novacky, 1990; Mistrik and Ullrich, 1996; Ullrich et al., 1998). Alkalinisation in the HRAOPs has been suggested as a mechanism, separate from biological assimilation, to promote removal of phosphate in the form of an insoluble hydroxyapatite. Thus, elevated pH (>10) can stimulate not only ammonia-N removal from the HRAOP by ammonia volatilization but phosphorus removal through phosphate precipitation with calcium, magnesium and non-chelated ferric iron (García et al., 2000; Craggs, 2005).

The above account on the biochemistry of nutrient abstraction and assimilation into biomass in HRAOPs has ignored, for the sake of brevity, some critical environmental (light and temperature), operational and other biological factors (zooplankton grazers and algal pathogens) that do impact wastewater treatment. However, this omission only serves to further strengthen the assertion by Oswald (1990) that correct design, locality and operation are paramount for successful implementation of this bioprocess technology. Secondary and tertiary treatment of wastewater can be fully accounted for by passage through a series of HRAOPs. Ideally suited to warm climates in which high BOD removal capacity is easily realised, these systems retain all of the advantages of WSPs and while land requirements are substantially more than needed for AS (not accounting for land used in sludge management), operational and capital costs have been estimated at one half and one fifth respectively of those required for AS (Park et al., 2011a; Craggs et al., 2011).

The final reaction in the IAPS bioprocess is tertiary treatment usually achieved in a series of MPs or by filtration (e.g. slow sand filter). Maturation ponds hold secondary treated effluent and are typically positioned downstream from conventional treatment systems (Shillinglaw, 1977). The main function of MPs is additional polishing of the water to clean and remove any residual pathogens carried forward from the secondary treatment process (Mara, 2005). Prevailing environmental conditions such as: high pH, low temperature, high dissolved oxygen (DO) and ultraviolet radiation are exploited (Von Sperling and de Lemos Chernicharo, 2005) and usually two or three ponds are constructed in series to provide the retention time (minimum of 12 days) needed for adequate pathogen removal. Maturation ponds provide little or no biological stratification, have high algae diversity which increases further across a pond series, and tend to be fully oxygenated throughout the day providing ideal conditions for faecal coliform/pathogen removal (Mara, 2005). A high pH is found in maturation ponds which impacts faecal bacteria mortality (von Sperling, 2007) and enhances nitrogen removal both by assimilation into biomass and loss via volatilization (Kayombo et al., 2005). In fact, ammonia removal in maturation ponds exceeds that of other tertiary treatment processes (e.g. constructed wetland) and in a comparative assessment was second only to aerated rock filtration (Johnson et al., 2007).

Quaternary and/or quinary treatment are not typically components of IAPS wastewater treatment systems and will not be discussed here.

2.3 IAPS Components and Function

In-Pond Digester and Advanced Facultative Pond – Facultative ponds are typically 0.9-2.4 m in depth, with a detention time up to 50 days (Nelson et al., 2004). Advanced facultative ponds receive raw wastewater directly into the IPD while secondary facultative ponds treat settled wastewater (Mashauri and Kayombo, 2002; Nelson et al., 2004). Very little information is available on the biology and function of IAPS in-pond digesters. Preliminary data derived from the pilot-scale IAPS at the Belmont Valley WWTW has indicated the potential to produce a biogas stream comprising more than 80% methane. 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. As indicated in Table 5, this equates to a theoretical maximum of approximately 40 m^3 biogas·d^{-1} for a 100-household system (i.e. ~600 PE).

Biogas (methane equivalent) at this maximum rate of production (i.e. 35 kℓ·d^{-1}) and with calorific value of ~49 MJ·kg^{-1} could theoretically give a total daily energy stream of 1.7 GJ from the Belmont Valley WWTW pilot IAPS. Unfortunately, these systems do not operate near the theoretical maximum and recent kinetic studies have revealed a biogas production rate with methane and energy content as indicated in Table 6.

Table 5 Methane production from IAPS in-pond digesters.

Production rate	Richmond, CA	Theoretical Maximum
m^3 CH_4·kg BOD_5^{-1}	0.15	0.53
m^3 CH_4·kg BOD_{ULT}^{-1}	0.24	0.85
m^3 CH_4·d^{-1} (400 PE)	7.68	27.23
m^3 CH_4·d^{-1} (500 PE)	9.60	34.03
m^3 CH_4·d^{-1} (600 PE)	11.52	40.84
If biogas is 86% CH_4, then total m^3 biogas·d^{-1} (500 PE)	11.16	39.57

Table 6 Mean biogas and methane yield from the Belmont Valley WWTW pilot-scale IAPS in-pond digester.

Biogas (m^3·d^{-1})	CH_4 (kg·d^{-1})	Energy Yield GJ·y^{-1}
7.54 ± 2.36	5.12 ± 1.60	94

More detailed studies on the co-digestion of algae biomass and domestic waste showed that whole, untreated algae biomass increased methane output by 200% whereas further fracturing of the biomass either by sonication alone or freeze-thaw followed by sonication increased methane yields by 500 and 650% respectively. This indicates that co-digestion of algae biomass on site can increase substantially the biogas yield and energy stream to amounts in excess of an already impressive 94 GJ·y^{-1}.

Facultative ponds consist of an aerobic, anoxic and an anaerobic layer (Charlton, 1997). Odour control, nutrient, and COD and BOD removal occur in this pond through a mutualistic relationship that exists between aerobic and anaerobic microorganisms (Muga and Mihelcic, 2008). Algae present in the aerobic layer of the pond and release of oxygen into this layer rendering it toxic to anaerobes, aerobic microorganisms utilize this oxygen to break down organic compounds to generate CO_2 which is used by the algae

(Kothandaraman and Evans, 1972) and the biochemistry involved in the AFP is shown schematically as follows:

Photosynthesis:	$CO_2 + H_2O \leftrightarrow CH_2O + O_2$
Aerobic oxidation:	$CH_2O + O_2 \leftrightarrow CO_2 + H_2O$
Organic acid formation:	$2CH_2O \leftrightarrow CH_3COOH$
Methanogenesis:	$CH_3COOH \leftrightarrow CH_4 + CO_2$
	$CO_2 + 4H_2 \leftrightarrow CH_4 + 2H_2$
Heterotrophic nitrification:	Fixed N $\leftrightarrow NO_3^-$
De-nitrification:	$2NO_3^- + 3CH_2O \leftrightarrow N_2 + 3CO_2 + 3H_2O$

Oxygen does not penetrate the entire depth of the system (Nelson et al., 2004). Turbidity, due to suspended matter, ensures that light does not penetrate to the base of the system (Kayombo et al., 2002), while the activity of aerobic microorganisms ensures that oxygen is used up in the top layer of the pond (Charlton, 1997). Thus, there is temperature, O_2, light, CO_2 and pH stratification within facultative ponds (Charlton, 1997; Kayombo et al., 2000; Nelson et al., 2004). As a consequence, a sludge layer develops at the base of this pond where anaerobic conditions dominate and the nutrients produced by anaerobic digestion are re-suspended and utilized by microorganisms in the aerobic layer (Banat et al., 1990; Green et al., 1995). Nutrient removal in AFPs can be as much as 70% and the oxygen-saturated surface layer of the pond facilitates the oxidizing of any H_2S released by the sulfate-reducing bacteria at the bottom of these ponds, eliminating its release to the atmosphere (Banat et al., 1990). Analysis of the gases collected using an inverted cone over the surface of the AFP showed that no H_2S was released. Detention time and influent BOD and COD concentrations significantly affect the performance of AFPs. As demonstrated by Esen et al. (1992) BOD removal efficiency increases continuously with increasing detention time up to 20-30 days. Thereafter, there is little change. By comparison, COD removal efficiency is at a minimum and increases linearly only after 27 days suggesting that both BOD and COD should be considered as part of any assessment of AFP efficiency. Generally this type of pond is utilized to remediate domestic wastewater but is also capable of treating industrially generated effluents (Green et al., 1995, Muga and Mihelcic, 2008) although water quality and nutrient content remain a consequence of the composition of the influent (Amahmid et al., 2002).

High Rate Algae Oxidation Pond – Oswald developed the HRAOP for combined wastewater treatment and nutrient recovery by algae (Oswald et al., 1955; Golueke et al., 1957, Oswald et al., 1957, Golueke et al., 1959). As illustrated in Figure 4, algae photosynthesis in the HRAOPs supplies aerobic bacteria with oxygen while bacterial oxidation produces CO_2, ammonia and phosphate which is assimilated by algae into carbohydrates, lipids, proteins and other organic compounds (Oswald et al., 1957). It is the mutualism between bacteria and uni- and/or multicellular prokaryotic or eukaryotic algae in HRAOPs that results in nutrient abstraction and wastewater remediation (Kayombo et al., 2002; Craggs et al., 2004). Raceways or HRAOPs typically consist of independent closed-loop recirculation channels in which paddle wheel-generated flow is guided around bends by baffles placed in the flow channel; such systems can yield productivities of greater than 10 g ash-free dry weight.m^{-2}.d^{-1} (Sheehan et al., 1998) although, yield does vary due to fluctuations in temperature and light intensity (Oswald et al., 1955; Incropera and Thomas, 1978; Tadesse et al., 2004; Sutherland et al., 2014a; 2014b). Engineering design and operating procedures for unmixed ponds and HRAOPs have been widely studied (Borowitzka, 2005; Greenwell et al., 2010; Rogers et al., 2014). Nutrient removal efficiency in HRAOPs is dependent on algae photosynthesis and water treatment efficiency can thus be measured as an increase in algae biomass (Oswald et al., 1955). To ensure best possible photosynthetic rates, shallow water depths of 0.2-0.3 m are typically used while surface area ranges between 0.5 and 1 ha (Oswald et al., 1953b; Oswald et al., 1953a; Greenwell et al., 2010; Rogers et al., 2014).

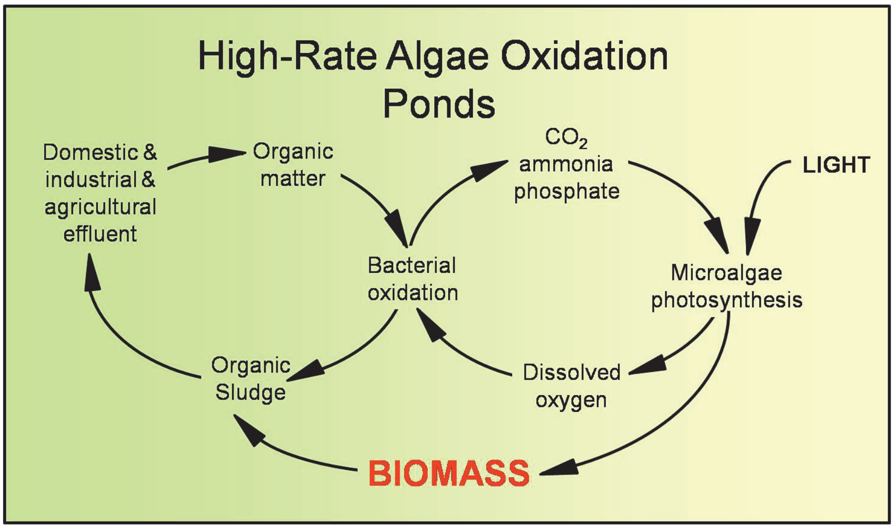

Figure 4 Mutualism between algae and aerobic bacteria in high rate oxidation ponds (Oswald et al., 1955).

Mixing, or turbulent flow, is essential to maintain optimum conditions for maximum production of microalgae in HRAOPs. Apart from preventing thermal and oxygen stratification, paddlewheel mixing maintains the surface velocity required to keep algae and algal flocs in suspension near the surface and within the sunlight penetration depth. In full-scale HRAOPs that have long channel lengths (>500 m) and circuit times of up to 90 min (e.g. Sutherland et al., 2014a), laminar flows and dead zones are often a problem (Grobbelaar, 2010). Using computational fluid dynamic modelling to characterise the energy demand for mixing of full-scale HRAOPs of various configurations, the most energy efficient design was established as a HRAOP with a minimum of three semi-circular deflector baffles and a modified 'end of centre divider' (Sompech et al., 2014). According to the authors, this design apparently eliminates dead zones completely. Similarly, Liffman et al. (2013) demonstrated that a modified HRAOP design, termed the 'medium box' (Figure 5), in which the 'end of centre divider' is tear-shaped such that water passes through the bends with the largest possible turning circle, and at near linear velocity.

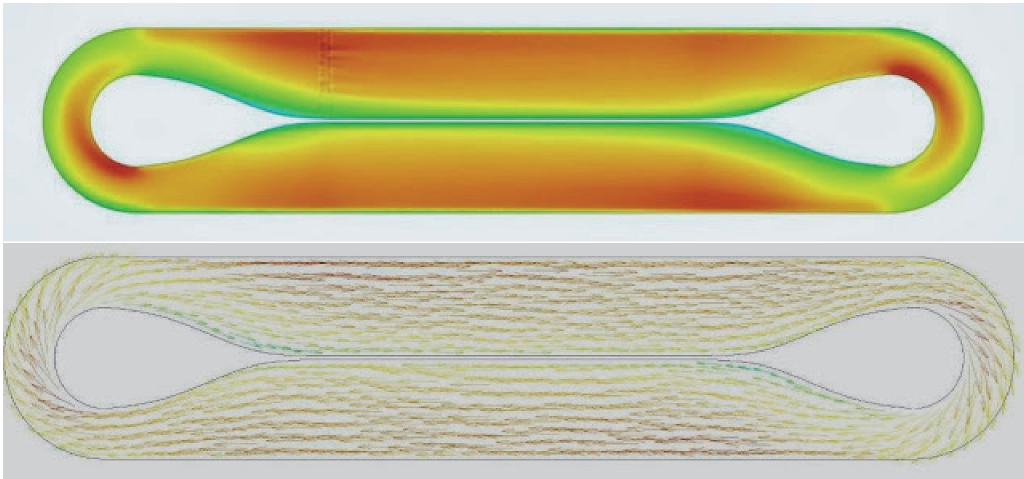

Figure 5 Normalized flow speeds in 'mixed box' designed HRAOP with flow in clockwise direction, in which red is high velocity and blue is low velocity (top). Velocity vector plot of culture medium in the HRAOP circulating in clockwise direction (bottom).

Both mixing and aspect ratio of the HRAOP affect algae productivity and hence water treatment efficiency. It was recently shown that increasing the mixing frequency (i.e. turbulent flow) increases algal biomass significantly for colonial species but not for single celled species. Furthermore, efficiency of NH_4-N uptake by all species was more efficient as mixing frequency increased (Sutherland et al., 2014d). Increased mixing also supported larger colonies with improved harvestability by gravity settling but at the expense of efficient light absorption and maximum rate of photosynthesis. Furthermore, using particle-tracing methods, Ali et al. (2015) showed that HRAOPs with a small aspect ratio were better suited to high algae productivity. Together, these findings support observations that show photosynthetic efficiency of algae is reduced where biomass production surface area is large and turbulent flow diminished (Simionato et al., 2013; Perin et al., 2014).

For HRAOPs, the driving systems required to circulate water through the raceways contribute most to energy consumption (Pate et al., 2011). The power installed to drive paddlewheel mixing is a function of raceway length, wetted area, method of construction and channel velocity. Remember; friction increases as the square of the velocity increases. For raceway mixing the paddlewheel is, possibly, the most efficient means of consistently maintaining channel velocity (Rappaport et al., 1976; Ahmad et al., 1988; Moulick et al., 2002). The paddlewheel is a pump and as such power must be applied to overcome the static head required to overcome the frictional head loss in the raceway. Design and construction of the volute for the pump (paddlewheel) is of utmost importance. The closer the tolerances, the more efficient the pump becomes. Unfortunately, with such close tolerances, foreign objects and especially stones present a problem in paddlewheel-driven HRAOPs particularly where wastewater is concerned. Even so, accurate grouting is required to achieve the required efficiency and is done by forming the radius of the sump with the wheel. By turning the wheel in the concrete before it sets then raising the assembly a few millimetres. This is done by means of jacking bolts, which allows the clearance between the wheel blade and the sump radius. As the static head in front of the paddlewheel is higher than that behind, the water will tend to slip backwards. To help overcome this either a multi-blade paddlewheel configuration or a propeller is used (Chiaramonti et al., 2012). Typically a paddlewheel with either a 6- or 8-blade wheel is used which reduces the shock on the drive and mounting assembly, compared to a wheel of fewer blades. Increasing blade number becomes impractical and does not significantly increase efficiency. Indeed, the greater the interaction area between blades and fluid the lower the paddle wheel efficiency and more shaft power consumption (Li et al., 2014).

UV disinfection is another prominent aspect of HRAOP utilisation in wastewater treatment and results in an effluent that is free of pathogens (Oswald et al., 1957; Oswald et al., 1955). In addition, the DO concentration in HRAOP can reach super saturation at 20 $mg \cdot \ell^{-1}$ (Park et al., 2011). Furthermore, domestic wastewater contains only half the carbon required to remove nitrogen by assimilation into algae biomass, specifically a ratio of 3:1 carbon to nitrogen in domestic wastewater in comparison to 6:1 in algae biomass (Park et al., 2011b). The carbon limitation is further aggravated by the daylight shift which elevates the pH of the system (Mashauri and Kayombo, 2002; Kayombo et al., 2003). This occurs when carbonates are removed by algae and hydroxyl ions are released (Craggs et al., 2004; Tadesse et al., 2004) as follows:

$$2HCO_3^- \leftrightarrow CO_3^{2-} + H_2O \leftrightarrow CO_2$$

$$CO_3^{2-} + H_2O \leftrightarrow 2OH^- + CO_2$$

The resultant CO_2 is fixed by algae and the hydroxyl ions accumulate to raise the pH above 10, which is lethal to faecal bacteria (Pearson et al., 1987; Green et al., 1995). High levels of DO (due to algal photosynthesis) and high visible light intensities, in the presence of dissolved humic substances are equally fatal to faecal bacteria (Curtis et al., 1992). Thus, the combination of high light intensity and elevated oxygen act as the major disinfection agents.

Water management procedures vary according to the purpose and intensity of operation of HRAOPs. For wastewater treatment, consortia of native algae are reported to achieve >96% nutrient removal with biomass production of ~9-17 tons.ha^{-1}.y^{-1} (Chinnasamy et al., 2010); high nutrient loading is reported to increase productivity and nutrient removal efficiency but does so at the expense of final water quality (Sutherland et al., 2014c); both biodegradation and photodegradation in HRAOPs appear to be important xenobiotic removal pathways (Matamoros et al., 2015); and recycling of harvested algae back to the HRAOPs maintains readily settleable species at >90% and improves removal of algae from the treated water (Park et al., 2011a). Even so, microalgae growth and nutrient assimilation in HRAOPs is constrained during wastewater treatment, particularly in spring and summer, due to carbon limitation (Sutherland et al., 2014a) which may be offset by direct CO_2 addition under automated pH-stat control (Park et al., 2011b; Sutherland et al., 2014b).

Algae Settling Pond – Biomass generated in the HRAOP must be removed and in conventionally designed IAPS this is usually achieved using an ASP. Algae from the HRAOP, due to paddlewheel mixing, flocculate and settle rapidly in the ASP. Settled solids or algae biomass do not decompose if harvested regularly. A hydraulic retention of 0.5 d is sufficient for adequate settling and yields slurry of typically 2.5-4 % solids.

2.4 The Belmont Valley WWTW IAPS

Rhodes University together with the WRC commissioned and built an Oswald-designed IAPS, specifically for South African conditions (Rose et al., 2002a; Rose et al., 2007; Mambo et al., 2014b). This pilot scale demonstration is located at the Belmont Valley WWTW where it receives and treats a constant supply of raw domestic sewage sourced from a splitter box immediately after the inlet works. A process flow illustrating the operating configuration of the Belmont Valley WWTW IAPS is presented in Figure 6. The system has been in continuous use since 1996 and receives ~75 kℓ·d^{-1} of raw municipal sewage. It is apparent from the schematic (Fig. 6) that any partially treated water and/or tertiary treated water (i.e. suitable for reclamation) is returned to the Belmont Valley WWTW. Thus, and due to research needs and logistical issues, no treated or untreated water from this demonstration system is discharged to environment.

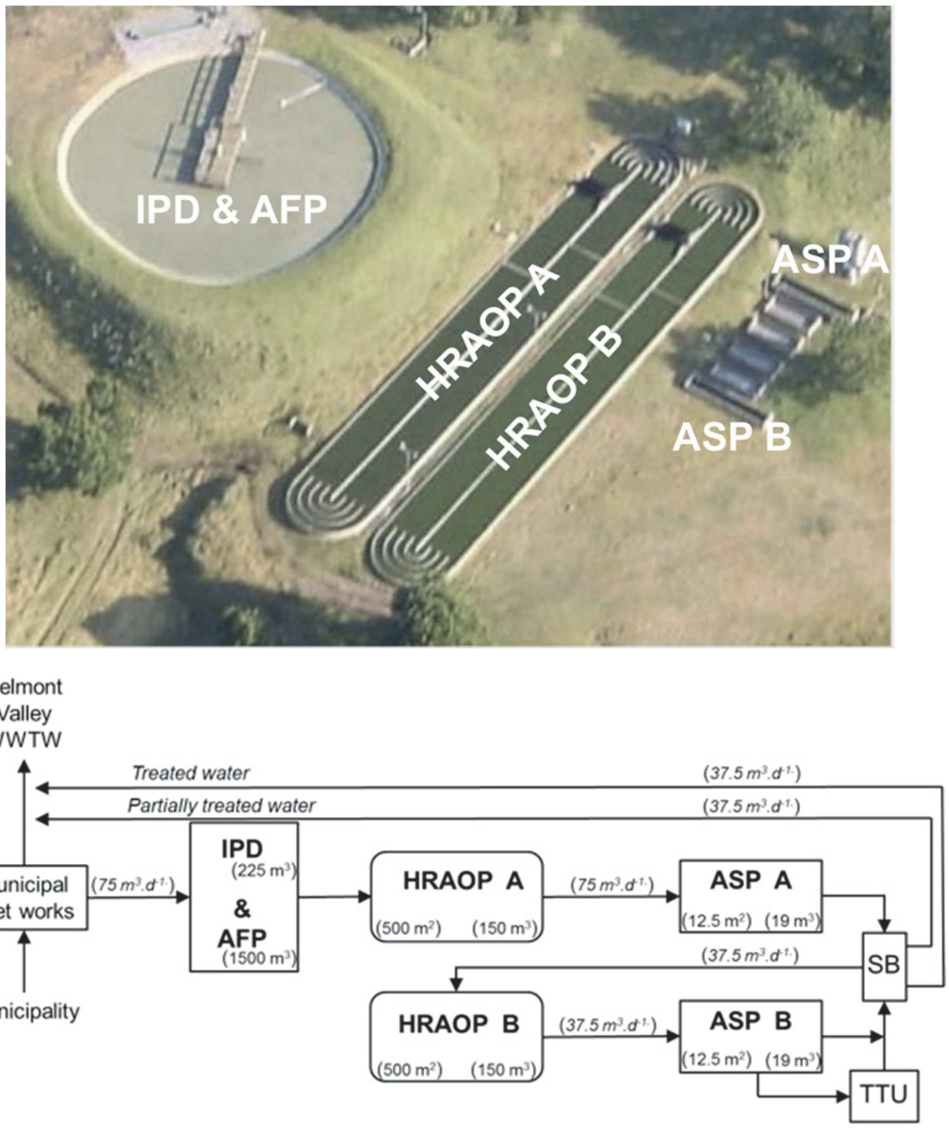

Figure 6 Aerial view (top) and process flow (bottom) of the pilot-scale IAPS located at the Belmont Valley WWTW. System receives raw municipal sewage, screened for the removal of plastics via a grit-detritus channel. Pond and reactor surface area, volume and flow rates are shown in parentheses. Effluent enters at the bottom of the IPD some 6 m below water level. SB = splitter box; TTU = tertiary treatment unit. Note: 1 m^3 = 1 kℓ.

While this IAPS is a passive, sequential, sewage treatment plant (STP) that functions virtually in perpetuity and without any need for faecal sludge handling, the technology has yet to be adopted by the wastewater sector for implementation nationally. Reasons for the status quo are unclear but in part due to, ignorance about the technology, the perception that the final effluent generated does not comply with discharge standards, a perceived skills shortage, and an apparent lack of will to address sewage treatment management issues due mainly to the high costs of infrastructure repair and upgrade. This is in direct contrast with global sentiment to IAPS technology which is currently in use in the U.S.A., India, New Zealand and many other countries (Table 4).

Criteria for the Belmont Valley IAPS for the treatment of municipal sewage were as follows: capacity of 500 PE based on an average water consumption and disposal per capita of 150 $\ell\cdot d^{-1}$, the design flow was calculated at 75 $k\ell\cdot d^{-1}$. With an ultimate BOD_{ult} assumed to be 80 $g\cdot PE^{-1}\cdot d^{-1}$, and an organic load to the system of 40 $kg\cdot d^{-1}$ was determined (Rose et al., 2002a). It was postulated that the resultant treated effluent would comply with environmental discharge (Rose et al., 2007).

To eliminate the need for faecal sludge handling and disposal and to ensure complete breakdown of biodegradable solids, the volumetric capacity of the IPD was designed at 0.45 m^3 per capita, rather than the more conventional 0.3 m^3 per capita. Thus, raw sewage, after screening, enters at least 6 m below water level near the bottom of the IPD. A "berm" wall (1.5 m below water level) extends the IPD 1.5 m above the floor of the AFP to direct gas flow and prevent any ingress of oxygen rich water. An upflow velocity of 1.0-1.5 $m\cdot d^{-1}$ in the IPD was estimated as sufficient to allow solids to settle and parasites (e.g. helminth ova, worms, etc.) to remain in the sludge layer. With a volumetric capacity of 225 m^3 hydraulic retention time (HRT) in the IPD, and at the designed flow, is 3 d. By reducing influent flow rate or increasing IPD volume digestion was managed to near completion. The overlying water of the AFP contains an oxygen rich layer near the surface and is populated by algae that sequester gases produced as a consequence of anaerobic digestion. Even so, large emissions of CO_2 and CH_4 from the surface area can be expected and methane production rates of 0.17 kg $CH_4\cdot kg^{-1}$ BOD_{waste} have been modelled for anaerobic ponds fed municipal waste (DeGarie et al., 2000; van der Steen et al., 2003). Furthermore, about 3.3 $k\ell$ $PE^{-1}\cdot y^{-1}$ of CH_4 is produced in real scale ponds at a daily sewage production of 100 g COD $PE^{-1}\cdot d^{-1}$. While CO_2 is the best studied and most known, CH_4, which constitutes up to 75 % of the total gas emitted during anaerobic wastewater treatment, is 25 times more potent as a greenhouse gas (Forster et al., 2007; Daelman et al., 2012). Similarly nitrous oxide, although emitted in relatively low amounts, is 300 times more damaging than CO_2 (Daelman et al., 2012; Strutt et al., 2008). While, harvesting and/or recycle of these gases can potentially avert any detrimental impact (Oswald, 1995; Green et al., 1995a), it is now understood that methane oxidizing bacteria (MOB), a relatively common group of bacteria capable of utilizing CH_4 as their sole carbon and energy source if sufficient O_2 is present, utilize in-pond algae-derived O_2 to consume much of the emitted CH_4 before it reaches the atmosphere (van der Ha et al., 2011; unpublished data). Even so, only part of the CH_4 produced by methanogens is consumed by MOB before it reaches the atmosphere and elementary extrapolation of measured pond emissions still show a total loss of about 3000 ℓ $CH_4 \cdot y^{-1}$, equalling a yearly contribution of 55 ton CO_2-equivalents$\cdot y^{-1}$ or an emission of 0.98 kg $CH_4 \cdot y^{-1}$ per inhabitant.

For the AFP, HRT was determined using a temperature dependant first order decay rate for residual BOD (Rose et al., 2002a) and for the Belmont Valley IAPS this is 20 days. Design of the outer reaches of the AFP was also important to limit, if not prevent, short circuiting from the IPD overflow to the AFP outlet, which is typically 0.5-1.0 m below water surface to avoid skimming off of floating material and algae.

Effluent from the AFP is gravity fed to the first of two HRAOPs connected in series and with a combined HRT of 6 days, is subjected to photosynthetic oxygenation. Excess oxygen is consumed by heterotrophic bacteria to degrade dissolved organic matter for assimilation by the algae biomass. At elevated pH other mechanisms also operate to reduce nutrient load (e.g. phosphate precipitation and ammonia volatilization) and most studies on the role of algae in HRAOPs point out that this indirect nutrient removal is often more important than direct uptake (Larsdotter, 2006).

Mixing, or turbulent flow and a HRAOP channel velocity of 50 $mm \cdot s^{-1}$ is sufficient to prevent algae settling and eliminate stratification but is very difficult to maintain due to frictional losses, especially in the bends. Thus, a linear velocity of 200-300 $mm \cdot s^{-1}$ is routinely used although this increases the energy demand. Power to drive the paddlewheels is a function of raceway length, wetted area, method of construction and channel velocity. As already mentioned, design and construction of the volute and paddlewheel is important. An 8-blade paddle configuration is used to mix algae in the HRAOPs and appears sufficient to reduce shock on the drive and mounting assembly (Rose et al., 2002a). Even flow velocity around the 180 degree bends is achieved using flow rectifiers (see Fig. 5). Various flow rectifiers were tested during commissioning of this pilot demonstration (e.g. teardrops, reverse teardrops, etc.) and the method determined as most successful is concentric semicircle walls spaced 1 m apart. Additional detail, including engineering diagrams and aspects of the design and operating configuration of this IAPS are described elsewhere (Rose et al., 2002a; 2007).

Algae biomass produced in the HRAOPs is removed by passage through two ASPs operated in series, each with HRT of 0.5 d. Unfortunately, algae production in the Belmont Valley WWTW IAPS has not been fully explored. Nevertheless, a similar Oswald-designed IAPS was shown to perform satisfactorily when implemented under desert conditions in Kuwait and provide a treated effluent with less than 20 mg·ℓ$^{-1}$ BOD, 130 mg·ℓ$^{-1}$ COD, 40 mg·ℓ$^{-1}$ TKN, and 25 mg·ℓ$^{-1}$ NH_4-N (Puskas et al., 1991). In this IAPS, algae production of 300-400 (summer) and 200-300 kg·ha^{-1}·d^{-1} (winter) was reported (Al-Shayji et al., 1994) and production was increased at shorter HRT, at HRAOP depths of 0.3-0.45 m, and with increasing nutrient load up to 40 kg (Banat et al., 1990).

The pilot scale IAPS commissioned at the Belmont Valley WWTW in Grahamstown in 1996 was to demonstrate and evaluate the performance of the system for deployment as a 'green' technology to address issues such as climate change and sustainable development (Rose et al., 2002a). The costs associated with construction and operation (including maintenance) of the IAPS were, at the time, viewed as highly competitive, job creation was evidently possible (Rose et al., 2002a, Harun, 2010) while improved access to clean water was and still is understood to stimulate social and economic development (Oswald, 1995). An extended study to evaluate operation and performance of this IAPS as a full municipal STP for South Africa was published in 2007 and revealed the following;

- The system did not achieve the 75 mg·ℓ$^{-1}$ discharge standard for CODt,
- 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).

It is difficult to reference the data obtained from the IAPS at the Belmont Valley WWTW against results for other systems due in part to incompleteness in mass balances and the apparent lack of empirical values to describe the nutrient load in both the raw sewage influent as well as the residual nutrient load in the final effluent (i.e. discharged from the final ASP) following treatment by operation of the full system. Also, these authors seemed more concerned with the performance of each of the component parts of the IAPS and little emphasis was placed on IAPS as a complete system for municipal sewage treatment. Consequently, much of the data is derived from operation of only a single HRAOP and attempts to develop this further as an 'I-HRAP' for use as a standalone tertiary treatment unit.

A further concern on the implementation and performance of this IAPS for sewage treatment is the absence of a final polishing step. As discussed above, the original AIWPS® designs always included a polishing step or tertiary treatment process comprising of either a MP series or similar to ensure that water quality of the final effluent meets the criteria for discharge, except total coliforms which requires additional disinfection (e.g. chlorination, ozonation, UV-radiation, etc.). In fact, a recent report on the operation of hectare-scale HRAOPs for enhanced sewage treatment strongly advocates that additional treatment of the ASP effluent is required to meet specific discharge standards (Craggs et al., 2012). These authors recommend the inclusion of one or a combination of 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. Without a final polishing step, and as demonstrated in other studies, the COD of the final effluent remains elevated resulting in the potential that discharged water from an IAPS will be detrimental to any receiving water bodies (Park and Craggs, 2011). Thus, it is surprising that the model proposed by Rose et al. (2007) to link water treatment and job creation initiatives "*which is dependent on the system to produce a water quality that at least meets DWA irrigation water discharge standards*" was based on a 'secondary treated' water. Clearly, any considered implementation of IAPS for treatment of municipal sewage must include in the process design, a final effluent polishing process.

2.5 IAPS Kinetic Parameters

In order to test the kinetic parameters as reported for the Belmont Valley IAPS (Rose et al., 2002a), the design criteria were re-evaluated using real values obtained by analysing the raw sewage influent entering the system (Rose et al., 2007) and the results are shown in Table 7. Actual strength of the raw sewage as measured over the course of two years revealed a mean organic loading (i.e. COD_{max}) of 1800 $mg \cdot \ell^{-1}$ and a maximum BOD of 1080 $mg \cdot \ell^{-1}$. These values are almost twice the initial design parameters and indicate that the strength of municipal sewage entering from the Belmont Valley WWTW IAPS, should be reduced by 50% in order achieve the correct loading rate (i.e. BOD_{ult}) to derive the required efficiency.

To achieve the correct organic load to the system, partially treated wastewater from HRAOP *A* (see Fig. 5), equivalent to 37.5 $k\ell \cdot d^{-1}$, must be redirected away from the Belmont Valley WWTW inlet works and to the IAPS inlet; and not to the AFP as indicated (Rose et al., 2002a). This would ensure maintenance of hydraulic load and reduce the organic load to potentially increase overall process efficiency. Recirculation of water from the HRAOPs back to the AFP was suggested as a possibility but only when water temperature in the HRAOPs exceeds water temperature in the AFP.

Table 7 Effect of loading rate on the kinetic parameters for IAPS implementation. Numbers were derived experimentally and used to test the model using the documented design parameters reported by Rose et al. (2002a).

Parameter	Design	Actual
Volume ($k\ell \cdot d^{-1}$)		
PE equiv	500	500
Vol per $PE \cdot d^{-1}$	0.15	0.15
Flow	75	75
Strength ($mg \cdot \ell^{-1}$)		
BOD	800	1080
COD	1000	1800
TKN	-	128
P_{total}	-	15
Loading ($kg \cdot d^{-1}$)		
BOD	**40**	**81**
COD	-	135
TKN	-	9.6
P_{total}	-	1.125

Evaluation of the design parameters for the IPD and AFP revealed the analysis presented in Table 8. Results show that BOD loading of the in-pond digester is close to 0.4. Values above 0.4 are indicative of malodours, which would be a distinct disadvantage in deployment of the IAPS technology for treatment of municipal sewage. Second, closer scrutiny of the parameters for the AFP indicated that with an effluent BOD at 80 $mg \cdot \ell^{-1}$ and according to first order kinetics, pond surface area and HRT should be 1389 m^2 and 33.3 d respectively which contrasts with the design parameters used of 840 m^2 and 20 d (Rose et al., 2002a).

Using the McGarry model (McGarry and Pescod, 1970) however, an AFP area of 382.8 m^2 and a HRT of 9.2 d were obtained which is similar to the design specifications of the Belmont Valley IAPS suggesting that the AFP parameters are accordingly sufficient for a

500 PE IAPS with an influent loading of 40 kg·d^{-1} BOD and indeed, the actual measured BOD of the outflow was between 300 and 444 mg·ℓ$^{-1}$ (cf. estimated BOD of outflow of 486 mg·ℓ$^{-1}$).

Based on measured values for NO_3-N + NH_4-N and PO_4-P in the AFP effluent of 80 and 15 mg·ℓ$^{-1}$ respectively, the specific loading of N and P to the HRAOPs were determined to be 60 and 11.25 kg·ha^{-1}. Using a raw sewage TKN value of 128 mg·ℓ$^{-1}$ (Table 7), the N and P loads were determined to be 6.24 and 0.96 kg·d^{-1} with a required HRAOP surface area for abstraction of these nutrients in the range 0.085-0.104 ha which compares favourably with the design specifications of the Belmont Valley IAPS HRAOPs. These have combined total surface area of 1000 m^2 (Rose et al., 2002a).

Table 8 Design parameters for the Belmont Valley IAPS In-pond digester and advanced facultative pond based on the model developed to determine the kinetic parameters for IAPS implementation.

Component Parameter	Value	Units	Comments
In-Pond Digester			
Optimum retention time	3	d	
Volume needed	225	kℓ	
Depth	5	m	
Area	45	m^2	
Diameter	7.5	m	
BOD loading	0.36	kgBOD· kℓ$^{-1}$·d^{-1}	Odour problem above 0.4
Estimated BOD reduction	55	%	
Estimated BOD of outflow	486	mg.ℓ$^{-1}$	
Advanced Facultative Pond (PFP)			
Depth	1.8	m	
Aspect	3		
Minimum temp	12	°C	For first order kinetics approach
Minimum temp	53.6	°F	For McGarry model
Effluent BOD	80	mg·ℓ$^{-1}$	
First Order Kinetics Approach			
Area	1389	m^2	First order kinetics eq 7.13
Diameter	42.04	m	
BOD surface loading	0.026	kgBOD·m^{-2}·d^{-1}	
HRT	33.33	d	Much longer than current 20 d
McGarry Approach			
Area	382.87	m^2	McGarry eq 7.17 (valid 15-30°C)
Diameter	22.08	m	
BOD surface loading	0.0952	kgBOD·m^{-2}·d^{-1}	
HRT	9.20	d	Close to Belmont Valley IAPS

The aforementioned exercise set out to establish parameters, based on the original design specifications of the Belmont Valley IAPS, for use in developing a model to derive draft kinetic parameters for implementing HRAOPs as a remediation strategy to mitigate overloading and/or underperformance of dysfunctional WWTW. One of the outcomes of this evaluation was the realisation that the Belmont Valley IAPS process flow is incorrectly configured and that partially treated water exiting HRAOP *A* should be returned to the IAPS inlet (not the Belmont Valley WWTW inlet works) to offset the very high influent BOD_{ult} which, at 81 $kg \cdot d^{-1}$, is twice the design capacity. It is this oversight which has undoubtedly resulted in substantial sludge accumulation in both the IPD and AFP (arrows, Fig. 7), revealed following drainage of the AFP to evaluate sludge build up, and which is affecting both biogas production (by the IPD) and quality of final treated water.

Figure 7 Evidence for sludge accumulation in the IPD (arrows; right panel) and the AFP (arrows; left panel) following drainage of water from the AFP.

2.6 Productivity, Population Dynamic and Biomass

Algae productivity in open ponds and HRAOPs has typically been estimated daily per unit area and expressed as g $DW \cdot m^{-2} \cdot d^{-1}$ or g $DW \cdot ha^{-1} \cdot d^{-1}$, i.e. aerial density (Richmond, 2008).

More recently, studies on the recycling of algae to improve species control and harvest efficiency from HRAOPs has led to a re-evaluation of the procedures used to calculate productivity (Park et al., 2011a). These authors argue that algae biovolume is a more accurate measure of relative algal dominance (%) than cell counts because not all algal cells are the same size. Based on geometrical equations to calculate the biovolume of algal species of different shapes from microscopically measured linear dimensions developed by Hillebrand et al. (1999) and Vadrucci et al. (2007), these authors derived equations for five of the most dominant algae species commonly found in wastewater HRAOPs including *Pediastrum* sp., *Desmodesmus* sp., *Micractinium* sp., *Dictyosphaerium* sp., *Chlorella* sp., and *Thalassiosira* sp. Measurement of biovolume assumes significance when accounting for population dynamic as these changes affect not only productivity in the mass culture of algae but nutrient abstraction efficiency. For example, for the Ruakura Research Centre, Hamilton, New Zealand HRAOPs (37°47'S, 175°19'E) *Pediastrum* sp. was replaced by *Micractinium* sp. in October 2009, which was replaced by *Pediastrum* sp. in December 2009, which was replaced by the unicellular diatom *Thalassiosira* sp. in March 2010, which was then replaced by *Dictyosphaerium* sp. in April 2010.

Similarly, the Belmont Valley IAPS HRAOPs (33°19' S, 26°33'E) show continual shifts in algae population dynamic with *Pediastrum* sp. replaced by *Micractinium* sp. in October 2012, which was replaced by *Pediastrum* sp. in December 2012, which was replaced by the unicellular diatom *Cyclotella* sp. in March 2013, which was replaced by *Pediastrum* sp. in December 2013, and then by *Dictyosphaerium* sp. in January 2014. Similar changes in the dominant algae (including *Dictyosphaerium* sp., *Chlorella* sp., *Micractinium* sp., and *Desmodesmus* sp.) of a small-scale pilot wastewater treatment HRAOP in Spain have also been reported (Garciá et al., 2000). An example of the microbial dynamic in the HRAOPs showing the shift from dominance by the green alga *Pediastrum* in favour of the diatom *Cyclotella* is presented in Figure 8.

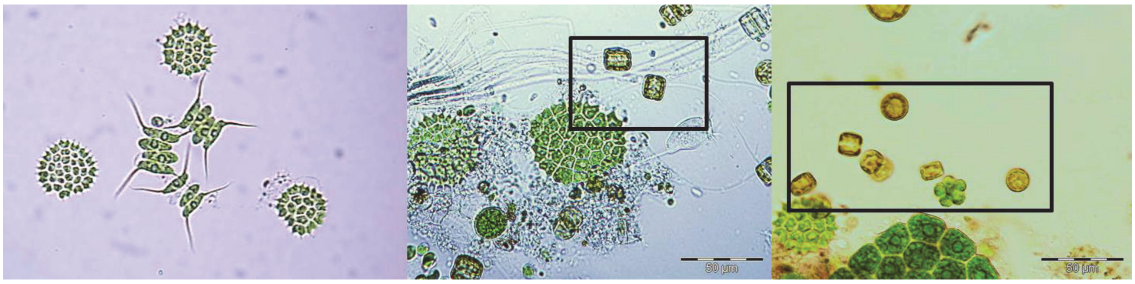

Figure 8 Transition of species dominance in the HRAOPs from *Pediastrum/ Scenedesmus* to *Cyclotella* (boxes) between March and April/May 2013.

Shifts in population composition, structure and species dominance are also related to changes in both operational and environmental conditions (in particular variations in solar radiation and pond water temperature) that impact biocatalyst selection, succession and co-existence (Benemann et al., 1977; Oswald, 1988). Changes in biocatalyst dynamic also impact efficiency of the water treatment process. It has been hypothesized that nitrogen-replete diatoms release NO_2^-, NH_4^+ or dissolved organic nitrogen following rapid increases in irradiance and consequently an increase in cellular electron energy (Lomas and Gilbert, 1999). Similarly, a decrease in temperature, due to the temperature dependency of biosynthetic enzymes, increases cellular energy. Indeed, release rates of NH_4^+ under increased irradiance were shown to be nearly five-fold greater than release rates at the growth irradiance, and to account for 84% of the NO_3^- uptake rate (Lomas et al., 2000). Thus, it is perhaps not surprising that in HRAOPs populated with a dominant species such as the diatom *Cyclotella*, levels of ammonium-N were in amounts 2.5 times above the general limit (Table 9). Elevated NH_4-N in the treated water may also have arisen due to the lower ambient temperatures experienced during the sampling period. Nitrifiers are characterised by having a low range of temperature tolerance, from about 8-30°C and they exhibit a very low metabolic rate below 15-20ºC (Davies, 2005).

Table 9 Effect of change in algae population dynamic on water quality of the final effluent from the Belmont Valley WWTW IAPS treating municipal sewage. Dominant biocatalyst was the diatom *Cyclotella*. Data were from samples collected at regular intervals between 1 March and 20 November 2013 and are presented as the mean ± SD.

Parameter (unit)	
pH	8.8 ± 0.4
Dissolved oxygen (mg·ℓ$^{-1}$)	5.5 ± 1.3
Electrical conductivity (mS·m^{-1})	116.4 ± 18.6
Nitrate/nitrite-N (mg·ℓ$^{-1}$)	7.0 ± 4.4
Ammonium-N (mg·ℓ$^{-1}$)	7.4 ± 1.1
Phosphate-P (mg·ℓ$^{-1}$)	10.2 ± 3.6
Chemical oxygen demand (mg·ℓ$^{-1}$)	97.7 ± 15.7
Total suspended solids (mg·ℓ$^{-1}$)	35.0 ± 12.3
Total coliforms (cfu·100 mℓ$^{-1}$)	>1000

Notwithstanding changes in algae population dynamic, approaches such as aerial density, volumetric mass density, and measurement of biovolume for the determination of HRAOP biomass assume that microalgae (unicellular and colonial) are the major if not the only catalysts involved in nutrient abstraction. Consequently, the contribution of bacteria to nutrient removal process during water treatment is obscured and even overlooked.

It is perhaps not surprising therefore, that early work on algae production in HRAOPs associated with wastewater treatment attempted to account for the contribution of both algae and bacteria when determining overall productivity (Banat et al., 1990). To achieve this, algae productivity reported as kg·ha^{-1}·d^{-1} was calculated by including the function C_s, which is the algae biomass in mg·ℓ$^{-1}$, determined by multiplying the total suspended solids (TSS; quantified as described by APHA, 2008) by a factor, *n*, or the algae ratio in TSS. Thus, HRAOP productivity was quantified per unit pond surface as,

$$P=10^{d}/_{t}\cdot n\cdot SS$$

where: P=algae productivity (kg·ha^{-1}·d^{-1}); D=pond depth (m); T=detention time (d); SS= total suspended solid (mg·ℓ$^{-1}$); and *n*=factor, expressing the algae ratio in the suspended solids which for near pure cultures is 0.9-1.0 (Al-Shayji et al., 1994).

By filtering and drying a sample of suspended solids, a measure of biomass is obtained which is referred to as the mixed liquor suspended solids (MLSS, mg·ℓ$^{-1}$) and usually used by process engineers to measure the bacterial biomass associated with activated sludge floc. Under some circumstances a significant proportion of the MLSS may be inorganic material. For this reason, it is often preferable to derive a weight for the organic matter alone. This is done by combusting the dried residue in a furnace at 500°C, reweighing, and obtaining the volatilised organic matter, by subtraction – referred to as the mixed liquor volatile suspended solids (MLVSS). However, even this weight is an imprecise measure of the active microbial biomass, since a significant part of the floc comprises inert organic matter.

Despite these shortcomings, MLSS is universally used in process control as a measure of biomass (Appendix A). MLSS values range from about 800-1500 $mg \cdot \ell^{-1}$ for extended-aeration and other low-rate systems, to 8000 $mg \cdot \ell^{-1}$ or more, for high-rate systems. It is also helpful to remember that higher efficiency of treatment would be achieved by increasing the MLSS, since the more organisms that are present in the mixed liquor the faster the BOD should be ingested and the water treated. Also known as the food to mass ratio, more commonly notated as the F/M ratio, MLSS is therefore an important part of the process and ensures that there is a sufficient quantity of active biomass available to consume the applied quantity of organic pollutant at any time. However, high MLSS concentrations can create problems in aeration and also in biomass settlement. Since HRAOPs function as aerators and ASP as clarifiers, it seems reasonable to suggest the use of MLSS as the most appropriate method for measurement of biomass in HRAOP wastewater treatment technology.

It is also possible to measure HRAOP productivity using a Secchi disc. A Secchi disk is a simple, standard tool used to measure water clarity. It is a 20 cm in diameter, black and white disk attached to a dowel rod, PVC pipe, rope or chain. Intervals are marked in cm on the rod, pipe, rope or chain and the disk lowered into the water while observing the depth at which it disappears. The disc is lowered further and then raised while observing the depth at which it reappears. A Secchi disk measurement (SDM) is the average of the two observations and productivity is determined based on the SDM value as follows:

$$P=2000\,{}^{d}/_{t}\cdot n\cdot SS({}^{20}/_{D}-1)$$

Where:

D = visual depth of the disk or SDM (cm) and 2 000 the light intensity ($\mu mol \cdot m^{-2} \cdot s^{-1}$).

2.7 HRAOP Kinetic Parameters

From the design parameters for IAPS (see Table 7 and 8) it is apparent that the required variables needed to derive the kinetic parameters for HRAOP implementation include influent flow ($k\ell \cdot d^{-1}$), NO_3-N + NH_4-N ($mg \cdot \ell^{-1}$) and PO_4-P ($mg \cdot \ell^{-1}$) concentration, and the production potential of the biomass used as catalyst in the HRAOPs. Using real time data for the AFP effluent from the Belmont Valley WWTW IAPS and the current flow of 75 $k\ell \cdot d^{-1}$ it was possible to predict the surface area of HRAOP required for nutrient removal as shown in Figure 9. This was achieved after development of a productivity model based on the relationship between biomass accumulation and nutrient loading (STOWA, 2010). Thus, based on climate conditions for Grahamstown, where average PAR is 87.8 $W \cdot m^{-2}$ and algae productivity of 8 g $DW \cdot m^{-2} \cdot d^{-1}$, an average productivity of 9.10×10^{-3} g $DW \cdot Wh^{-1}$ PAR was estimated. In Figure 9A, the current operating conditions of the pilot IAPS are shown and results indicate that an average total HRAOP surface area of 0.635 ha is required for removal of N and PO_4-P. This result compares favourably with the design specifications and current operating configuration, which with a total HRAOP hydraulic retention time of 6 days provides a surface area of 0.6 ha. As indicated in the model, simulation of a reduction in N in the AFP effluent from 80 to 45 $mg \cdot \ell^{-1}$ does not impact the HRAOP surface area required (Fig. 9B) whereas with an increase in NO_3-N + NH_4-N from 80 to 150 $mg \cdot \ell^{-1}$ a HRAOP surface area of almost 0.8 ha is required (Fig 9C).

Nitrogen content of algae varies between 4 and 8 % (w/w) depending on physiological state and nutrient load whereas PO_4-P content is much lower and that of S is about 0.5 % (w/w). Provision of S is generally not a problem whereas N and PO_4-P are typically limiting. Algae contains approximately 7 times more intracellular N than PO_4-P and when this ratio is below 7, nitrogen limitation is experienced whereas above 7, PO_4-P limitation occurs. The AFP of the Belmont Valley IAPS produces an effluent with N:PO_4-P between 4.1 and 7.5 (mean 5.6) and the HRAOP biomass contains between 5 and 6.5% (w/w) N. Most algae adjust to lower nutrient levels and only when intracellular levels approach 3% (w/w) for N and 0.2-0.4% (w/w) for PO_4-P is there true limitation. Furthermore, under limiting PO_4-P algae continue to extract most of the available N while in high N effluent with reduced PO_4-P, algae lose the ability to assimilate N efficiently. Similarly, at high PO_4-P and with reduced N, PO_4-P removal efficiency declines.

A

Flow (m3/day)	75	influent	
		N- NO3 + N-NH4 (mg/l)	80
Algae production (gr MLSS / Wh PAR)	9.1E-03	P-PO4 (mg/l)	15
P content algae sludge produced (%)	8%		
N content algae sludge produced (%)	1.0%		

Month	Sunlight (W PAR/m2)	Algae production (gr-DS * m-2 * d-1)	Surface required (m2) for N	Surface required for P (m2)	Surface required algae route (m2)	Surface required pH route (m2)
january	107.65	23.51	3190	4785	4785	3190
february	105.47	23.03	3256	4884	4884	3256
march	102.67	22.42	3345	5017	5017	3345
april	73.54	16.06	4670	7004	7004	4670
may	60.39	13.19	5686	8530	8530	5686
june	46.27	10.11	7422	11133	11133	7422
july	57.49	12.56	5973	8960	8960	5973
august	93.53	20.43	3672	5507	5507	3672
september	90.61	19.79	3790	5685	5685	3790
october	96.94	21.17	3542	5314	5314	3542
november	112.90	24.66	3042	4563	4563	3042
december	106.17	23.19	3234	4852	4852	3234
Avg.	***87.80***	***19.18***	***4235***	***6353***	***6353***	***4235***

B

Flow (m3/day)	75	influent	
		N- NO3 + N-NH4 (mg/l)	45
Algae production (gr MLSS / Wh PAR)	9.1E-03	P-PO4 (mg/l)	15
P content algae sludge produced (%)	8%		
N content algae sludge produced (%)	1.0%		

Month	Sunlight (W PAR/m2)	Algae production (gr-DS * m-2 * d-1)	Surface required (m2) for N	Surface required for P (m2)	Surface required algae route (m2)	Surface required pH route (m2)
january	107.65	23.51	1794	4785	4785	1794
february	105.47	23.03	1831	4884	4884	1831
march	102.67	22.42	1881	5017	5017	1881
april	73.54	16.06	2627	7004	7004	2627
may	60.39	13.19	3199	8530	8530	3199
june	46.27	10.11	4175	11133	11133	4175
july	57.49	12.56	3360	8960	8960	3360
august	93.53	20.43	2065	5507	5507	2065
september	90.61	19.79	2132	5685	5685	2132
october	96.94	21.17	1993	5314	5314	1993
november	112.90	24.66	1711	4563	4563	1711
december	106.17	23.19	1819	4852	4852	1819
Avg.	***87.80***	***19.18***	***2382***	***6353***	***6353***	***2382***

C

Flow (m3/day)	75	influent	
		N- NO3 + N-NH4 (mg/l)	150
Algae production (gr MLSS / Wh PAR)	9.1E-03	P-PO4 (mg/l)	15
P content algae sludge produced (%)	8%		
N content algae sludge produced (%)	1.0%		

Month	Sunlight (W PAR/m2)	Algae production (gr-DS * m-2 * d-1)	Surface required (m2) for N	Surface required for P (m2)	Surface required algae route (m2)	Surface required pH route (m2)
january	107.65	23.51	5981	4785	5981	5981
february	105.47	23.03	6105	4884	6105	6105
march	102.67	22.42	6271	5017	6271	6271
april	73.54	16.06	8756	7004	8756	8756
may	60.39	13.19	10662	8530	10662	10662
june	46.27	10.11	13916	11133	13916	13916
july	57.49	12.56	11200	8960	11200	11200
august	93.53	20.43	6884	5507	6884	6884
september	90.61	19.79	7106	5685	7106	7106
october	96.94	21.17	6642	5314	6642	6642
november	112.90	24.66	5703	4563	5703	5703
december	106.17	23.19	6065	4852	6065	6065
Avg.	***87.80***	***19.18***	***7941***	***6353***	***7941***	***7941***

Figure 9 Effect of NO_3-N+NH_4-N loading on the surface area requirement of HRAOPs for nutrient removal. A) Current IAPS operating conditions; B) reduced N loading; and C) increased N loading.

This model was also used to determine the effect of productivity on HRAOP surface area requirement, which is particularly important in regions strongly affected by season. In the winter months (April-August) much lower productivities are typical and for the Belmony Valley WWTW IAPS, HRAOP surface area requirement increases from 0.6 to 1 ha suggesting HRT closer to 10-12 in the winter months which is easily achieved by reducing the flow from 75 to 35 kℓ·d^{-1} as shown in Figure 10 or, by including a third HRAOP to provide additional surface area as flow reductions are not logical during commercial operation (see Appendix B for recommendation).

Flow (m3/day)			35		influent		
					N- NO3 + N-NH4 (mg/l)	80	
Algae production (gr MLSS / Wh PAR)			9.1E-03		P-PO4 (mg/l)	15	
P content algae sludge produced (%)			8%				
N content algae sludge produced (%)			1.0%				

Month	**Sunlight (W PAR/m2)**	**Algae production (gr-DS * m-2 * d-1)**	**Surface required (m2) for N**	**Surface required for P (m2)**	**Surface required algae route (m2)**	**Surface required pH route (m2)**
january	107.65	23.51	1489	2233	2233	1489
february	105.47	23.03	1519	2279	2279	1519
march	102.67	22.42	1561	2341	2341	1561
april	73.54	16.06	2179	3269	3269	2179
may	60.39	13.19	2654	3981	3981	2654
june	46.27	10.11	3464	5195	5195	3464
july	57.49	12.56	2788	4181	4181	2788
august	93.53	20.43	1713	2570	2570	1713
september	90.61	19.79	1769	2653	2653	1769
october	96.94	21.17	1653	2480	2480	1653
november	112.90	24.66	1419	2129	2129	1419
december	106.17	23.19	1509	2264	2264	1509
Avg.	***87.80***	***19.18***	***1976***	***2965***	***2965***	***1976***

Figure 10 Effect of influent flow on HRAOP surface area requirement for nutrient removal.

2.8 Compliance and Standards

According to the General Authorisations in terms of Section 39 of the Water Act (Republic of South Africa, *Water Act 1998*) the discharge of domestic and industrial wastewater for irrigation purposes up to 2Mℓ per day must comply with the general limit values of amongst others: faecal coliforms (per 100 mℓ) ≤ 1 000; pH 5.5-9.5; NH_4-N ≤ 3 mg·ℓ$^{-1}$; NO_3+NO_2-N ≤ 15 mg·ℓ$^{-1}$; PO_4-P ≤ 10 mg·ℓ$^{-1}$; electrical conductivity 70-150 mS·m^{-1}; COD ≤ 75 mg·ℓ$^{-1}$ and TSS ≤ 25 mg·ℓ$^{-1}$. These general authorisations apply also to the discharge of up to 2 Mℓ of wastewater on any given day into a water resource that is not a listed water resource provided the discharge complies with the general wastewater limit values as set out in Table 9.

Since commission of the Belmont Valley WWTW IAPS in 1996 there has been no need to de-sludge the system and no sludge handling/management has been required. Performance monitoring data for IAPS as a sewage treatment technology in South Africa is nevertheless needed not only to inform and educate through dissemination but also to develop a rollout strategy for implementation of full-scale commercial plants.

Prior research focussed on the four component ponds of the IAPS as standalone processes and the optimization of each but did not address performance of the system as a whole (Rose et al., 2002a; Rose et al., 2007). As a consequence, there exists the perception that the treated effluent from IAPS does not meet the final COD and TSS concentrations due in part, to suspended algae moving over the weir of the algae settling ponds (Meiring and Oellermann, 1995). In fact, a recent report on the operation of hectare-scale HRAOP for enhanced wastewater treatment strongly advocated additional treatment of the outflow from ASP by polishing to meet specific discharge standards (Craggs et al., 2012). These authors recommended 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 and one that compliments the low cost, environmental aspect of this sewage treatment technology. Despite concerns, and in an effort to redress prevailing oversight, studies were initiated to examine the water quality of the final effluent from an IAPS treating municipal sewage. Thus, operation of the Belmont Valley WWTW IAPS was monitored for a two-year period to evaluate the quality of the treated water and to assess the contribution of various tertiary treatment processes to enhance water quality prior to discharge.

After due consideration of available technologies, time, and cost of construction and implementation together with suitability for use, a maturation pond (MP) series, slow sand filtration (SSF) and a controlled rock filter (CRF) were selected as candidate unit processes for tertiary treatment of the Belmont Valley IAPS final effluent. The unit processes were designed to reduce overall pathogen content and improve effluent quality using published criteria (Ellis, 1987; Shillinglaw, 1977) and the specifications are presented in Appendix C. Composite water sampling was used to ensure that the values for the measured parameters were indeed thorough and comprehensively derived indicators of system performance. Sampling was at weekly intervals over two 8 month periods during summer from

September 2012 to May 2013 and September 2013 to May 2014 and methods of analysis of the physical, chemical and microbial characteristics of the treated water are recounted in Appendix A. In addition, water quality data from earlier studies (i.e. 2002 and 2006) on performance of the Belmont Valley WWTW IAPS treating municipal sewage were re-evaluated and a summary of all results is presented in Table 10.

Based on the data presented in Table 10 it is clear that the Belmont Valley WWTW pilot-scale IAPS consistently generates a final treated water with TSS that does not comply with discharge standards (Mambo et al., 2014a). This is perhaps not surprising as the system was designed, constructed, and has been operating without a TTU. Furthermore, system configuration and operation has limited the efficiency of the HRAOPs for nutrient removal. The partially treated effluent after HRAOP *A* should not be returned to the Belmont Valley WWTW inlet works but rather, used to dilute influent BOD_{ult} to achieve the correct loading of 40 $kg \cdot d^{-1}$ according to design specification (see; 2.5 IAPS Kinetic Parameters, above). The apparent reason for adopting the aforesaid operational configuration is unknown. Furthermore, and as indicated from the results presented in Table 11, the introduction of an appropriate TTU in the IAPS process flow would ensure that water quality is sufficient for both irrigation and discharge for volumes up to 2 Mℓ of any given day. Thus, the present investigation coupled with analysis of data for the Belmont Valley WWTW IAPS system from 2002 and 2006 strongly support the conclusion of Craggs et al. (2012) that additional treatment of the effluent is required to reduce residual TSS in the treated water.

Table 10 A summary of water quality data of the final effluent from the Belmont Valley WWTW IAPS. Mean values were obtained from two earlier studies (i.e. 2002 and 2006) and from composite sample analysis (i.e. 2012/2013 and 2013/2014). Also shown are the general authorization limits for either irrigation or discharge of up to 2 Mℓ on any given day (DWA, 2013).

Parameter (units)	General limit: Irrigation[A]	General limit: Discharge[A]	Water quality of final effluent			
			2002	2006	2012-13	2013-14
pH	5.5-9.5	5.5-9.5	10.5	9.5	9.4 ± 1	9.1 ± 1
Dissolved oxygen ($mg \cdot ℓ^{-1}$)	>2	>2	NA[B]	NA	5.5 ± 1	5.7 ± 2
Electrical conductivity ($mS \cdot m^{-1}$)	70 $mS \cdot m^{-1}$ above intake to a maximum of 150 $mS \cdot m^{-1}$	70 $mS \cdot m^{-1}$ above intake to a maximum of 150 $mS \cdot m^{-1}$	NA	NA	108 ± 19	112 ± 14
Chemical oxygen demand ($mg \cdot ℓ^{-1}$)	75	75	65	60	72 ± 13	66 ± 12
Nitrate/nitrite-N ($mg \cdot ℓ^{-1}$)	15	15	17.5	15	12 ± 1	2.3 ± 2
Ammonium-N ($mg \cdot ℓ^{-1}$)	6	3	7	1.5	2.9 ± 1	2.6 ± 1
Phosphate-P ($mg \cdot ℓ^{-1}$)	10	10	2.6	5.4	5.3 ± 2	4.3 ± 2
Total suspended solids ($mg \cdot ℓ^{-1}$)	25	25	60	60	34 ± 13	35 ± 14
Total coliforms ($cfu \cdot 100\ mℓ^{-1}$)	1 000	1 000	>1 000	>1 000	>1 000	>1 000

[A] General Authorisations in terms of section 39 of the national water act (Republic of South Africa, *Water Act 1998*)
[B] Not available

Table 11 Summary data on water quality of the effluent from the Belmont Valley WWTW IAPS before and after tertiary treatment either by a maturation pond series (MPS), slow sand filtration (SSF), or controlled rock filtration (CRF). Data for water quality were determined on a per week interval over a period of 8 months in the summer of 2013/2014.

Parameter (units)	Water quality of final effluent			
	IAPS	+ MPS	+ SSF	+ CRF
pH	9.4 ± 1	9.9 ± 0.5	8.3 ± 0.7	8.0 ± 0.4
Dissolved oxygen ($mg \cdot \ell^{-1}$)	5.5 ± 1	13.5 ± 3.6	6.0 ± 2.5	12.6 ± 1.4
Electrical conductivity ($mS \cdot m^{-1}$)	107 ± 19	95 ± 47	95 ± 48	100 ± 12
Chemical oxygen demand ($mg \cdot \ell^{-1}$)^A	72 ± 13	72 ± 10	59 ± 12	62 ± 4
Nitrate/nitrite-N ($mg \cdot \ell^{-1}$)	12.4 ± 4	4.0 ± 1.9	6.7 ± 2.5	5.1 ± 1.7
Ammonium-N ($mg \cdot \ell^{-1}$)	2.9 ± 1	0.5 ± 0.5	2.3 ± 0.9	0.3 ± 0.1
Phosphate ($mg \cdot \ell^{-1}$)	5.3 ± 2	4.3 ± 1.7	1.4 ± 0.6	0.5 ± 0.2
Total suspended solids ($mg \cdot \ell^{-1}$)	35 ± 13	22 ± 12	19 ± 8	19 ± 9
Total coliforms ($cfu \cdot 100\ m\ell^{-1}$)	>1 000	<1 000	<1 000	<1 000

^{A} After removal of algae by filtration

2.9 Gap Analysis of IAPS as a WWT Technology

Water quality issues outlined above, and in particular elevated TSS and to a lesser extent COD, have contributed to a perception that IAPS does not meet all of the criteria for municipal sewage treatment (Meiring and Oellermann, 1995). Although there are no known commercially operated systems, efforts by proponents of IAPS to have the technology constructed and implemented at full scale have not been dampened. Indeed, many South African-based projects have been initiated including a UNEP WioLap sponsored IAPS (1 $M\ell \cdot d^{-1}$, Bushman's River, Nlambe Municipality), two Partners-for-Water sponsored IAPS (2 $M\ell \cdot d^{-1}$ Grahamstown, Makana Municipality; 1.5 $M\ell \cdot d^{-1}$, Alice, Amathole Municipality), and the conversion of a WSP system to an IAPS (2-3 $M\ell \cdot d^{-1}$, Bedford, Amathole Municipality). In each case the projects proceeded through the design stage but failed at implementation. Reasons for these failures though many and varied, were explored and analysed in a recent study by Nemadire (2011) who noted that the failure of IAPS to meet South African authorisation limits, preference for other technologies including AS, and delays due to conflicts with stakeholders were among the major contributing factors.

Most recently, the Department of Science and Technology (through the Water Research Commission) awarded funds to Rhodes University to manage the design, construction and implementation of IAPS for municipal sewage treatment. These projects are joint ventures between Rhodes University, Chris Hani District Municipality and uMjindi Local Municipality respectively, and the Water Research Commission. These projects are currently in the design and planning stage for implementation in Tarkastad and Barberton. However, prior failure of four similar projects suggests that due caution be exercised before proceeding with construction and commissioning of the technology. In an effort to address concerns and technology weaknesses already highlighted a gap analysis of IAPS was undertaken. Gap analyses typically compare best practice with current processes to determine the “gaps” so that “best practice” is selected for implementation. In this study IAPS configuration and operation was evaluated in terms of its component parts and water treatment efficacy. Both metadata and real time data derived from the pilot IAPS at the Belmont Valley WWTW were used to supplement the analysis.

For the purposes of gap analysis, criteria were defined as either primary or secondary, where secondary criteria were grouped according to each primary criterion (Table 11). Qualitative gap analysis was based on the primary criteria of, 1) status of the technology which included as secondary criteria: water quality, sludge handling, biomass beneficiation, operation/maintenance, energy balance, greenhouse gas emission, operator training, market readiness; 2) design and process flow including: inlet works, organic loading, primary treatment, secondary treatment, biomass harvesting, tertiary treatment, biocatalyst consistency; and 3) cost. Each primary factor provided the basis for comprehensively evaluating the technology through defining appropriate secondary criteria as specified below. In addition, a quantitative assessment of the gap analysis was achieved by scoring each criterion on a scale from 1-10, where 10 represents the ideal.

Results from a gap analysis of the primary and secondary criteria used to evaluate IAPS as a WWT technology, and listed in Table 12, are presented in Figure 11. It is clear that large gaps exist in terms of technology status, process and design, and overall costs. In most cases the gap is >40%. Evaluation of the current international status of the technology confirmed that IAPS treated water has elevated TSS and COD and is considered not suitable for discharge to the environment without additional tertiary treatment (i.e. MPS, SSF, CRF, etc.). Although there is no faecal sludge handling, biomass harvesting and dewatering is required on a routine basis. Efficiency of the harvesting/dewatering process by passive settling has not been quantified and it is distinctly possible that the settled algae contribute to elevated TSS and COD of the treated water. Harvesting of biomass at the Belmont Valley WWTW IAPS is carried out bi-monthly and there is presently no beneficiation process in place. The system is dependent on fossil fuel derived electricity despite it being a source of bio-methane which along with other greenhouse gasses, is currently vented to atmosphere. Although considered near market ready, no operator

Table 12 Results of a gap analysis of IAPS as a commercial WWT technology.

Primary	Secondary	Current Practices	Sc.	Gap	Best Practices	Sc.
Status of Technology	*Water quality*	Not suitable for discharge to environment	5	Additional treatment needed	Suitable for discharge to a water course	7
	Sludge handling	No, but there is accumulation	6	Organic loading not checked	Faecal sludge — None	9
	Biomass valorisation	Algae biomass discarded	1	Algae biomass not valorised	Algae sludge digested to methane and/or fertilizer production	9
	Operation/maintenance	Spares not available on site	1	No equipment or support	Replacement paddles on site	8
	Energy balance	Consumes fossil fuel	1	No capacity methane to electricity	Energy generating system	7
	Greenhouse gas emission	Vented to the atmosphere	2	No gas capture / carbon sequestration status unknown	Near neutral	7
	Operator training	Minimal (low skill)	8	Training not provided	Minimal but skilled	8
	Market readiness	Near-ready	7	Require commercial scale demonstration	Accepted WWT technology	8
IAPS Design and Process	*Inlet works*	Absent	1	Include in commercial design	Part of process design/operation	7
	Organic loading	Overloaded / incorrect configuration	1	Unreliable data from pilot system	Operating according to design parameters	8
	Primary treatment	Combined IPD-AFP	5	Separate AD and AFP for inspection and maintenance	Separate AD and AFP	7
	Secondary treatment	Two HRAOPs	6	More HRAOPs/increase size	Adequate size/number of HRAOPs	9
	Biomass harvesting	Passive settling in ASP	2	Inadequate harvest/dewatering	Active harvesting and dewatering	8
	Tertiary treatment	Absent	1	Final polishing and disinfection (e.g. chlorination / UV)	Maturation pond, slow sand filtration, rock filter, drum filter, etc., and chlorination	7
	Biocatalyst consistency	Determined by environment microalgae = optimal; diatom = poor	5	Erratic species dynamic may compromise water quality	Active selection for large colonial microalgae for ideal nutrient abstraction and biomass	7
Costs	*Construction*	Case/site specific	7	Design and build full scale IAPS	Commercial scale fully costed IAPS	8
	Operation	Minimal but unquantified	6	Operate full scale system	Reliable operating cost data	9
	Maintenance	Minimal but unquantified	6	Maintain full scale system	Maintenance and troubleshooting routine	9

training or standard operating procedures are in place (see Appendix E). Reliable techno-economic data is also conspicuous by its absence.

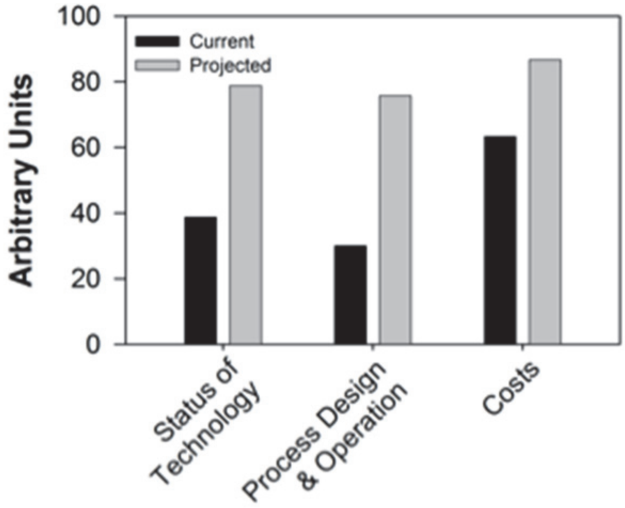

Figure 11 Quantification of the scores for the primary criteria used in a gap analysis of IAPS.

In terms of process design, original specifications were for a 500 PE system to treat raw sewage with BOD of 40 $kg \cdot d^{-1}$. Recalculation of the kinetic parameters revealed that the organic strength of the influent was not commensurate with this design and that the actual BOD_{ult} load is closer to 80 $kg \cdot d^{-1}$ (Table 8). This increase in organic loading to the system might be expected to decrease the efficiency of the technology and provide a treated water with elevated TSS and COD.

The pilot scale IAPS at the Belmont Valley WWTW, upon which this gap analysis was carried out, lacks an inlet works, and tertiary treatment and disinfection as components in the process flow. Both the organic load into the system and the dewatering/harvesting of biomass appear to be compromised and directly contribute to elevated TSS and COD of the treated water. Population dynamics of the algae/bacterial biocatalyst (i.e. MLSS) within HRAOPs may also contribute to reduced nutrient abstraction particularly when diatoms dominate and outcompete microalgae (Fig. 8). Variations in the dominant biocatalyst also influence efficiency of passive settling causing poor dewatering which presumably also contributes to elevated TSS and COD. Faecal sludge handling is not normally required in IAPS operation and the skill and energy needs are minimal making this technology attractive in terms of environment impact. Even so, efforts to beneficiate products of IAPS water treatment by downstream value adding have not been realised. Indeed, the products which include treated water for recycle and reuse, biogas for electricity/heating, and biomass have for most part not been thoroughly quantified. Lastly, the gap analysis revealed surprisingly limited and only rudimentary information relating to the costs of construction, implementation and operation of IAPS.

2.10 Summary and Conclusions

Successful wastewater management reduces faecal oral diseases and environmental pollution caused by sewage (Cisneros et al., 2011). In the year 2000 it was reported that approximately 2.4 billion people did not have access to clean water and sanitation, which led to 1.7 million preventable deaths (2012). Also, the United Nations estimated that 3.4 billion people already live in water stressed countries (UNFPA 2005). Thus, there is a clear need for authorities to implement sustainable water management technologies. The present report has re-appraised the Belmont Valley WWTW IAPS as a technology for municipal sewage treatment. A model was used to re-evaluate the kinetic parameters of both the IPD/AFP and HRAOP components of this WWT system which revealed the following;

- Actual strength of the raw sewage entering this demonstration IAPS is two-fold design capacity and should be diluted by 50% in order achieve the specified design loading rate (BOD_{ult}) of 40 $kg \cdot d^{-1}$;
- Based on measured values, surface area of the HRAOPs for nutrient abstraction was calculated as 0.085-0.104 ha which compares favourably with the design specifications (i.e. 0.1 ha);
- Analysis of productivity (measured as MLSS) confirmed that a total HRAOP surface area of ~0.6 ha is sufficient in summer, and increases to ~1.0 ha in winter, and that this additional surface area is catered for in the design.

Quality of IAPS-treated water was monitored over extended operating periods before and after tertiary treatment and the results revealed;

- The treated water complies with the general limit values for either irrigation or discharge into a water resource that is not a listed water resource for volumes up to 2 Mℓ of treated wastewater on any given day;
- Parameters including COD, TSS, pH, DO, EC, and N and P values of the treated water were within the general limit for all tertiary treatment processes tested, i.e. MPS, SSF and CRF.

Results from a gap analysis of IAPS as a WWT technology based on the pilot-scale system located at the Belmont Valley WWTW revealed the following;

- Operational oversights which should have been accounted for in process flow configuration have contributed to a build-up of sludge in the IPD and AFP;
- Large gaps exist in terms of technology status, design and process operation, and cost of implementation, and in most cases the gap is >40%;
- Faecal sludge handling is not required in IAPS operation and energy needs are minimal.

3. COD and TSS in IAPS-Treated Water

One of the major limitations to the commercialisation and full scale implementation of IAPS as a municipal WWT process has been the consistently elevated COD and TSS in the final effluent from the treatment process. The general authorisation limits for either irrigation or discharge stipulate a COD of 75 $mg \cdot \ell^{-1}$ or less and a TSS content equivalent to or less than 25 $mg \cdot \ell^{-1}$. IAPS and other high rate algae pond WWT plants however, seldom meet these criteria which has prompted at least one author to advocate additional treatment to meet specific discharge standards (Craggs et al., 2012). An attempt was therefore made to characterise the COD and TSS components in water emerging from the Belmont Valley WWTW demonstration IAPS and to consider other unit operations that might ensure compliance and standard for this bioprocess technology.

Prior to COD and TSS characterisation studies it is worth revisiting some of the definitions and underlying concepts that are used to describe organic matter to elaborate its complexity, potential origin, and persistence or recalcitrance.

Organic matter in aquatic systems is typically a complex mixture of molecules such as carbohydrates, amino acids, hydrocarbons, fatty acids and phenolics, natural macromolecules and colloids (e.g. humics), sewage/industrial particulates, soil organic matter, living phytoplankton and other plant and animal material. The more reactive constituents of organic matter (e.g. carbohydrates) make a significant contribution to heterotrophic metabolism in aquatic systems. For example, fulvic acids and other humic substances affect the behaviour and transport of metals by forming chelates. These compounds also interact with organic pollutants and adsorb to the surfaces of mineral solids thus affecting surface chemistry and rates of aggregation.

Most waters contain organic matter that can be measured as total organic carbon (TOC). Sources of organic carbon include living material, waste materials, and effluents. Organic matter from living material may arise directly from photosynthesis or indirectly from organic matter senescence and decay. An indication of the amount of organic matter present can be obtained by measuring related properties including BOD, COD, turbidity and colour. The COD usually includes the BOD as well as other chemical demands producing the relationship, COD > BOD > TOC.

Total organic carbon consists of dissolved organic carbon (DOC) and particulate organic carbon (POC) and is therefore affected by pronounced fluctuations in suspended solids. The DOC and POC levels are determined separately after passage of the sample through a filter of approximately 0.4-0.7 µm pore diameter. Typically, DOC levels exceed POC levels in the range 6:1-10:1, except in highly turbid waters where POC dominates (Wetzel, 1975).

The POC fraction of TOC arises either from allochthonous inputs (e.g. leaf litter) or autochthonous inputs from the littoral and pelagic zones of carbon flux. Much of the metabolism and decomposition of POC takes place in the sediments or en route during sedimentation and depends on composition of parent material, climate, topography, hydrology, and vegetation (Mitchell and McDonald, 1995; Håkanson, 1993; Heikkinen, 1994; Midgley and Schafer, 1992). In addition, some land use activities (e.g. animal husbandry) tend to increase the potential for higher levels of POC and TOC, while other activities (e.g. clear cutting) tend to decrease organic carbon inputs (Shields and Sanders, 1986; France, 1995).

Dissolved organic carbon is composed primarily of (i) non-humic substances, a class of compound that includes carbohydrates, proteins, peptides, fats, pigments and other low molecular weight compounds, and (ii) humic substances which form most of the organic matter in waters, and consist of coloured hydrophilic and acidic complexes ranging in molecular weight from the hundreds to thousands of kDa (Wetzel, 1975). Humic substances are formed largely as a result of microbial activity on plant and animal material and are more persistent than non-humic substances, which are easily utilized and degraded by microorganisms (i.e. substances are labile) and exhibit rapid flux rates in aquatic systems.

The measure of the amount of 'pollution' (that cannot be oxidised biologically) in a water sample is termed COD while BOD is the measure of the quantity of O_2 required by aerobic organisms to decompose this organic matter. In South Africa, the DWS ensures compliance with Section 39 of the Water Act (Republic of South Africa, *Water Act 1998*). This legislation is concerned with prevention of pollution, and therefore sets concentration limits on dissolved organic carbon (as COD), nitrogen and phosphates. It also attempts to limit the discharge of known toxic chemicals by setting allowable concentration limits in the effluent.

Recognising that effluents contain unidentified toxic chemicals, a more pragmatic approach to regulation has been introduced in Europe, using direct toxicity assessment (DTA) tests. In the U.S.A. these have been in use for many years and are known as whole effluent toxicity (WET) assays. These tests are used to measure the toxic effect of effluents on representative organisms from receiving waters. A similar approach is gaining ground in South Africa although it is not yet part of the legislation governing discharge of treated effluents to water resources (including listed water courses).

Municipal sewage is made up largely of organic carbon, either in solution or as particulate matter. About 60% is in particulate form, and of this, slightly under a half is large enough to settle out of suspension. Particles of 1 nm to 100 µm remain in colloidal suspension and during treatment become adsorbed on to the sludge flocs. The bulk of the organic matter is easily biodegradable, consisting of proteins, amino acids, peptides, carbohydrate, fats and fatty acids. The average carbon to nitrogen to phosphorus ratio (i.e. C:N:P or Redfield ratio) is variously stated as approximately 100:17:5 or 100:19:6, which is close to the ideal for the growth of microorganisms including algae.

For control of biological processes in a treatment plant, it is necessary to have some knowledge of the organic strength, or organic load, of the influent wastewater. Three different measures of this are available, and they each have their merits and weaknesses. The TOC is determined after combustion at very high temperatures and measurement of the resultant CO_2. However, TOC includes stable organic carbon compounds that cannot be broken down biologically. Organic carbon can also be measured by chemical oxidation. The sample is heated in strong acid containing potassium dichromate, and the carbon oxidised is determined by the amount of dichromate used in the reaction. The result is expressed in units of oxygen, rather than carbon (i.e. COD). Again it is an analytically simple method. However, its weakness is that a number of recalcitrant organic carbon compounds that are not biologically oxidisable, are included in the value obtained. Conversely, some aromatic compounds, including benzene, toluene, and some pyridines, which can be broken down by bacteria and algae, are only partly oxidised in the COD procedure. Overall however, COD will overestimate the carbon that can be removed.

The current method used to determine biodegradable carbon, is the 5-day biological oxygen demand (BOD_5). This is a measure of the oxygen uptake over a 5-day period by a small 'seed' of bacteria when confined, in the dark, in a bottle containing wastewater. During this time the biodegradable organic carbon is taken up and there is a corresponding decrease in the dissolved oxygen, as some of the carbon is used for microorganism respiration, a form of biological oxidation. Rather unhelpfully, the biodegradable carbon, as in the COD test, is expressed in oxygen units. This is because the test was originally introduced to measure oxygen depletion in receiving waters caused by residual degradable carbon in the effluent. Its main value is in regulating the composition of effluents from the treatment water. For process management, where knowledge of the organic loading of the influent is required, BOD_5 is of limited value, because of the 5 days required to make the measurement. There are now moves afoot to replace the use of BOD_5 as a measure of influent strength, with a short-term test (BODST), which can be carried out over a timescale of 30 minutes to several hours. Values obtained for BOD_5 are always lower than those for COD, for two reasons: 1) microorganisms cannot degrade some of the compounds oxidized chemically in the COD test; and 2) some of the carbon removed during the BOD test is not oxidised, but ends up in new biomass. So, BOD measures the biodegradable carbon that is actually oxidised.

The ratio of BOD_5/COD depends on the composition of the wastewater. For municipal sewage, and also wastewater from abattoirs, dairies, and distilleries, the ratio is about 0.5-0.6. However, for effluent leaving a WWTW, after treatment, the value should be closer to 0.2. This is because readily biodegradable organic carbon has been removed during treatment, leaving behind compounds that are not readily broken down termed 'hard' BOD. These will be readily measured by chemical oxidation, but will not be readily degraded and removed. Notwithstanding the aforementioned methodological constraints, here we describe attempts to characterise the COD and TSS components in final effluent from the Belmont Valley WWTW IAPS and to consider other unit operations, which might ensure compliance and standard for this bioprocess technology.

3.1 IAPS Configuration, Water Sampling and Analysis

The IAPS at the Belmont Valley municipal WWTW treats 75 $k\ell \cdot d^{-1}$ raw domestic sewage based on a design of 500 PE. This pilot consists of an IPD, AFP, two HRAOPs and two ASP configured in series. Since deployment in 1996 the system has been utilized to remediate municipal sewage and the process flow is summarized in Figure 12.

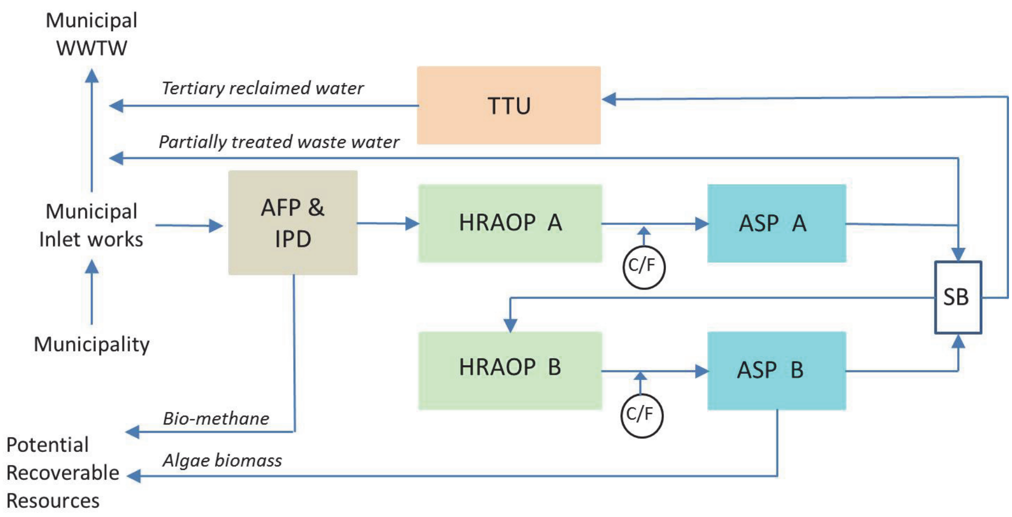

Figure 12 Schematic of the operating process flow of the Belmont Valley municipal WWTW IAPS. Effluent enters at the bottom of the IPD some 6 m below water level. C/F=coagulation/flocculation; SB=splitter box; TTU=tertiary treatment unit.

The typical operating configuration is: 3 d in the IPD → 20 d AFP → 2 d HRAOP A → 0.5 d ASP A → 4 d HRAOP B → 0.5 d ASP A → return to Belmont Valley WWTW inlet works. This system has been in continual use since implementation and affords secondary treated water for reclamation according to its design specifications.

Composite samples of treated water were obtained by collecting aliquots over a 24 h period. Samples were stirred and 500 mℓ abstracted for analysis after determination of temperature, pH, EC, and DO. Temperature and EC were measured using an EC Testr 11 Dual range 68X 546 501 detector (Eutech Instruments, Singapore), while DO was determined using an 859 346 detector (Eutech Instrument, Singapore). The pH was measured using a Hanna HI8 424 microcomputer pH meter (Hanna Instrument, Romania).

Total dissolved solids and total volatile solids were determined according to Standard Methods (1998). Where specified, samples were sequentially filtered using Whatman No. 1 filter paper (pore size, 11 µm) followed by Whatman glass fibre filters (pore size, 1.6 µm) and then Whatman nylon filters (pore size, 0.45 and/or 0.22 µm) prior to analysis. Test kits (Merck KGaA, Darmstadt, Germany) were used to determine NO_3-N+NO_2-N, PO_4-P, NH_4-N, SO_4-S, and COD according to the manufacturer's instructions.

Carbohydrates were quantified using the phenol sulphuric acid assay according to Dubois et al. (1959). Determination of reducing sugar concentration was carried out according to Miller (1959). Protein concentration was determined according to Bradford (1976) while alpha amino nitrogen was quantified using a modified Ninhydrin assay which was carried out according to Hwang and Ederer (1975). Humic-like substances were quantified as described by Wang and Hsieh (2000).

For estimation of faecal and total coliforms, MacConkey and m-Fc agar (Biolab, South Africa) were prepared according to the manufacturer's instructions and 100 µℓ aliquots of water sample spread-plated and the Petri dishes incubated at 30ºC and 45ºC respectively for 24 h.

For analysis of elemental C, N, S and H, filtered water samples were frozen using liquid nitrogen, lyophilised and analysed using an Elementar vario MICRO CUBE Bioanalytical elemental analyser (combustion time of 10 s) with helium as carrier and the analyte gases sequentially separated according to temperature.

3.2 Characterisation of COD and TSS

Analysis of the COD in final effluent after sequential filtration of the water through 11, 1.6, 0.45 and 0.22 µm pore size filters revealed there to be no significant change (Table 13). Initially, a preliminary investigation revealed that percentage change relative to COD and $COD_{filtered}$ remained unchanged despite the decreasing filter pore size (Experiment 1, Table 13). This prompted a more rigorous analysis spanning several months which again revealed no significant difference between COD and $COD_{filtered}$ in IAPS-treated water (Experiment 2, Table 13). Taken together, these findings strongly suggest that the persistence of COD in IAPS final effluent is largely due to DOC rather than POC.

Table 13 Recalcitrance of COD in the final effluent of the Belmont Valley WWTW IAPS, treating municipal sewage. Data followed by the same letter are not significantly different (one-way ANOVA, p<0.05).

Filter pore size (µm)	Unfiltered	11	1.6	0.45	0.22
			COD, mg·ℓ$^{-1}$ (%)		
Experiment 1					
Mean	126.8 ± 36.7a	94.0 ± 15.6a	74.4 ± 7.5a	68.8 ± 10.6a	71.3a
% change in COD	0	-25.8b	-41.3b	-45.7b	-43.8b
% change in $COD_{filtered}$	-	0	-20.8c	-26.8c	-24.1c
Experiment 2					
Mean	88.6 ± 25.5a	76.8 ± 15.6a	57.6 ± 11.1a	58.8 ± 15.8a	n.d.
% change in COD	0	-13.3b	-34.9b	-33.6b	n.d.
% change in $COD_{filtered}$	-	0	-25.0c	-23.4c	n.d.

Partial support for the COD being predominantly dissolved organic carbon in origin was obtained by measuring the amount of humic acid-like material in the water both before and after sequential filtration and the results are shown in Table 14. Levels of humic acid-like substances were low and only after filtration through a 0.45 µm filter, was there any real reduction in content of humic acid-like material suggesting that the bulk of this component comprised smaller humics (i.e. low molecular weight humics) and possibly fulvic acids.

Table 14 Concentration of humic acid-like substances in water from the Belmont Valley IAPS final effluent. Humics were analysed as described in Materials and Methods. Data are the mean ± S.D. of at least four determinations.

Filter pore size (µm)	Humic-like substances (mg·ℓ$^{-1}$)
Unfiltered	10.2 ± 0.10
11	9.7 ± 0.04
1.6	9.2 ± 0.10
0.45	6.4 ± 0.10

In addition to the above analyses, IAPS-treated water was analysed for C, N, H, and S using an elemental analyser and the C:N ratio determined (Table 15). Results show that the relative concentrations of these elements were unaffected by filtration and that the C:N ratio closely approximated the Redfield ratio (i.e. 6.6) indicating a nutrient composition typical of that found in both freshwater and marine environments (Redfield, 1958).

Table 15 Carbon, nitrogen, and sulphur content of the IAPS final effluent determined before and after sequential filtration of water.

Filter pore size (µm)	C	N	H	S	C:N
			(%)		
Unfiltered	6.8	1.0	0.9	2.7	6.8
11	6.8	1.2	0.7	2.4	5.7
1.6	6.6	1.3	0.8	2.5	5.1
0.45	5.9	0.8	0.7	2.3	7.4

More important perhaps was the very high and sustained sulphur content detected in IAPS-treated water. Dimethyl sulphide (DMS), the most important form of sulphur gas, is formed by bacterial degradation of dimethylsulfoniopropionate (DMSP) and the process, originally believed to be confined to marine algae, can be dominant in freshwater systems where the product affects the odour quality of drinking and recreational water systems (Yoch, 2002). Large amounts of DMS originate from the decomposition of the precursor DMSP and it is now known that all species of algae and all natural waters contain DMS in amounts comparable to marine waters. However, in contrast to marine waters, DMS generally constitutes only 5 to 25% of all the sulphides present, whereas Me_2SH usually predominates (40-80%). H_2S+COS, CS_2 and DMDS are commonly present, whereas other species are rarely detected (Caron, 1990). Diatoms are a rich source of DMS, e.g. *Thalassiosira pseudonana*, *Chaetoceros* sp., *Navicula* sp., and *Nitzschia* sp. (Kasamatsu et al., 2004; Van Bergeijk et al., 2002) and nitrogen limitation causes up to a two-fold increase in total DMSP per cell and up to a three-fold increase in DMS per cell (Yang et al., 2011). Although there is scant information on this topic as it concerns the role of algae in the treatment of municipal sewage, there is sufficient circumstantial evidence to suggest that both DMSP and DMS will be prevalent in the algae biomass produced during wastewater treatment. This aspect deserves further detailed study.

Metabolic processes are distinctly coupled in microbial communities such as those present in HRAOPs: photosynthetic primary producers (bacteria and/or unicellular algae) release C- and N-based DOM comprising various organic compounds and amino acids that are readily assimilated and re-mineralized by heterotrophic bacteria/archaea and protozoa (Pomeroy, 1974; Azam, 1983). Phytoplankton and bacteria are the main sources of DOM (Jioa et al., 2010) and the biopolymers produced arise by mechanisms including direct release, mortality by viral lysis, regulated exocytosis of metabolites and polymergels, grazing, and apoptosis. Apoptosis, the commonest phenotype of programmed cell death (PCD), is well documented in chlorophytes allowing cellular materials to become dissolved in the environment and it has recently been elegantly demonstrated that PCD in microalgae causes the release of organic nutrients which are used by others in the population as well as co-occurring bacteria to re-mineralize the dissolved material promoting algal growth. Algal PCD is therefore a mechanism for the flow of dissolved photosynthate between unrelated organisms. Ironically, PCD also plays a central role in the organism's own population growth and in the exchange of nutrients in the microbial loop (Orellana et al., 2013). In fact, this study demonstrated that a significant proportion of the algae population (55% ± 15) undergoes PCD at night after daytime growth. This prompted an in depth screening of the IAPS final effluent for compounds that might have originated due to PCD of algae in either the HRAOPs or the ASPs and the results are shown in Table 16.

In addition to the expected nutrients nitrate/nitrite-N, ammonium-N, and phosphate a range of compounds typically associated with dissolved organic matter (carbon) were detected including carbohydrates, reducing sugars, protein, alpha amino-N, and humics indicative of operation of a microbial loop which presumably involves PCD of microalgae. Again, this is an aspect of IAPS that needs further investigation in order to confirm this phenomenon.

Table 16 Total soluble solids (TSS) and biochemical composition of IAPS-treated water. Data were from samples collected at regular intervals between 1 March and 20 November 2013 when the diatom *Cyclotella* was dominant and are presented as the mean ± SD.

TSS	35.0 ± 12.3
Metabolite	
Nitrate/nitrite-N (mg·ℓ$^{-1}$)	7.0 ± 4.4
Ammonium-N (mg·ℓ$^{-1}$)	7.4 ± 1.1
Phosphate (mg·ℓ$^{-1}$)	10.2 ± 3.6
Humic substances (mg·ℓ$^{-1}$)	6.0 ± 1.7
Carbohydrates (mg·ℓ$^{-1}$)	49.7 ± 15.9
Reducing sugars (mg·ℓ$^{-1}$)	19.3 ± 9.4
Protein (mg·ℓ$^{-1}$)	15.3 ± 3.9
Alpha amino nitrogen (mg·ℓ$^{-1}$)	1.8 ± 1.1

3.3 Consideration of Other Unit Operations

Perhaps the best studied example of an IAPS and its performance for the treatment of municipal sewage is the pilot plant designed for use in arid regions shown in Figure 13 (Esen et al., 1987; 1991; Puskas and Esen, 1989; Banat et al., 1990; Puskas et al., 1991; Al-Shayji et al., 1994). Similar to the Belmont Valley IAPS, the system described by Banat et al. (1990) comprised of a deep AFP followed by shallow HRAOPs and separation or sedimentation ponds (i.e. ASPs) and was based on the typical Oswald design. In this system the facultative ponds were designed to receive wastewater and partially reduce organic loading through fermentation (presumably an IPD) while the HRAOPs were for both wastewater treatment and the production of algae biomass. This system therefore resembles very closely the demonstration IAPS under study in the present project. According to the authors the algal-bacterial ponding system performed satisfactorily providing treated effluent with less than 20 mg·ℓ$^{-1}$ BOD, 130 mg·ℓ$^{-1}$ COD, 40 mg·ℓ$^{-1}$ total nitrogen and 25 mg·ℓ$^{-1}$ NH_4-N. Average production of algal biomass was 250 kg·ha^{-1}·d^{-1}. Proper disinfection was achieved, indicated by average bacterial counts of 5 N/mℓ total coliforms and 1000 N/ml total bacteria (Puskas et al., 1991).

Furthermore, seasonal weather variations, dense wastewater and fluctuating organic and hydraulic load, did not appear to adversely affect performance of the system. Even so, and in order to increase biomass yield, these authors determined that productivity was dependent on efficiency of floc formation and settling and stated that "*A minimum of a 2-h residence time in the settling tanks is needed to achieve adequate separation of the phases, assuming healthy algae culture that are susceptible to flock formation. The flock formation and settling properties are sensitive to the weather and operational parameters (e.g. sunshine, hydraulic conditions in settling tank, variations of the flow rate, and pond depth). If conditions are inadequate, 20-30% of the biomass can be lost. Satisfactory biomass production can be achieved only with careful plant operation and maintenance*." (Al-Shayji et al., 1994).

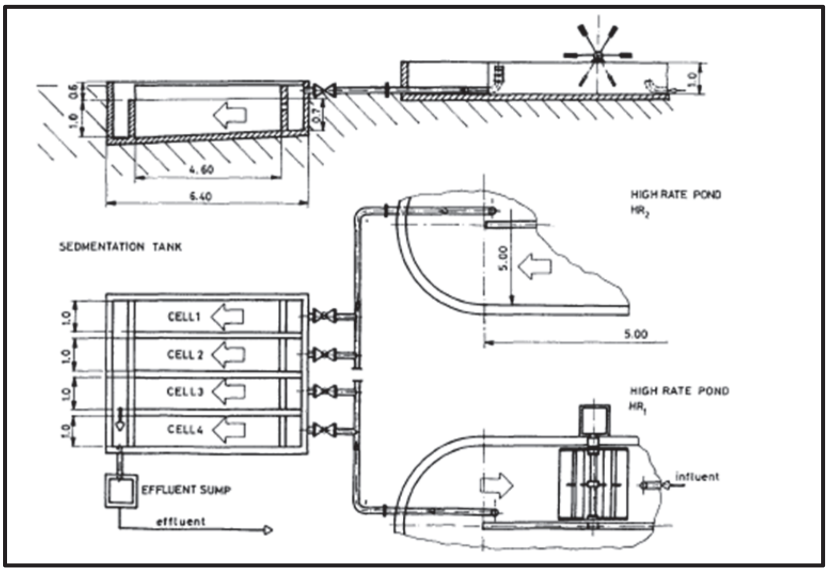

Figure 13 Algae production system, high-rate ponds followed by sedimentation tank (Puskas et al., 1994).

This pilot system routinely yielded efficient algae sedimentation resulting in an appropriate effluent with total suspended solids less than 20-30 mg·ℓ^{-1} but only when operated at optimum. More typically, and as stated by Esen et al. (1991), the effluent was rich in algae, and did not meet stringent water-quality criteria on suspended solids. In mitigation of these negatives, slow sand filtration was introduced to remove algae and algae residues from HRAOP effluent. Thus, when agricultural sandy soil with an effective grain size of 0.08 mm was used as the filter medium, an average filtration rate of about 1.3 $m^3 \cdot m^{-2} \cdot d^{-1}$ was obtained and the final effluent had a BOD value less than 20 $mg \cdot \ell^{-1}$ and undetectable faecal coliforms (Esen et al., 1991).

The contribution of slow sand filtration, controlled rock filtration, and maturation ponds as tertiary treatment units has been investigated and results show that incorporation of either into the IAPS design and operation ensures compliance of the HRAOP effluent for either irrigation or discharge to a watercourse (see Table 11).

Gravity sedimentation is the most common and cost effective method of algal biomass removal in wastewater treatment because of the large volumes of wastewater treated and the relatively low value of algal biomass generated (Nurdogan and Oswald, 1996). However, algae settling ponds, and as indicated above, have relatively long retention times (1-2 d) and only remove 50-80% of the biomass (Nurdogan and Oswald, 1995; Brennan and Owende, 2010; Park and Craggs, 2010; Park et al., 2011a). Chemical and mechanical approaches to algae removal have the disadvantage that they increase the overall operating costs making the bioprocess far less attractive. Thus, it is clear that an alternative simple and cost effective method is required. One such method and one that has been used at Belmont Valley WWTW IAPS, involves the recycling of algal biomass already harvested by gravity settling to increase the dominance of readily settleable algae. Success in this regard was demonstrated earlier in small-scale laboratory cultures (Benemann et al., 1977; Weissman and Benemann, 1979). More recently, the influence of recycling selectively harvested algae on species dominance and harvest efficiency in a pilot-scale wastewater treatment HRAOP was investigated over one year (Park et al., 2011b). These researchers demonstrated that recycling of harvested algae biomass back to the HRAOP maintained the dominance of a single readily settleable species (*Pediastrum* sp.) at >90% over one year (compared to HRAOPs with no recycling in which only a 53% dominance was recorded) and increased the average size of *Pediastrum* sp. colonies by 13-30%. It follows that algae of larger biovolume will more likely form larger flocs that readily settle in the ASPs.

Algal species typically found in HRAOPs associated with WWTW include: *Desmodesmus*, *Micractinium*, *Pediastrum*, *Dictyosphaerium* and *Coelastrum*. As indicated by others, these algae often form large settleable colonies (diameter: 50-200 μm), which enable cost-effective and simple biomass removal by gravity sedimentation (Lavoie and de la Noue, 1986; Garcıá et al., 2000; Craggs et al., 2011; Park et al., 2011a) and, as might be expected, the HRAOPs of the Belmont Valley IAPS are also dominated by these colonial microalgae species.

A final consideration involves the incorporation of drum filtration at the expense of ASPs. In deliberation with Royal Haskoning-DHV (previously DHV) it was concluded that addition of gravity fed in-line drum filters might ensure efficient nutrient abstraction, biomass accumulation and removal (i.e. mitigating problems association with gravity sedimentation of algae) and a simplified process flow is illustrated in Figure 14.

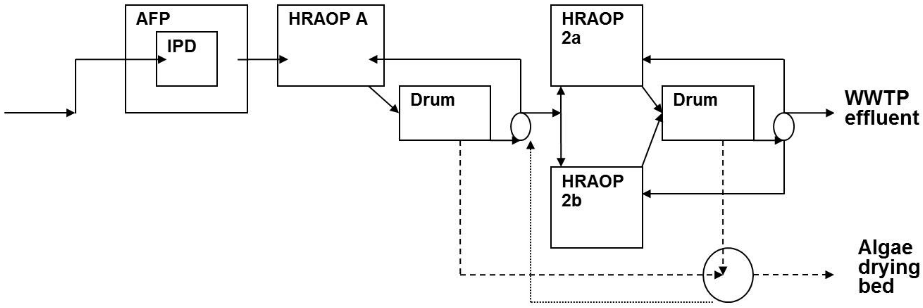

Figure 14 Process flow of a concept IAPS with in-line drum filters for removal of MLSS.

In-line drum filters are not heavy, relatively small, and are mounted on prefabricated concrete floors. Wash water flows freely to a thickening tank and the settled algae discharged. The overflow is returned by natural flow back to the sumps between HRAOP *A* and HRAOP 2*A* and 2*B*. It was rationalized that this change might, for commercial purposes, reduce the footprint, and eliminate the need for ASPs and possibly TTUs.

3.4 Summary and Conclusion

Species composition of HRAOPs, gravity settling, and ASP operation can contribute COD and TSS during wastewater treatment. Analysis revealed that residual TSS/COD comprises mostly soluble organic carbon (i.e. carbohydrates, protein and lipids) presumably derived from the MLSS. Although not confirmed, it is tempting to suggest that PCD occurs in HRAOPs and ASPs, which causes release of ‘nutrient’ to fulfill the microbial loop (Orellana et al., 2013). This is supported by the C:N ratio of sequentially filtered water samples which was closely allied to the Redfield ratio. As pointed out by Al-Shayji et al. (1994), algae floc formation and settling are sensitive to climate and operational parameters (e.g. sunshine, hydraulic conditions in settling tank, variations of the flow rate, and pond depth) and if conditions fluctuate, 20-30% of the biomass can be lost – emphasising the need to harvest and valorise the biomass.

4. IAPS for Domestic Sewage Treatment: A Life Cycle Assessment

Wastewater treatment systems and in particular STPs must be manufactured, installed, and operated. These steps often have environmental impacts, from use of chemical compounds and manufacturing emissions to land disturbance, sludge production and energy consumption. Therefore, in order to holistically assess the environmental impact of a particular system, it is crucial to look at its full life cycle through an analysis called a life cycle assessment (LCA).

An LCA quantifies the environmental impact of a product or system from its origins to its disposal. It often includes extraction of raw materials, processing, transportation, manufacture, use of product, and lastly disposal and end of life (Dixon et al., 2003). The International Organization for Standardization has outlined LCA methodologies (ISO 14040, 2006). Applying the LCA approach to STPs provides insight into the overall environmental impact of a system (Dixon et al., 2003). The approach is also useful in water management as it emphasizes the entire life cycle of a system without necessarily focusing on particular process steps or certain aspects of a system. In fact, Emmerson et al. (1995) argue that LCA has potential for wider application within the water industry. Furthermore, because LCA assesses all impacts using a consistent framework, it minimizes the potential for problem shifting.

The main environmental impacts focused on in this study included greenhouse gas (GHG) emissions (CO_2, CH_4 and NOx) and energy use (i.e. $kWh \cdot M\ell^{-1}$ wastewater treated). These impacts were chosen because of readily available data and the fact that some of the largest environmental damage associated with wastewater treatment results from GHG emissions and energy consumption (Lim et al., 2008; Brune et al., 2009; Flores-Alsina et al., 2011). Furthermore, the production of energy is associated with several environmental concerns, such as the release of airborne pollutants that contribute to global warming, acidification and generation of low-level or tropospheric ozone (Emmerson et al., 1995). Other impact categories that are important but were not included in this analysis are: abiotic depletion, caused mostly by utilizing resources for the construction; eutrophication and human and ecosystem toxicity, which could result from the discharge of partially treated wastewater or water that does not meet the standards for discharge; and faecal sludge disposal, due to the presence of parasites, pathogens and/or certain pollutants. These impact categories have been excluded from the present analysis due to a lack of consistent data across all of the technologies currently used in South Africa and, there is no available data for IAPS.

In the present study the environmental impact and associated energy flows and greenhouse gas emissions (i.e. global warming potential and/or carbon footprint) of an IAPS for municipal sewage treatment are elaborated.

4.1 Scope of Study

IAPS are constructed wastewater treatment plants that treat municipal sewage and produce a final effluent that is suitable for either irrigation or discharge to a water resource that is not a listed resource. As a STP, IAPS use minimum electrical power and no thermal energy. Rather, IAPS make use of the internal energy contained within the feedstock (sewage) to break down particulates (both suspended and dissolved) through the synergistic interaction of anaerobic and aerobic processes. Solar energy, which is harvested during photosynthesis, is used to reduce the eutrophication potential of the effluent by enhancing uptake and partition of waterborne nutrients into an actively growing biomass. These nutrients are essentially sequestered in the biomass (i.e. MLSS) and passively removed from the process stream using ASPs. Photosynthesis also generates an abundance of oxygen which supports aerobic bacteria to facilitate further decomposition of organic material into *ortho*-phosphate, NO_3-N/NH_4-N, and CO_2. Photosynthetic oxygenation of the effluent, which can approach saturation, also prevents growth of faecal coliforms and other anaerobes while solar irradiance, especially UV-B, acts in concert as a disinfectant to reduce levels of pathogenic bacteria.

Integrated algae pond systems typically have an estimated life span exceeding 25 years. Component parts of the plant have a shorter life-span. This LCA considers only the process stream. Construction inputs are excluded from this study.

4.2 Assumptions and Limitations

Conclusions that may be drawn from an LCA are limited by the quality of data available and how well results can be compared across studies or extended to other situations. Additionally, system boundaries usually vary considerably from study to study, since these are chosen based on the purpose of the LCA. For example, if the goal is to compare two treatment technologies with identical collection systems, then this individual phase would be excluded.

Available LCA literature covers different phases of the wastewater treatment process, and data and assumptions are not always transparent. Characterization methods also vary between studies, as do the impact categories that are analysed or reported in detail. For example, some studies, such as Machado et al. (2007), only report on CO_2 emissions, while others report all greenhouse gas emissions in terms of carbon dioxide equivalents (CO_2e) or overall global warming potential, such as Ortiz et al. (2007). These discrepancies are noted here when reporting the findings.

The life cycle phases that are covered by each study also vary greatly. These boundary conditions are critical because they can have a significant effect on the LCA results (Dixon et al., 2003). A few studies include transport and collection of municipal waste (Tillman et al., 1998; Neumayr et al., 1997), as well as processes for final sludge disposal. The majority of the studies consider operation and maintenance, and a few also consider the construction phase. Most of the LCA studies exclude the end of life, or capital disposal phase, since this generally does not contribute significantly to the overall impact (Emmerson et al., 1995; Zhang and Wilson, 2000; Machado et al., 2007; Ortiz et al., 2007). It should be noted, however, that according to Machado et al. (2007) the end of life disposal of constructed wetlands contributes significantly to ozone layer depletion, and to a small extent to abiotic depletion and acidification. Still, construction and operation combined are major contributors (Ortiz et al., 2007). The following diagram (Figure 15) illustrates the system boundaries used for LCA of wastewater treatment systems. In the present study the LCA is confined to operation and maintenance of a hypothetical IAPS of identical capacity to that of the Belmont Valley WWTW demonstration system.

An ideal LCA study should include the impact of the sewer system in addition to a full life cycle impact of the downstream IAPS treatment plant, and incorporate any solids handling and disposal. Regarding sewer systems, because this information is lacking from many studies and since there is a dearth of information on sewer system-fed IAPS's (i.e. no commercial IAPS are integrated into sewerage networks in South Africa) we chose not to examine this component. However, a summary of available data from the *ecoinvent database* v 2.2*,* *http://db.ecoinvent.*org is provided. While, solids handling and disposal have significant environmental impact (Hospido et al., 2004; Gaterell et al., 2005), these are usually not included in studies that compare different wastewater technologies since focus is on operation and maintenance, and similar sludge disposal scenarios are assumed (i.e. incineration, land filling, or land application). There are, however, studies that consider different sewage sludge scenarios (Suh and Rousseaux, 2002; Houillon and Jolliet, 2005; Murray et al., 2008) and these show that environmental impact varies depending on the final fate of the solids. As far as IAPS is concerned there is no solids residue (i.e. sludge) or handling which places solids handling outside of the system boundaries. As emphasized by other authors, there is no need for sludge management during operation of an IAPS and the time in which sludge residues accumulate to require removal and disposal is of the order of decades (Green et al., 1995; Oswald et al., 1995).

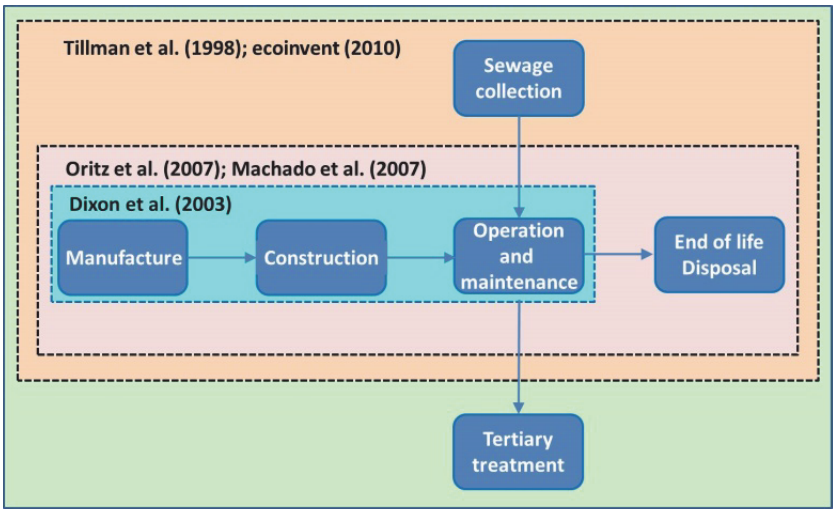

Figure 15 Common system boundaries used in LCA of wastewater treatment systems.

A life cycle approach assesses the system holistically; however LCAs may not be able to fully capture the environmental impact or benefits known from the system since LCA results are very specific to the assumptions that are made, and therefore cannot always be generalized to represent a system's environmental performance under all conditions (Tangsubkul et al., 2005). LCA studies may also fail to account for non-physical impacts, such as biodiversity, habitat, and aesthetics (Dixon et al., 2003). For example, as Brix (1999) points out, constructed wetlands serve a variety of functions in addition to treating wastewater, such as creating habitat for biodiversity and open space for public use. Similarly, IAPS creates a niche for enhanced biodiversity. Another potential difficulty arising from comparisons made across LCA studies is the use of different functional units. A functional unit is the measure used to quantify the impacts in a life cycle study. For wastewater treatment, common functional units include amount of energy consumed or emissions generated per person or per volume of treated water. The following assumptions and limitations apply to the present study:

i. The IAPS demonstration plant at the Belmont Valley WWTW is configured for research purposes and does not reflect a process flow for treating municipal sewage as would a commercial facility (cf. Figures 16 and 17). Consequently, pumping would be rendered obsolete in a commercial IAPS since all flow would be gravitationally driven and the orientation and layout of the IAPS would not be governed by the same spatial constraints as those of the Belmont Valley demonstration plant.

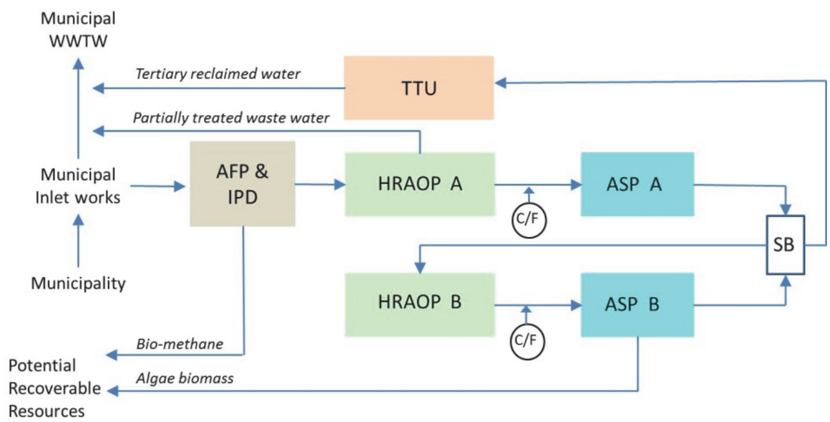

Figure 16 Schematic illustrating the process flow for the demonstration IAPS designed, constructed and operational at the Belmont Valley WWTW, Grahamstown. AFP=advanced facultative pond; IPD=in-pond digester; HRAOP=high rate algae oxidation pond; C/F= coagulation/ flocculation; TTU=tertiary treatment unit; ASP=algae settling pond; SB =splitter box.

ii. Measurement of process inputs in this exercise is restricted to the electrical pumps (which are excluded in a commercial IAPS) and the paddle wheel electric motors. Neither addition of chemicals nor fossil fuel is needed in operating and maintaining an IAPS.

iii. Measurement of inputs to and outputs from the IAPS biocatalysts are estimated from the standing biomass in the HRAOP (Table 17). The standing algae biomass, which varies seasonally, was estimated to be 0.15 $g \cdot \ell^{-1}$. Productivity is averaged across the HRAOPs and estimated at 5.79 $g.m^{-2} \cdot d^{-1}$ based on geographical location, where light and temperature are limiting factors, but not nutrients. Bacterial productivity in the HRAOPs is not accounted for.

iv. The Belmont Valley IAPS and the hypothetical commercial equivalent of identical capacity each service 500 PE and deliver the same water treatment efficiency, and geographical location is assumed to be the most likely source of variance in performance (all things being equal) between two comparable facilities.

v. Based on research outcomes, and to fulfil the requirement for a six day HRT in the HRAOP series, a commercial IAPS will have HRAOP surface area and volume 33% larger than the current demonstration facility, either in the form of an additional HRAOP (i.e. HRAOP 3 operating in parallel with HRAOP 2), or with HRAOP 1 and HRAOP 2 built 50% larger, or with HRAOP2 twice the size of HRAOP1 (Figure 17).

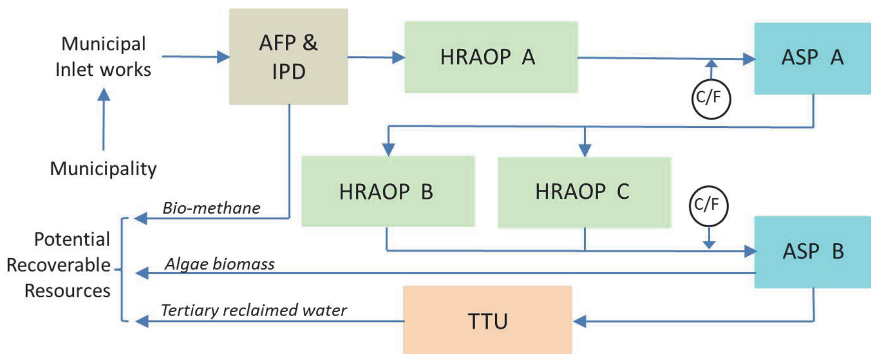

Figure 17 Process flow for a commercial IAPS. AFP=advanced facultative pond; IPD=in-pond digester; HRAOP=high rate algae oxidation pond; C/F= coagulation/ flocculation; TTU=tertiary treatment unit; ASP=algae settling pond; SB =splitter box.

vi. CO_2 fixation is estimated at 1.88 g $CO_2 \cdot g^{-1}$ algae. CO_2 release from algae post recovery from the HRAOPs through decomposition or processing is not considered as its use has not been defined. Options include digestion to methane and as a soil conditioner/fertiliser to displace conventional inorganic fossil fuel-derived fertilisers.

vii. The global warming potential (GWP) of gases released from the anaerobic digester raises an issue over allocation of the impact. It can be argued that the impacts of these gases (CO_2 and CH_4), which are released during the anaerobic decomposition of sewage, should be allocated to the source of its generation, and not to the wastewater treatment process (i.e. IAPS).

viii. The inlet works is calculated to generate a *pro rata* volume of compacted screenings (plastic, grit, etc.) and can be calculated based on the volume off-take to the Belmont Valley IAPS from the main municipal sewage feed into the WWTW. In view of a recent publication by the Department of Public Works (DPW, 2012) to direct the design process for designing the best and most appropriate wastewater process for sewage (up to 100 $k\ell \cdot d^{-1}$) which is generated by small scale (on site) operations the dearth of data on functional IAPS wastewater treatment systems in South Africa, an overview of the requirements for an inlet works is presented in some detail. Life cycle assessment of this infrastructure does not appear to be available. Even so, any environmental impact caused by installation of this infrastructure will be identical irrespective of the choice of wastewater treatment technology.

4.3 Measurements and Units

The functional unit is kW or MW expended per cubic meter of wastewater treated by the system. A second functional unit that will be evaluated is the amount of carbon dioxide generated to treat one cubic metre of wastewater. The system boundaries include the head of works, but not the maturation ponds. The entry point into the IAPS is after the inlet works where part of the inflow is diverted into the IAPS anaerobic digester and AFP. The exit point of the IAPS is its discharge from the final ASP. One debatable point is whether the CO_2 and CH_4 generated through the anaerobic digestion of sewage falls outside of the system boundaries and should be allocated to the producers (i.e. the users of the sanitation service upstream of the IAPS) or whether these GHG should be included in the system boundaries because it is a consequence of the IAPS process. For this exercise, the production of these gases will be included within the system boundaries. Details of all calculations are shown in Appendix D.

The mechanical equipment that forms part of the Belmont Valley IAPS includes two paddlewheels and five pumps. The power and daily usage of these electrical motors are:

(1) Paddlewheel drive for HRAOP *A* = 0.25 kW @24 $h \cdot d^{-1}$

(2) Paddlewheel drive for HRAOP *B* = 0.37 kW @ 24 $h \cdot d^{-1}$

(3) Raw sewage feed pump at inlet works to IAPS inlet box = 0.45 kW @ 24 $h \cdot d^{-1}$

(4) Submersible pump for return flow to WWTW = 0.25 kW @ 12 $h \cdot d^{-1}$

(5) Feed from first ASP *A* to HRAOP *B* = 0.5 kW @ 24 $h \cdot d^{-1}$

(6) Submersible pump from ASP *B* to drying beds = 0.25 kW @ 5 $h \cdot d^{-1}$

(7) Final return flow to inlet box pump = 2.2 kW @ 24 $h \cdot d^{-1}$

The mechanical equipment that forms part of the commercial equivalent includes up to three paddle wheel drives, but no pumps. The power ratings and daily usage of these electrical motors are:

(1) 3 × paddlewheels for high rate algae ponds, HRAOP *A* + HRAOP *B* + HRAOP *C* = 3 × 0.25 kW @24 $h \cdot d^{-1}$

Table 17 Input variables and constants for both the Belmont Valley and commercial IAPS.

Flow rate into IAPS ($m^3 \cdot d^{-1}$)	75
Person equivalent (PE)	500
Algae productivity ($g \cdot m^{-2} \cdot d^{-1}$)	5.79
GHG production (t $CO_2 \cdot kWh^{-1}$)	0.000843
CO_2 fixed ($g \cdot g^{-1}$ algae)	1.88
IPD conversion efficiency	85%
CH_4; lower heating value (LHV, $MJ \cdot kg^{-1}$)	50.1
CH_4 GWP as CO_2 (t $CO_2 \cdot t\ CH_4^{-1}$)	21

4.4 Results and Discussion

A sewer system that serves a large population has lower PE impact than a system that serves a small population.

The PE environmental impact associated with passage of sewage through the sewer system is inversely related to the number of people served by the system. This finding is evident in the GHG emissions resulting from the collection and treatment of municipal sewage.

As shown in Table 18, the PE impact increases with smaller sewer systems. Thus, a sewer system serving a large PE of 233 000 emits 0.03 kg $CO_2e \cdot PE^{-1} \cdot y^{-1}$ while a sewer for 806 PE emits 6.55 kg $CO_2e \cdot PE^{-1} \cdot y^{-1}$. In addition, there are studies indicating that sewer systems may have an impact in small conventional WWTW. Lassaux et al. (2007) considered a small conventional AS system and gravity flow sewers made of concrete, with pipes of 500 mm (i.d.) and showed that the second largest environmental impact is a result of sewer system construction preceded only by emissions from operation, and followed by emissions from water discharge to the environment (since it may include some untreated water).

Table 18 Global warming impact from sewer systems.

Class[A]	1	2	3	4	5
Person equivalents (PE)	233 000	71 113	24 864	5 321	806
Average length of sewer (km)	583	242	109.4	30.3	6.13
GWP 100Y (kg CO_2e)	665 250	632 700	597 500	565520	527 690
CO_2e per person per year	0.03	0.09	0.24	1.06	6.55

Note: CO_2e refers to carbon dioxide equivalents, which include carbon dioxide (CO_2), methane (CH_4) and nitrous oxide (N_2O) emissions. All emissions are expressed in terms of CO_2 by using the global warming potential of each gas to convert individual emissions to equivalent CO_2 emissions (*ecoinvent database v 2.2, http://db.ecoinvent.org*).
[A](Class 1 corresponds to 233,000 PE; Class 2 to 71,113 PE; Class 3 to 24,865 PE; Class 4 to 5,321 PE; Class 5 to 806 PE)

A second study, out of Europe, considered the life cycle impact of an unplasticized polyvinyl chloride (PVC-U) solid wall sewer system. A PVC-U solid wall pipe is commonly used in storm water and sewer systems. According to this study production of raw material for PVC pipes and polypropylene manholes, and to a lesser extent installation of the pipe system, were the major contributors to GHG emissions. Use, maintenance, and end of life stages appear to have been negligible. The carbon footprint for this sewer system, expressed per 100 m of pipe (250 mm i.d.) for a 100-year lifespan was 25.78 kg $CO_2e\cdot PE^{-1}\cdot y^{-1}$ (12 500 PE). This number is similar to results contained in the *ecoinvent database* for medium-sized systems (TEPPFA, 2010).

The environmental impact of a sewer system can be calculated based on distance from home to a centralized wastewater treatment system and the capacity of the WWTW.

This relationship depends on the size of the sewer system because the PE impact for large systems is lower than that of small systems (Table 18). To determine the environmental impact of connecting into a sewer system, the following equation can be used and takes the general form:

$$\text{Total kg } CO_2e = \text{Multiplying Factor} \times \text{Distance from central treatment (km)} \times \text{PE}$$

The multiplying factor for each class of system as defined in the *ecoinvent database* is obtained by dividing the total impact by the PE and the length of sewer in km (Table 19). Thus, the multiplying factor is expressed in kg $CO_2e\cdot km^{-1}\cdot PE^{-1}$. For example, Class 5 (806 PE at 6 km) corresponds to 1.07 kg $CO_2e\cdot km^{-1}\cdot PE^{-1}$. For Class 4 (5 322 PE at 30 km) the impact is 3.51×10^{-2} $CO_2e\cdot km^{-1}\cdot PE^{-1}$. Consequently, as the size of the sewer system increases, the multiplying factor decreases. It should be noted however that these equations provide only an estimate of the impact in kg of CO_2e per person per year.

Table 19 Equations to calculate global warming impact from sewers based on size of WWT facility.

Class	WWTW size (PE)	Equation
5	<806	n.a.
5	806-5,321	= 1.07 × distance (km) × PE
4	5,322-24,864	= 3.51×10^{-2} × distance (km) × PE
3	24,865-71,113	= 2.20×10^{-3} × distance (km) × PE
2	71,113-233,000	= 3.68×10^{-4} × distance (km) × PE
1	>233,000	= 4.09×10^{-5} × distance (km) × PE

Distance (km) is the distance from central facility
n.a. = not applicable

It is clear from this assessment that a 500 PE IAPS similar to the demonstration pilot at the Belmont Valley WWTW has negligible global warming impact from any associated sewer system. At 1000 PE, and assuming a sewer of 1 km, the global warming impact of the sewer system would be 1.07 kg $CO_2e\cdot PE^{-1}\cdot y^{-1}$.

Sewage collection represents a small portion of the overall impact of centralized wastewater technologies, especially for larger treatment systems.

While sewer collection does account for some life cycle GHG emissions its impact is relatively small compared to other life cycle phases such as plant construction, wastewater treatment, and disposal, at least for small lengths of pipelines (1 km). This observation stems from a comparison of emissions data on centralized treatment and the results are summarized in Table 20. For centralized wastewater systems that service 24 000 PE, the impact from sewers is less than 1% of all environmental impacts. For the smaller systems (i.e. < 24 000 PE), the emissions from sewage collection are 3.43% and 17.84%.

All WWTW are serviced by an inlet works: environmental impact unknown.

The purpose of screening is to remove larger floating and recalcitrant organic solids that do not aerate and decompose. The grit channel removes heavy inorganic matter like grit, sand, gravel, road scrapings and ashes. These particles are discrete and do not decay but can damage pumps making sludge digestion difficult.

Table 20 Contribution of sewer to overall global warming impact in wastewater treatment.

Sewer class	Collection (kg $CO_2e\cdot PE^{-1}\cdot y^{-1}$)	Sewage treatment (kg $CO_2e\cdot PE^{-1}\cdot y^{-1}$)	Total (kg $CO_2e\cdot PE^{-1}\cdot y^{-1}$)
1	0.03	20.35	20.38
2	0.09	27.14	27.23
3	0.24	29.10	29.34
4	1.06	29.95	31.02
5	6.55	30.16	36.71

According to a recent publication titled, "*Small Waste Water Treatment Works: DPW Design Guidelines*", guidelines to direct the design process for the best and most appropriate wastewater process for effluent that is generated by small-scale on-site operations up to 100 $k\ell \cdot d^{-1}$, are standard (DPW, 2012). Since the inlet works for the two IAPS under study would be identical, inclusion into the LCA will make no differences to the outcome. Even so, based on sewage flow into the IAPS, the inlet works is calculated to generate a *pro rata* volume of 2.9 $kg \cdot d^{-1}$ of compacted screenings (plastic, grit, ashes, etc.), which is disposed of to landfill.

Electrical paddlewheel motors are the only source and represent a small portion of the overall environmental impact during operation of an IAPS for sewage treatment.

The Belmont Valley WWTW IAPS treats approximately 75 $k\ell \cdot d^{-1}$ of municipal sewage. Of this 37.5 $k\ell \cdot d^{-1}$ is returned to the Belmont Valley WWTW as partially treated effluent after a two day detention in HRAOP *A*. From Table 21, and based on current operating configuration, half of the effluent (i.e. 37.5 kℓ) is gravitated to the second HRAOP, HRAOP *B*, where it is treated for a further four days. Energy consumption for complete treatment of this portion amounts to 2.34 $kWh \cdot k\ell \cdot d^{-1}$. Inclusion of the partially treated secondary water from HRAOP *A* yields an overall energy consumption of 2.23 $kWh \cdot k\ell \cdot d^{-1}$.

Table 21 Comparative performances of the current Belmont Valley WWTW IAPS and a commercial equivalent of identical capacity. Daily and annual energy requirements for operating these IAPS are shown and the impact on greenhouse gas emissions.

Energy inputs	Action	$kWh \cdot d^{-1}$	$kWh \cdot y^{-1}$	$kWh \cdot M\ell^{-1}$	$CO_2e \cdot y^{-1}$ (t)	$CO_2e \cdot M\ell^{-1}$ (t)	$CO_2e \cdot PE^{-1} \cdot y^{-1}$ (t)
Belmont Valley WWTW IAPS	Pumps	78.6	28 694	983	24.2	0.88	0.048
	Paddles	14.9	5 431	186	4.6	0.17	0.009
	Total	**93.5**	**34 126**	**1 169**	**28.8**	**1.05**	**0.057**
Commercial IAPS	Pumps	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.
	Paddles	18.0	6 570	225	5.5	0.20	0.011
	Total	**18.0**	**6 570**	**225**	**5.5**	**0.20**	**0.011**

n.a. = not applicable

In contrast to the Belmont Valley WWTW IAPS, a commercial IAPS of identical capacity is calculated to consume ~80% less energy and generate six times less environmental impact at 0.011 t $CO_2e \cdot PE^{-1} \cdot y^{-1}$ (Table 21).

The IPD at the base of the AFP generates additional emissions though conversion of organics into CO_2 and CH_4 and allocation of these GHG to the IAPS sewage treatment process affords three GWP scenarios.

Scenario 1: Any CH_4 global warming potential (i.e. 21 CO_2e) is reported as t CO_2 and added to the weight of CO_2 generated by operation and maintenance of the IAPS.

Scenario 2: CH_4 is flared to CO_2 and the weight of product CO_2 is added to the CO_2 by operation and maintenance of the IAPS.

Scenario 3: CH_4 is combusted to CO_2 and the energy recovered is converted into electrical energy and used in lieu of electrical energy derived from fossil fuel to power the IAPS. Thus, CO_2 generated by energy harvest and re-use is used to displace CO_2 generated from fossil fuels and the environmental impact of each scenario is illustrated in Table 22.

Table 22 Impact of operational scenario on GHG emissions from the IPD. Negative values indicate CO_2 sequestration.

	Value	**Units**
Scenario 1	15.21	t $CO_2 \cdot y^{-1}$
Scenario 2	3.25	t $CO_2 \cdot y^{-1}$
Scenario 3	-1.04	t $CO_2 \cdot y^{-1}$

It is evident from the calculated data that flaring of any harvested CH_4 substantially reduces the GWP of this potent GHG. A further reduction is possible if the anaerobic digester derived CH_4 is converted into electrical energy and then used to drive the paddlewheels in each HRAOP. In fact the latter approach has the overall effect of mitigating GHG emissions by facilitating carbon sequestration during municipal sewage treatment.

Microalgae in the HRAOPs sequester CO_2 from both the atmosphere and the effluent.

The ability of microalgae to utilize CO_2 as a substrate for energy production and growth is influenced by both irradiance and ambient temperature and is in many instances seasonal. An average algae productivity value has been calculated for HRAOPs based on location and the estimated photosynthetic capacity of the species present. Thus, HRAOPs function as a carbon sink as CO_2 is assimilated into algae biomass. This algae biomass has value and can be beneficiated in any of a number of ways, for example as a soil conditioner and fertiliser to offset use of fossil fuel derived inorganic fertilisers, as an energy feedstock for biogas and/or biofuels, as a substrate for the synthesis of bioplastics, and as a product for recovery of bulk commodities such as lecithin and protein (Perez-Garcia et al., 2011; Soratana and Landis, 2011; Chen et al., 2012; Chu 2012; Gómez et al., 2013). Results in Table 23 show that microalgae in the HRAOPs of an IAPS treating municipal sewage of design capacity 500 PE, sequesters about 6 tonnes of CO_2 per year or 12 kg $CO_2e \cdot PE^{-1} \cdot y^{-1}$.

Table 23 Comparative projections of CO_2 sequestration and O_2 production in HRAOP of the Belmont Valley WWTW IAPS and a commercial equivalent of identical capacity. The commercial HRAOPs have 33% additional surface area.

	Belmont Valley IAPS	**Commercial IAPS**
CO_2 fixed ($kg \cdot d^{-1}$)	10.88	16.32
CO_2 fixed ($t \cdot y^{-1}$)	3.97	5.96
CO_2 fixed ($t \cdot M\ell^{-1}$)	0.14	0.20
O_2 produced ($kg \cdot d^{-1}$)	7.91	11.87
O_2 produced ($t \cdot y^{-1}$)	2.89	4.33
O_2 produced ($t \cdot M\ell^{-1}$)	0.10	0.15

The results in Table 23 notwithstanding, GHG production potential for a commercial IAPS and the demonstration system at the Belmont Valley WWTW differ considerably. These differences are attributed to the extensive use of pumps at the Belmont Valley WWTW needed to offset design constraints and to counter topography. Table 24 illustrates that for a properly designed IAPS system, in which the process flow is driven by gravity, considerably less GHG emissions can be achieved. Biological processes in the IPD also release GHG in the form of CO_2, CH_4 and N_2O, during the breakdown of organic material. No additional energy inputs are required as all of the energy used in decomposing the organic component in the effluent is derived from chemical energy locked in the organic material itself. Microalgae CO_2 sequestration in the HRAOP further reduces the net CO_2 emissions from the IAPS and this can be enhanced by increasing algae productivity, and thus the biomass while CH_4 capture and re-use in electrical generation offsets emissions from fossil fuel derived electricity.

In a rationally designed IAPS, where pumping is replaced by gravity flow, where algae biomass production is enhanced and where CH_4 is recovered and used to generate electricity, GHG emissions and overall environmental impact can be substantially reduced. Furthermore, were the biotic GHG (i.e. gases generated by anaerobic digestion) to be allocated to the producer of the effluent, IAPS treating municipal sewage could serve as a CO_2 sink while achieving effluent treatment.

Table 24 Comparison of GHG impacts between the Belmont Valley IAPS and a commercial equivalent of identical capacity. Note that the reallocation of GHG produced by anaerobic digestion to the producers of sewage influences the carbon footprint of the installation.

Belmont Valley IAPS Net effects	**t $CO_2 \cdot y^{-1}$**	**t $CO_2 \cdot M\ell^{-1}$**	**Commercial IAPS Net effects**	**t $CO_2 \cdot y^{-1}$**	**t $CO_2 \cdot M\ell^{-1}$**
GHG production (abiotic)	28.77	0.99	GHG production (abiotic)	5.54	0.19
CO_2 sequestration	3.97	0.14	CO_2 sequestration	5.96	0.20
GHG production (biotic)	15.21	0.52	GHG production (biotic)	15.21	0.52
Net GHG production	40.01	1.37	Net GHG production	14.79	0.51
If biotic GHG reallocated	24.80	0.85	If biotic GHG reallocated	-0.42	-0.01
If CH_4 energy used	20.52	0.70	If CH_4 energy used	-4.70	-0.16

4.5 Summary and Conclusions

To gain insight of the environmental impact of operating an IAPS, a concept system, of identical capacity to the demonstration plant (i.e. 500 PE) and configured for commercial use (i.e. complete gravitational flow and HRAOPs 33% larger) was used to model both energy flows and GHG emissions for the LCA.

Recourse to the literature revealed that a 500 PE IAPS similar to the demonstration pilot at the Belmont Valley WWTW has negligible global warming impact from any associated sewer system. However, GHG emissions increase as the size of the sewer feeding a STP declines. Sewers are coupled to STP via the inlet works. Recent information on appropriate inlet works designs for all WWTW treating up to 100 $k\ell \cdot d^{-1}$ will contribute to emissions and full life cycle data is needed to determine more accurately the LCA of a fully operational commercial scale IAPS. Even so, the emissions impact of such an inlet works will be identical across all technologies.

The environmental impacts of a commercially operated IAPS were as expected, just 20% those of the Belmont Valley demonstration IAPS, with a net GHG production of 0.51 t $CO_2 \cdot M\ell^{-1}$ of wastewater treated. After reallocation of biotic greenhouse gases and with energy re-use from harvested biogas, the 500 PE IAPS has overall emissions for operation of -0.16 t $CO_2 \cdot M\ell^{-1}$ of wastewater treated. This, coupled with the impact from sewers which is typically less than 1% and not applicable for class 5 sewers (i.e. <806 PE), strongly indicates that IAPS as a wastewater treatment technology has the potential to decrease the life cycle impacts of wastewater treatment when used on small scale.

5. Implementing IAPS as a WWT Technology in South Africa

Challenges resulting from the dire state of water and sanitation infrastructure and service delivery in South Africa demand that closer scrutiny be given to all available WWT technology solutions. Thus, the present study had as its major objective a re-evaluation of IAPS as a sewage treatment technology and, to seek and provide answers to questions posed by authorities prior to implementation of this technology. Within this framework the aims of the present study were:

- Determine the state of the art of IAPS as a WWT technology;
- Re-evaluate the design and operating configuration of the Belmont Valley IAPS and determine the parameters needed to ensure system performance and sewage treatment efficiency to produce a quality treated water; and to,
- Identify and recommend best practice to ensure successful implementation and technology transfer.

The information contained in this report has emphasised that IAPS is a contemporary WWT technology and has been implemented at pilot, demonstration, and full commercial scale in many countries. Furthermore, the HRAOP component of IAPS is currently the subject of intensive study aimed at increasing both WWT efficacy and the production of biomass. The latter is of particular importance given current global interest in the water-energy-food nexus (Murphy and Allen, 2011), algae-to-energy systems (Jeon et al., 2013; Rogers et al., 2014; Psycha et al., 2014), CO_2 sequestration and the mitigation of climate change (Klinthong et al., 2015), and as a source of commodity products for economic growth and development (see, DST Strategic Plan for the Fiscal Years 2015-2020; the NERC UK report *A UK Roadmap for Algal Technologies 2013*; the U.S. Senate Report 113-164 – Agriculture, Rural Development, Food and Drug Administration, and Related Agencies Appropriations Bill, 2015; and, Algae Industry Project Book 2015).

From a technical perspective, the research described in this report has confirmed that IAPS when configured and operated correctly, and with appropriate tertiary treatment, yields a treated water that meets the general authorisation limit values for either irrigation or discharge to a water resource that is not a listed water resource. Thus, motive and opportunity exist for the implementation of commercial scale IAPS in South Africa by both district and local municipalities.

This chapter sets out a framework and plan for the establishment and implementation of IAPS for municipal sewage treatment. The downstream advantages of efficient and effective WWT across South Africa are obvious with clear benefits to, amongst others, the environment, agriculture, tourism, and community health. However, challenges around WWT, experienced by the majority of municipalities (lack of skilled and trained staff, old and outdated infrastructure, limited budgets, etc.) as indicated in the Green Drop Report (DWS, 2012), tend to mask the value and as yet unexplored potential of wastewater. In addition to the financial and ecological benefits of IAPS, this chapter highlights the potential synergies between WWT and food security. This link is emphasized on the basis of the critical importance of low input costs towards sustainable and financially viable food security, hence the essential role of IAPS in supplying renewable energy, biomass and clean water as low-cost vectors for sustainable food security in the most vulnerable groups of society. However, in the absence of real-time process product data, quantifcation of the synergy between IAPS as a WWT technology and food security awaits implementation and operation of a full-scale system under South African conditions.

To date, five partially funded projects for the implementation of IAPS as a WWT technology have been initiated with various district and local municipalities. However, and as mentioned elsewhere in this report, none of these projects has materialised. Two of the projects are reiterated below as case studies to highlight the need for an appropriate framework and plan for implementation of IAPS technology.

Case Study 1 – A proposal based on the above concept titled "Demonstration Project for the Integral Algae Ponding System (IAPS) for Wastewater Treatment" was submitted and ZAR 7.6 million awarded with a planned start date of 1 April 2011. The project proceeded to 'preliminary design report' stage. Thereafter, and most unfortunately, the partner municipality withdrew due to an apparent inability to meet commitments and the project was eventually terminated (effective February 2013), preventing both implementation of IAPS technology and realisation of the downstream benefits (i.e. clean water, renewable energy, and biomass). The overall project objective was to demonstrate the robustness and sustainable characteristics of IAPS and to quantify the system capabilities and outputs. Measurable deliverables were:

- Increase municipal WWTW capacity by up to 2500 PE;
- Produce 4.5 Gℓ clean water per year for irrigation or infiltration;
- Confirm that energy required for IAPS is less than 10 $kWh \cdot y^{-1} \cdot PE^{-1}$ compared to 25 $kWh \cdot y^{-1} \cdot PE^{-1}$ for AS;
- Confirm that construction cost for a 2 $M\ell \cdot d^{-1}$ IAPS is at least 30% less than the construction cost of a 2$M\ell \cdot d^{-1}$ AS plant;
- Produce 5000-7500 kg algae-based fertilizer;
- Promote the technology to at least 50 municipalities after first year of operation.

Case Study 2 – A proposal titled "The Establishment of an Integrated Algal Pond System (IAPS) for the Treatment of Municipal Waste Water" was submitted and ZAR 8.5 million awarded with a planned start date of 1 October 2014. At time of writing, this project had progressed to 'preliminary design report' stage. Unfortunately, the partner municipality finds itself in the unenviable situation of having exhausted all funds it had committed to this project. Although this municipality is in negotiations to secure the necessary monies, the project funders have indicated that without co-funding there exists a real possibility that the project will be terminated. Should this be the eventual outcome it will again prevent both implementation of the technology and realisation of the downstream benefits (i.e. clean water, renewable energy, and biomass). The overall project objective was to upgrade capacity to 2.07 $M\ell \cdot d^{-1}$ (30 y design horizon) by converting the current conventional oxidation pond WWTW to an IAPS to demonstrate the robustness and sustainable characteristics of IAPS and to create a viable platform to launch sustainable food security and job creation. Measurable deliverables were:

- Increase municipal WWT capacity to 12 500 PE;
- Produce water suitable for irrigation and/or infiltration;
- Confirm that energy required for IAPS is less than 10 $kWh \cdot y^{-1} \cdot PE^{-1}$;
- Confirm the construction costs for a 2 $M\ell \cdot d^{-1}$ IAPS;
- Integrate co-product streams for valorisation via community engagement;
- Technology transfer and operator training.

In both cases the proposed projects addressed core concepts of IAPS technology and set out to facilitate further exploration of the synergies between WWT and food security at full commercial scale. Successful project outcomes would also have provided the platform for elaboration of sustainable municipal waste-to-algae-to-energy systems.

The implementation of IAPS as a municipal sewage treatment technology can occur in several ways. First, an entire IAPS system can be installed as 'greenfields'. Second, an existing pond-type WWTW (e.g. WSP) can be upgraded or converted to an IAPS as a 'brownfield' installation. Third, component parts of an IAPS (e.g. AFP and/or HRAOPs) can be used in an existing WWTW, including AS, to either intercept culprit flows or to enhance capacity of the works. For a 'greenfield' installation, several additional components must be in place prior to IAPS installation or, accounted for in the planning and design stages. Included are: a reticulated sewerage system, an inlet works, a MP series or holding reservoirs, and end use. Thus, for a successful project outcome it is critical that customer need be identified and recognised in advance. In short, this requires; identification of the need/problem; and, establishing whether true value and benefit will be derived after technology implementation.

5.1 Towards a Strategy for Implementation of IAPS

Location – The advantages of using IAPS to treat municipal sewage include the perceived low cost of construction, operation and maintenance, and the potential of process products such as treated water for recycle and re-use, biogas and biomass to contribute to food security. To realise the full benefit of IAPS as a WWT technology, ideal locations are typically small towns (including villages and settlements) and peri-urban areas. Although peri-urban areas are neither geographically nor conceptually well defined, these are typically located somewhere in-between the urban core and the countryside and have been traditionally regarded, from an urban planning perspective, as ground for urban sprawl (i.e. housing developments) and the location of regional and trans-regional infrastructures (e.g. factories, etc.).

Problem/need – Identification of the customer problem/need is usually case specific but typically involves the urgent requirement by a municipality for water and sanitation infrastructure or the upgrading of existing infrastructure that is either of insufficient capacity or in a state of disrepair. Secondary needs arise where there is demand for process products e.g. water for irrigation, biogas for electrical energy, and biomass for processing to compost and/or fertilizer to support high-value horticulture and agriculture. These secondary needs can also be used as drivers to support new local economic and employment initiatives.

Engagement – An appropriately informed decision to implement a water and sanitation technology is usually taken by the municipality after consultation with the community it serves and either the water or engineering sectors, or both. Once funds for the project have been received, engagement with technology service providers can commence. In the event IAPS is selected as the technology of choice, EBRU is the current technology service provider in South Africa. In response to enquiries and opportunities, EBRU will engage a consulting engineering concern to assist with project roll out. This usually occurs as follows: site visit and data retrieval, compilation of an inception report, preliminary process design, final design report, preparation of tender documents, tender process, appointment of contractors, construction, commissioning and optimisation, and operator training and handover. It is estimated that the entire process from first site visit to handover should occur within a 24 month period. Community engagement is key and is encouraged throughout and where possible, use is made of local engineering services and contractors.

Agreement – The implementation strategy outlined above is underpinned by several tiers of agreements, letters of intent, and signed documentation including memoranda of understanding between the technology service provider, the consulting engineers, and the customer municipality. Even so, and irrespective of whether partial or full funding was obtained, the case studies described earlier illustrate that implementation of an IAPS project and completion thereof are challenges that have yet to be successfully overcome.

A major factor contributing to termination of the project implementation process appears to be the inability of customers to meet financial obligatons. Reasons for this are varied and may be as a consequence of either administrative oversight, i.e. municipal funds were never officially committed to the project, or over design, i.e. available funding and planned capacity were not commensurate. Similar problems can arise if the water and sanitation project forms part of a much larger civil engineering project and is dependent on successful completion of the latter. In mitigation, and where funding is from a third party (e.g. DWS, DPW, WRC, etc.), one possible mechanism that can be used is to require the customer to enter into a 'scope of work' or similar legally binding agreement in which the financial obligations of all funding parties are clearly identified. This would ensure that sufficient funding is available for the project to be implemented completely.

5.2 Proposed Steps for Project Implementation

Based on the preceding discussion it is proposed that the major steps outlined in Table 25 be followed for IAPS project implementation in South African municipalities once funding has been approved/awarded.

Table 25 Major steps for implementation of IAPS as a municipal wastewater treatment technology.

Action	Response
Letter: Approval/award of funds	Letter of acceptance/commitment
Meeting: Project initiation and site visit	• Host meeting and site visit • State problem/need clearly • Supply requested information
Report: Project inception and provisional costing	• Ensure report meets customer vision • Ensure problem/need is addressed • Ensure funding is sufficient for budget
Agreement: MoU – technology provider, consulting engineers, customer	Prepared jointly and effective on date of signature
Agreement: SoW – third party funder, customer	Prepared jointly and effective on date of signature
Report: Preliminary process design	• Ensure report meets customer vision • Ensure problem/need is addressed • Ensure funding is sufficient for budget
Report: Final design and budget	• Ensure report meets customer vision • Ensure problem/need is addressed • Ensure funding is sufficient for budget
Tenders	Normal procurement process
Appointment of contractors	Normal procurement process
Construction	On-site monitoring and progress
Commissioning	On-site monitoring and progress • Operator training
Report: Final project report	Handover

The actions/responses listed in Table 25 are not exhaustive and are simply a guide to be used to avoid the situation of not being able to meet financial obligations. All other meetings, reports, agreements, and appointments required in project management to ensure a successful outcome are largely the domain of the consulting engineers.

6 Concluding Remarks

Following a WRC-funded technology transfer project, a demonstration IAPS was constructed at the Belmont Valley WWTW (Grahamstown) and used to develop an integrated management approach to municipal sewage treatment (Rose et al., 2002a; 2007). This was followed by a series of studies carried out to demonstrate the versatility of the system for the treatment of different wastewaters (Rose et al., 1996; Rose et al., 2002b; 2002c). In addition, and as discussed elsewhere in this report, there are many examples of other wastewaters that have successfully been treated at pilot scale using IAPS technology. The purpose of the work described in this report was to re-evaluate IAPS as a WWT technology for South Africa to inform the municipal decision-making process. Thus, it is clear that IAPS is a contemporary WWT technology and that this bioprocess is the subject of intense research worldwide and for a variety of applications. Most notably, IAPS is being explored in relation to the water-energy-food nexus due to the potential of the system to generate clean water for re-cycle and re-use, methane-rich biogas, and biomass for high-value horticulture/agriculture and/or, as a renewable source of commodity products (Cowan, 2010; Murphy and Allen, 2011; Cowan and Mlambo, 2015). In view of a recent scoping study aimed at introducing the biorefinery concept to the South African wastewater sector (Verster et al., 2014) it is suggested that IAPS represent an ideal platform for realisation of municipal biorefineries. Technical aspects of the studies described in this report provide strong support for this conclusion.

First, this report has confirmed, using the Belmont Valley WWTW IAPS as a South African example, that when configured and operated correctly, and with appropriate tertiary treatment, this technology yields a treated water that is compliant for pH, DO, EC, COD, nitrate/nitrite-N, ammonium-N, phosphate-P, TSS and total coliforms, in terms of the DWS general authorisations for either irrigation or discharge. The IPD-AFP of this IAPS produces a methane-rich biogas with equivalent energy yield of ~55 $kWh.PE^{-1}.y^{-1}$, while a nutrient-rich biomass (>3 tonnes $DW \cdot y^{-1}$) is available for preparation up to 15 kℓ of liquid fertilizer concentrate. Second, LCA modelling, to map both energy flows and GHG emissions for operation of IAPS as a municipal WWT technology, revealed that a commercial system would yield -0.16 tonnes $CO_2 \cdot M\ell^{-1}$ of wastewater treated indicating a technology with an ability to mitigate climate change. Thus, there is substantiated motive to implement IAPS for municipal wastewater treatment in South Africa.

Opportunities to construct commercial scale IAPS in South Africa have arisen in the recent past but these projects have unfortunately failed. As a consequence, this report has also elaborated a strategy and proposed a step-by-step procedure for implementation of IAPS-based municipal sewage treatment projects. This strategy emphasises the four key points that must be addressed in any planned implementation of IAPS technology viz. location, customer problem/need, engagement, and agreement.

The current state of municipal WWT infrastructure in South Africa, the lack thereof in many instances, and the continual need to improve water and sanitation service delivery dictates that an IAPS process information document be available and which answers most if not all questions posed by authorities wanting to use this technology. This report has been drafted specifically for that purpose. Lastly, and as echoed by Verster et al. (2014) the wastewater sector is positive towards integrating WWT with recovery of products to add value and simultaneously, close the nutrient cycle. In response of ever diminishing natural resources together with environmental degradation and rapidly accelerating climate change, it is becoming increasingly urgent that a zero-waste paradigm be adopted in the design and implementation of technology solutions to facilitate re-cycle and re-use.

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Composite sampling is the best method for determining performance of a WWTW. Water from the required sampling point in the process flow is continuously collected for 24 h using a set-up similar to that illustrated in the following diagram (Fig. A.1).

Figure A.1 Setup for collection of a composite sample using a standard municipal dustbin over a 24 h period.

Sub-sampling – A 500 mℓ bottle is rinsed using a small amount of the sampled water from the collection vessel. Discard the washing water and fill bottle to the brim and take to the laboratory for analysis (APHA, 1998).

Physicochemical analyses – Electrical conductivity and temperature is measured using an EC Testr 11 Dual range 68X 546 501 detector (Eutech Instruments, Singapore), while the dissolved oxygen is determined using an 859 346 detector (Eutech Instrument, Singapore) or similar. The pH is measured using a standard laboratory pH meter after calibration. Test kits available from many suppliers and based on protocols described in Standard Methods (APHA, 2008) are used to determine nitrate-N (Merck, 1.14773.0001; 20-300 mg.ℓ^{-1}), *ortho*phosphate (Merck, 1.14848.0001; 0.01-5 mg·ℓ^{-1}), ammonium-N (Merck, 1.14752.0001; 0.013-3.86 mg·ℓ^{-1} and chemical oxygen demand A and B (Merck, 1.14538.0065 and 1.14539.0495; 100-1500 mg·ℓ^{-1}) according to the manufacturer's instructions. Whatman Grade 1 filter paper (11 µm, 1 002 125) is used to determine COD.

Microbial analysis – MacConkey (HG0 00C 92 500) and m-Fc (HG0 0C 120 500) agar (Biolab, South Africa) are made up according to the manufacturer's instructions. A 100 µℓ aliquot of the water sample is spread onto the agar plates. The Petri dishes are then incubated at 30 and 45 °C respectively for 24 h and the appropriate indicator colonies counted.

Total suspended solids – A pre-weighed Whatman Glass Fibre A (1.6 µm, 1 820 110) filter is placed in an oven at 105°C for 1 h. Then, 20 mℓ of water sample is filtered. A 20 mℓ milli-Q water serves as the blank. The filter paper is then oven dried overnight at 105°C according to Standard Methods (1998) and TSS determined as follows;

mg Total Suspended Solids/ℓ = ((A-B) × 1000)/mℓ sample

where A = weight of filter (or filter and crucible) + residue in mg, B = weight of filter (or filter and crucible) in mg, and C = mℓ of sample filtered.

Biocatalyst productivity – Obtain a sample of mixed liquor by taking six 100 mℓ samples of water from HRAOP B directly opposite the paddlewheel and transfer to the laboratory;

Measure the sample volume.

Remove a filter paper disc (0.45 μm) from the desiccator and record its weight.

Place the filter paper into the filter holder connected to a vacuum flask (low vacuum) and wet using a small amount of water to create a good seal.

Stir the sample of mixed liquor to ensure homogeneity and transfer 5 mℓ from a graduated measuring cylinder into the filter holder using a Pasteur pipette.

After the mixed liquor has passed through the filter, run three portions of 10 mℓ distilled water through the filter holder to rinse any particles that may be adhered to the glass. Allow the vacuum pump to run an additional three minutes. This will help remove any extra water from the filter before drying.

Switch the vacuum pump off and remove the filter from the filter holder and place in a weighing dish. Repeat above process for as many replicates as might be needed.

Place the filter(s) into a drying oven, set to 103-105°C, for one hour. Upon drying, replace filter discs into a desiccator for 30 minutes. After 30 minutes in the desiccator, the filter discs are weighed and MLSS calculated as follows:

MLSS (mg.ℓ^{-1}) = [(A - B)×1000]÷[Volume of sample in mℓ]

where;
A, is the sample + filter disc weight
B, is the weight of the filter disc

Appendix B: Recommended Process Flow for Commercial IAPS

Operating configurations for the Belmont Valley WWTW 500 PE IAPS. Current configuration releases 37.5 $k\ell \cdot d^{-1}$ of partially treated wastewater which is returned to the Belmont Valley WWTW inlet works. Correct operating configuration demands that 37.5 $k\ell \cdot d^{-1}$ be returned to the IAPS inlet in order to dilute the BOD_{ult} load from 80 to the designed 40 $kg \cdot d^{-1}$. Commercial operation of the IAPS requires that the HRAOP component of the IAPS (or implemented as standalone remedial technology) be configured to winter conditions with a total HRT >6 days. The design and configuration is based on HRAOP of 500 m^2 and 150 $k\ell$.

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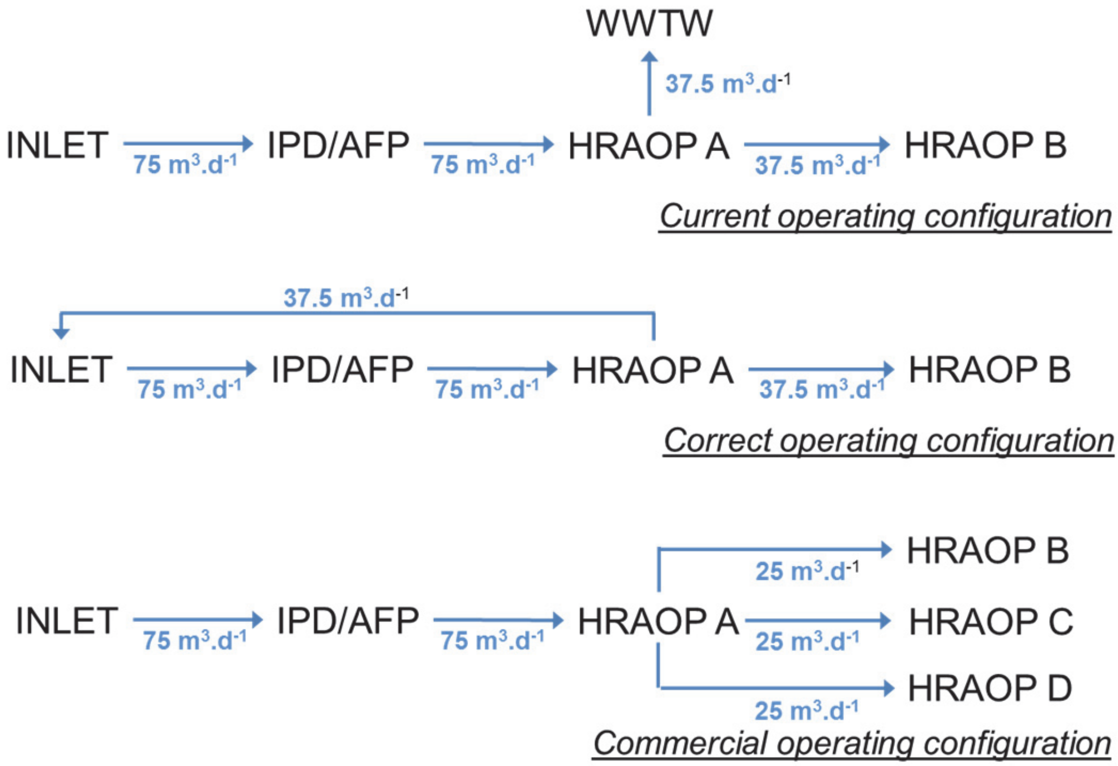


Appendix C: Design of Tertiary Treatment Units

Maturation pond design and construction – Maturation ponds contain the secondary treated effluent and are normally located after a conventional municipal system. The main function of MPs is to provide additional polishing to the water by removing pathogens from the final effluent and decreasing settleable solids and dissolved plant nutrients (Mara, 2005).

The quantity and size of MPs ais determined by the final bacteriological quality required and by standards that apply to effluent discharge. In South Africa, the general limit for faecal coliforms is 1000 counts.100 $m\ell^{-1}$. In the present work, three MP's were used for the IAPS and these were configured in series which is reported to be suitable for treating wastewater from a single facultative pond (Mohammed, 2006). Similarly, Mara (2005) indicated that a series of MPs rather than a large individual MP provides better hydraulic efficiency and pathogen removal.

Maturation pond depth is usually between 1 and 3 metres with an extended retention time for maximum pathogen removal. Shallower ponds (0.4 m) are however more effective at decreasing pathogens and less land area is required (Kruzic and White, 1996). For the present study, MP 1 (Fig. C.1) was constructed with a water depth of 1 m, from an inlet set at 1.2 m, to prevent water overflow and increase UV light penetration. As stated by Pearson et al. (1995), positioning and depth of inlet and outlet pipes tends to be more important for effluent quality and treatment competence than pond geometry itself. To achieve the effluent quality required MPs require a long retention time – usually in excess of 11 d and up to 37 d. According to Craggs (2005), a retention time of between 10 and 20 d is sufficient for faecal coliform removal to levels less 1000 MPN per 100 mℓ. Thus, a total retention time across the MP series was, based on flow rate, configured to 12 d, i.e. 4 d in each pond.

MP 1 was constructed using PVC lining (5 × 1.2 m) which was supported by steel fencing on the outside. The baffle, also of PVC lining is supported at the bottom by a weight to allow water flow under the baffle. MP 2 and 3 are 1 kℓ plastic containers equipped with an identical baffle system and the systems plumbed using 15 mm piping.

Hydraulic retention time and flows through the MP series were constrained by the size of the receiving unit(s). Thus, the first MP with area 19.63 m^2 and depth 1.02 m, allowed for a holding volume of ~20 m^3. Using the expression;

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

where, A= area (m^2); Q= influent flow ($k\ell{\cdot}d^{-1}$); Θ_{m1}= retention time (days) and D = depth, the flow rate to MP 1 was 4.9 $k\ell{\cdot}d^{-1}$, with both MP 2 and MP 3 receiving effluent at a flow rate of 0.2 $k\ell{\cdot}d^{-1}$ to give a HRT of 12 d (4 d in each pond) throughout the MP series (Fig. C.3).

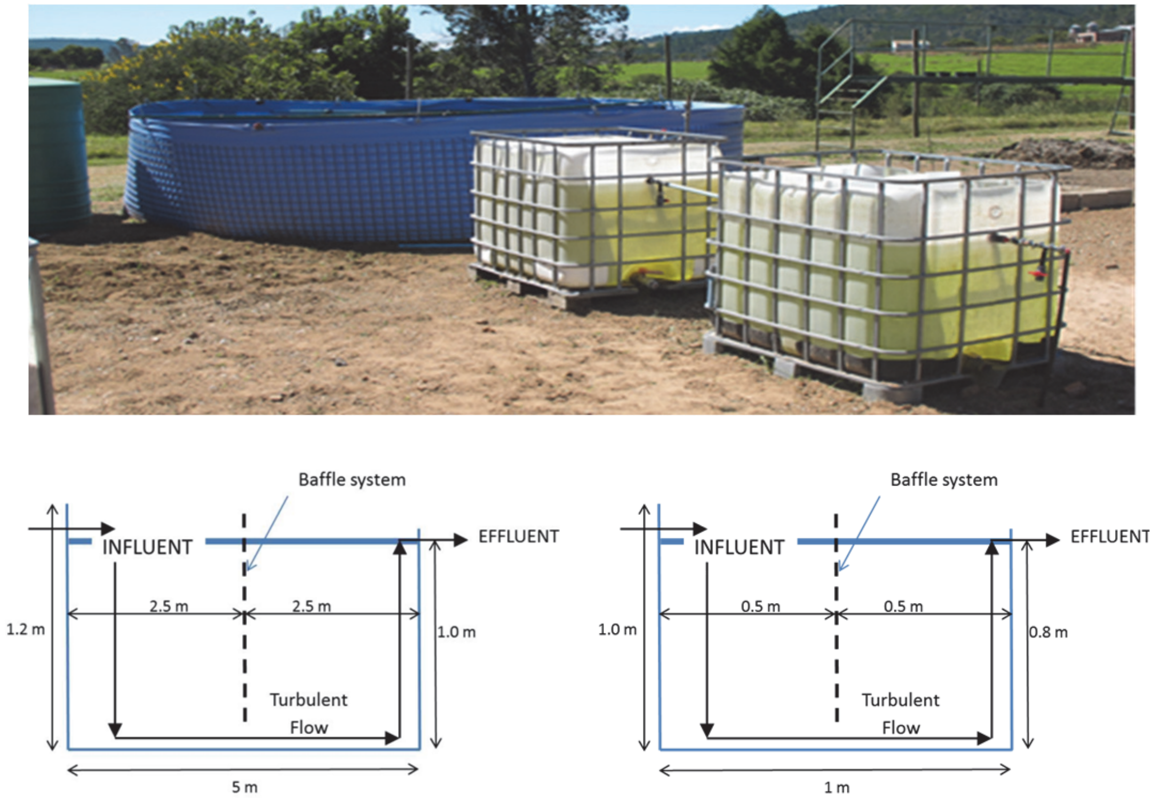


Figure C.1 Configuration of a maturation pond series to serve as a tertiary treatment unit (TTU) to IAPS. Top: layout of the MP series. Bottom left: schematic of MP 1 with single baffle to prevent water from short-circuiting and turbulence. Water level is at 1 m and volume of 20 kℓ. Bottom right: schematic of MP 2 and 3 each with single baffle to prevent short-circuiting and turbulence. Water level is at 0.8 m and volume of 1.0 kℓ.

Slow sand filter design and construction – Slow sand filtration (SSF) was one of the first modern treatment techniques used for the purification of drinking water, produces a high quality filtrate, and is a method employed extensively throughout the potable water industry (Ellis, 1987). Sand filtration is a biofilm-driven process and relies on removal of nutrients by diverse microbial populations and the mineralization and biodegradation of organic matter (Gaur et al., 2010). As a consequence, this method has been used widely to treat surface waters and secondary effluents. Sand filters are designed on the basis of hydraulic and organic loading capacity (Tyagi et al., 2009).

For the purposes of the present study, SSFs were constructed in 1500 L JoJo® tanks (1.5×1 m internal diameter) and comprised a 0.2 m layer of gravel covered by Geofabric (BIDIM®) followed by a 0.5 m layer of fine sand covered in Geofabric and a 0.8 m head of water (Figure C.2). A high water head pressure was necessary to overcome the effect of the Schmutzdecke (biofilm) which can cause clogging and decrease water flow into and through the system (Massmann et al., 2004). Thus, and according to recommendations (McNair et al., 1987; di Bernardo, 2002) two SSFs were constructed in parallel – one in operation while the second is cleaned (scraping the biological layer from the surface of the sand). To reduce time to cleaning, a layer of BIDIM® covers the fine sand layer preventing algae ingress into the substrate and facilitating easy and rapid cleaning.

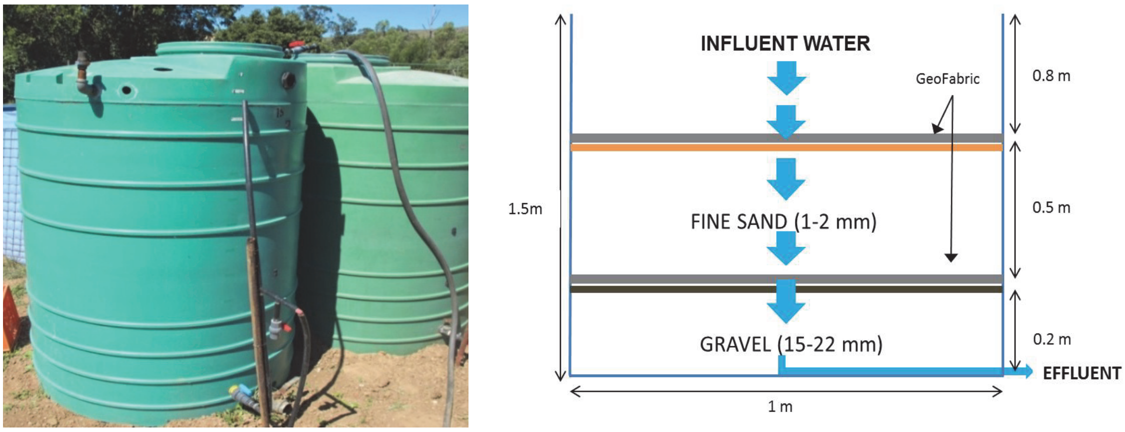
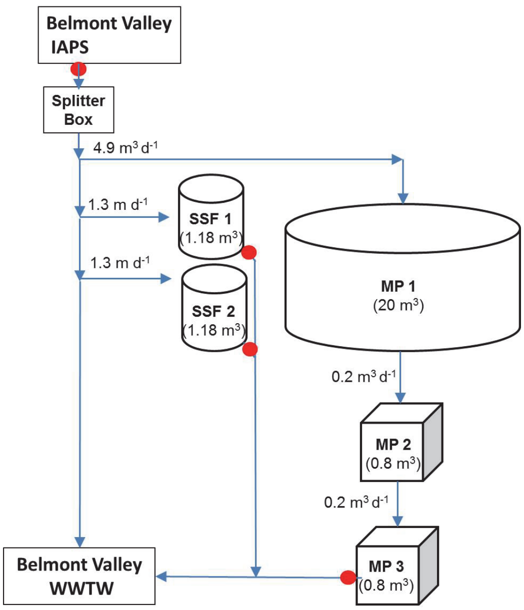


Figure C.2 Slow sand filtration as a tertiary treatment unit (TTU) to IAPS. Left: slow sand filter (SSF) contained inside a 1500 L Jojo® tank. Right: schematic of the SSF comprising of a layer of fine sand overlaying a layer of gravel each separated by GeoFabric.

Hydraulic retention time and flows through the SSFs were constrained by the size of the filter(s). Thus, the SSFs with area 0.785 m^2 and volume 1.18 kℓ, allowed for a hydraulic loading rate of ~1.3 $m{\cdot}d^{-1}$ calculated using the expression;

$$A = Q/HLR$$

where, A = area; Q = flow rate ($k\ell{\cdot}d^{-1}$); and HLR= hydraulic loading rate ($m{\cdot}d^{-1}$) and the design layout and process flow are shown in Figure C.3. It is important that the hydraulic loading rate of the SSF range between 0.37 and 0.56 $k\ell{\cdot}m^{-2}{\cdot}d^{-1}$ to prevent breakthrough (Middlebrooks et al., 2005).



C.3 Design layout and process flow of the TTUs positioned after IAPS treatment of domestic wastewater. IAPS effluent, after algae settling, is distributed to the MP series and SSFs from a splitter box. SSFs receive 1.3 $m{\cdot}d^{-1}$ whereas MP 1 receives 4.9 $k\ell{\cdot}d^{-1}$and MP 2 and 3 each 0.2 $k\ell{\cdot}d^{-1}$. SSF = Slow Sand Filter; MP = Maturation pond series. Sampling points are shown in red.

Controlled rock filter design and construction – Rock filtration is a low-cost wastewater cleaning system used to polish waste stabilization pond effluent (Hamdan and Mara, 2009). It cleans wastewater by allowing algae to attach to the rock surface as the water passes through the rock bed. The algae degrade biologically over time (Middlebrooks et al., 2005) through senescence and programmed cell death and release nutrients which are then consumed by bacteria that grow on the surface of the rocks (Mara, 2013).

A Controlled Upflow Rock Filter (RF) was constructed in series comprising three plastic containers each measuring a 1.0 × 1.0 m and containing a gravel sand bed 0.6 m in depth (Fig. C.4). CRF's have an average particle size of between 5-20 mm and the gravel particles used in the present work were 15-22 mm in diameter (Hussainuzzaman and Yokota, 2005). The positioning of the inlet piping (15 mm) to the three gravel sand filters was at the bottom of the CRF's which allowed upflow of water into the system for improved efficiency (Middlebrooks et al., 2005).

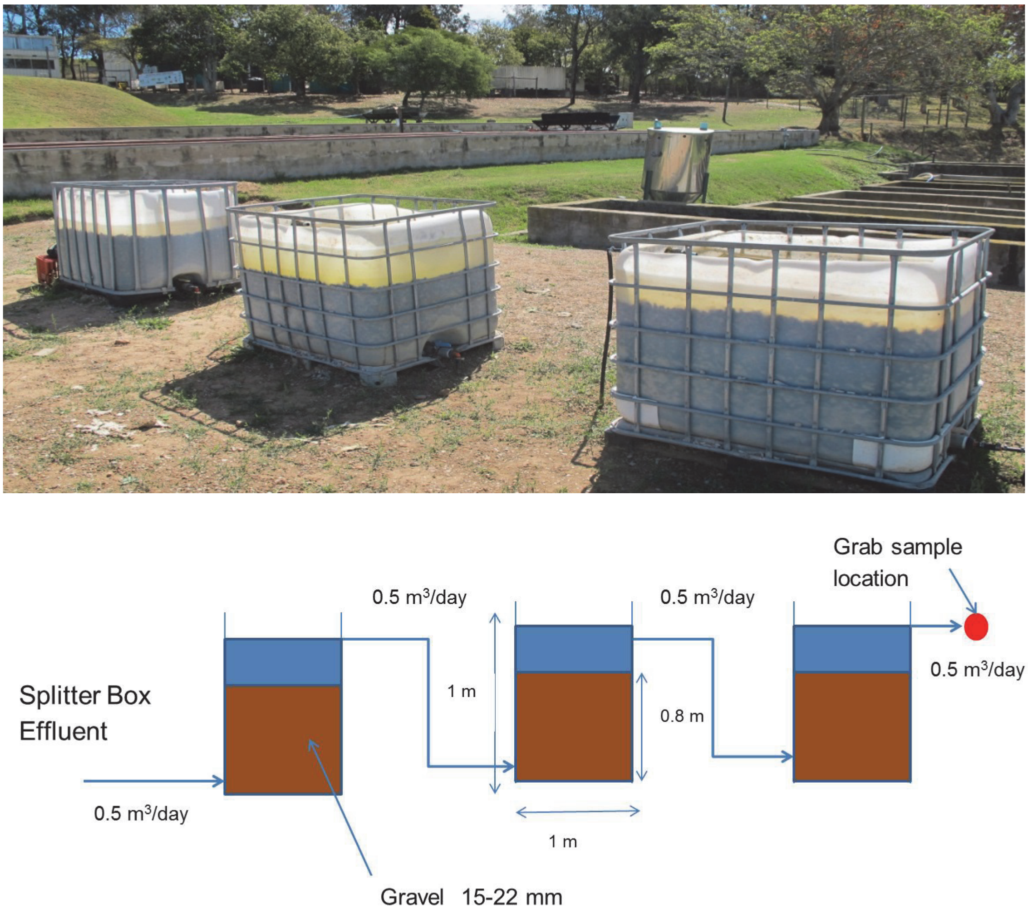


Figure C.4 Photograph and schematic diagram of the controlled rock filter series. Each container is 1 × 1 m with the gravel sand to a depth of 0.6 m and a water head of 0.3 m. The flow rate was 0.5 $k\ell \cdot d^{-1}$ and the rock particles ranged between 15-22 mm.

Hydraulic Loading Rate (HLR) is the most critical factor in the design and construction of controlled rock filters and it is important that the flow of the wastewater is below the rock surface to prevent algae growth and insect annoyance. It is suggested that in order to get good results with 1-2 cm rock, there needs to be a HLR of between 0.15-0.30 $k\ell \cdot m^3 \cdot d^{-1}$ (Middlebrooks et al., 2005). For the purposes of the present study, HLR was calculated using the expression;

$$A = Q / HLR$$

Where A = Area, Q = flow rate ($k\ell \cdot d^{-1}$) and HLR (hydraulic loading rate) (m/h)

$$1.0 = 0.5/ \text{HLR}$$
$$\text{HLR}= 0.5/ 1.0$$
$$\text{HLR}= \underline{\mathbf{0.5\ m.d^{-1}}}$$

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Appendix D: Life Cycle Assessment Data

INPUT DATA

EBRU IAPS

	values	units
Productivity	5.79	$g/m^2/d$
Area	1,000	m^2
Daily productivity	5.79	kg/d
Annual productivity	2.11	t/y

CO_2 fixed	1.88	g/g algae
CO_2 fixed/d	10.88	kg/d
CO_2 fixed/y	3.97	t/y
CO_2 fixed/ML	0.145	t/ML

Oxygen production

CO_2 MW	44	g/mol
O_2 MW	32	g/mol
O_2 produced	0.73	g O_2/g CO_2
O_2 produced	7.91	kg/d
O_2 produced	2.89	t/y
O_2 produced	0.11	t/ML

Commercial IAPS

	values	units
Productivity	5.79	$g/m^2/d$
Area	1,500	m^2
Daily productivity	8.68	kg/d
Annual productivity	3.17	t/y

CO_2 fixed	1.88	g/g algae
CO_2 fixed/d	16.32	kg/d
CO_2 fixed/y	5.96	t/y
CO_2 fixed/ML	0.218	t/ML

Oxygen production

CO_2 MW	44	g/mol
O_2 MW	32	g/mol
O_2 produced	0.73	g O_2/g CO_2
O_2 produced	11.87	kg/d
O_2 produced	4.33	t/y
O_2 produced	0.16	t/ML

Abbreviated table

CO_2 fixed/d	10.88	kg/d
CO_2 fixed/y	3.97	t/y
CO_2 fixed/ML	0.145	t/ML
O_2 produced	7.91	kg/d
O_2 produced	2.89	t/y
O_2 produced	0.11	t/ML

CO_2 fixed/d	16.32	kg/d
CO_2 fixed/y	5.96	t/y
CO_2 fixed/ML	0.218	t/ML
O_2 produced	11.87	kg/d
O_2 produced	4.33	t/y
O_2 produced	0.16	t/ML

SUMMARY EMISSIONS DATA

1

EBRU IAPS — Energy inputs

Activity	kWh/d	kWh/y	t CO_2/y	t CO_2/ML
Pumping	78.6	28,694	24.2	0.88
Paddles	14.9	5,431	4.6	0.17
Total	**93.5**	**34,126**	**28.8**	**1.05**

Commercial IAPS — Energy inputs

Activity	kWh/d	kWh/y	t CO_2/y	t CO_2/ML
Pumping	0.0	0.0	0.0	0.00
Paddles	18.0	6,570	5.5	0.19
Total	**18.0**	**6,570**	**5.5**	**0.19**

2

EBRU AD/PFP — GHG production by the AD

	t CO_2/y	t CO_2/ML
CO_2 yield	1.54	0.056
CH_4 yield	13.05	0.477
$CH_4 \rightarrow CO_2$ yield	1.71	0.062

Commercial AD/PFP — GHG production by the AD

	t CO_2/y	t CO_2/ML
CO_2 yield	1.54	0.056
CH_4 yield	13.05	0.477
$CH_4 \rightarrow CO_2$ yield	1.71	0.062

3

EBRU HRAPS — CO_2 sequestration and O_2 production

	t CO_2/y	t CO_2/ML
CO_2 fixed	3.97	0.145

	t O_2/y	t O_2/ML
O_2 produced	2.89	0.11

Commercial HRAPS — CO_2 sequestration and O_2 production

	t CO_2/y	t CO_2/ML
CO_2 fixed	5.96	0.218

	t O_2/y	t O_2/ML
O_2 produced	4.33	0.16

4

EBRU IAPS — Net effects

	t CO_2/y	t CO_2/ML
GHG production (abiotic)	28.77	1.05
CO_2 sequestration	3.97	0.15
GHG production (biotic)	14.59	0.53
Net GHG production	39.38	1.44
If biotic GHG reallocated	24.80	0.91
Methane energy used	20.52	0.76

Commercial IAPS — Net effects

	t CO_2/y	t CO_2/ML
GHG production (abiotic)	5.54	0.19
CO_2 sequestration	5.96	0.22
GHG production (biotic)	14.59	0.53
Net GHG production	14.17	0.50
If biotic GHG reallocated	-0.42	-0.03
Methane energy used	-4.70	-0.17

IAPS ENERGY AND CO_2 FOOTPRINT

EBRU IAPS

Loading rate	75	m^3/d	Treatment	
AD/PFP treatment	75	m^3/d	100%	vol
HRAP 1 treatment	37.5	m^3/d	50%	vol
HRAP 2 treatment	37.5	m^3/d	100%	vol

0.000843 t CO_2/kWh

Energy inputs

Flows								
Site	Method	Duration (h/d)	kWh	kWh/d	kWh/y	kWh/ML	t CO_2/y	t CO_2/ML
Head of works								
AD/PFP	pump	24	0.45	10.8	3942	144.0	3.32	0.121
HRAP 1	gravity	24	0	0	0	0	0.00	0.000
Settler 1	gravity	24	0	0	0	0	0.00	0.000
Splitter box	pump	12	0.25	3	1,095	80.0	0.92	0.034
Drying bed 1a	pump	0.015	0.25	0.0037	1.36	0.050	0.00	0.000
Drying bed 1b	pump	0.015	0.25	0.0037	1.36	0.050	0.00	0.000
HRAP 2	pump	24	0.5	12	4,380	160.0	3.69	0.135
Settler 2	gravity	24	0	0	0	0	0.00	0.000
Drying bed 2a	pump	0.015	0.25	0.0037	1.36	0.050	0.00	0.000
Drying bed 2b	pump	0.015	0.25	0.0037	1.36	0.050	0.00	0.000
Splitter box return	pump	24	2.2	52.8	19,272	704.0	16.25	0.593
Mixing (paddle wheels)								
HRAP 1	drive	24	0.25	6.0	2,190	80.0	1.85	0.067
HRAP 2	drive	24	0.37	8.88	3,241	118.4	2.73	0.100
Total				**93.5**	**34,126**	**1,247**	**28.77**	**1.051**

Commercial IAPS

Loading rate	80	m^3/d	Treatment	
AD/PFP treatment	80	m^3/d	100%	vol
HRAP 1 treatment	80	m^3/d	100%	vol
HRAP 2 treatment	80	m^3/d	100%	vol

Flows								
Site	Means	Duration (h)	kWh	kWh/d	kWh/y	kWh/ML	t CO_2/y	t CO_2/ML
Head of works								
AD/PFP	gravity	24	0	0.0	0	0.0	0.00	0.000
HRAP 1	gravity	24	0	0.0	0	0.0	0.00	0.000
HRAP 2	gravity	24	0	0.0	0	0.0	0.00	0.000
Settler 1 & 2	gravity	24	0	0.0	0	0.0	0.00	0.000
Drying bed 1a	gravity	0.015	0	0.0	0	0.0	0.00	0.000
Drying bed 1b	gravity	0.015	0	0.0	0	0.0	0.00	0.000
Drying bed 2a	gravity	0.015	0	0.0	0	0.0	0.00	0.000
Drying bed 2b	gravity	0.015	0	0.0	0	0.0	0.00	0.000
Mixing (paddle wheels)								
HRAP 1	drive	24	0.25	6.0	2190	80.0	1.85	0.063
HRAP 2	drive	24	0.25	6.0	2190	80.0	1.85	0.063
HRAP 3	drive	24	0.25	6.0	2190	80.0	1.85	0.063
					0			0.000
Total				**18.0**	**6570**	**240**	**5.54**	**0.190**

HRAOP ALGAE PRODUCTION

EBRU IAPS

	values	units
Productivity	5.79	$g/m^2/d$
Area	1,000	m^2
Daily productivity	5.79	kg/d
Annual productivity	2.11	t/y

CO_2 fixed	1.88	g/g algae
CO_2 fixed/d	10.88	kg/d
CO_2 fixed/y	3.97	t/y
CO_2 fixed/ML	0.145	t/ML

Oxygen production

CO_2 MW	44	g/mol
O_2 MW	32	g/mol
O_2 produced	0.73	g O_2/g CO_2
O_2 produced	7.91	kg/d
O_2 produced	2.89	t/y
O_2 produced	0.11	t/ML

Commercial IAPS

	values	units
Productivity	5.79	$g/m^2/d$
Area	1,500	m^2
Daily productivity	8.68	kg/d
Annual productivity	3.17	t/y

CO_2 fixed	1.88	g/g algae
CO_2 fixed/d	16.32	kg/d
CO_2 fixed/y	5.96	t/y
CO_2 fixed/ML	0.218	t/ML

Oxygen production

CO_2 MW	44	g/mol
O_2 MW	32	g/mol
O_2 produced	0.73	g O_2/g CO_2
O_2 produced	11.87	kg/d
O_2 produced	4.33	t/y
O_2 produced	0.16	t/ML

Abbreviated table

CO_2 fixed/d	10.88	kg/d
CO_2 fixed/y	3.97	t/y
CO_2 fixed/ML	0.145	t/ML
O_2 produced	7.91	kg/d
O_2 produced	2.89	t/y
O_2 produced	0.11	t/ML

CO_2 fixed/d	16.32	kg/d
CO_2 fixed/y	5.96	t/y
CO_2 fixed/ML	0.218	t/ML
O_2 produced	11.87	kg/d
O_2 produced	4.33	t/y
O_2 produced	0.16	t/ML

ANAEROBIC DIGESTION IN IN-POND DIGESTER

The commercial IAPS and the Belmont Valley WWTW IAPS are assumed to have the same specifications and performances.

CH_4 and CO_2 production calculations

Sewage MW	($C_{18}H_{19}O_9N$)

a (C)	*b* (H)	*c* (O)	*d* (N)	H_2O	CH_4	CO_2	NH_3	%CH_4	COD/VS
18	19	9	1	9.50	8.75	9.25	1	46%	1.541

L CH_4 yield (g VS)	L CO_2 yield (g VS)	L CH_4 yield (g COD)	L CO_2 yield (g COD)
0.499	0.527	0.350	0.340

B_0 (L per g VS)	$Y_{N\text{-}NH3}$ (mg g VS^{-1})	Biogas CH_4	Biogas CO_2
2.86	43.3	49%	51%

CH_4 and CO_2 yield calculations

Effluent concentration	0.1	g/L	Annual yield		CO$_2$ equiv	
Ash content	30%					
Flow rate	80	m^3/d				
Sewage inventory	8	kg/d				
VS inventory	5.6	kg/d				
Conversion efficiency	85%					
CO_2 density	1.977	kg/m^3				
CH_4 density	0.717	kg/m^3				
CO_2 yield (L)	2.510	m^3/d				
CO_2 yield (kg)	4.22	kg/d	1.539	t/y	1.539	t CO_2/y
CH_4 yield (L)	2.374	m^3/d				
CH_4 yield (kg)	1.70	kg/d	0.621	t/y	13.047	t CO_2/y

Energy recovery from CH_4

	CH_4 LHV		Annual yield		Thermal energy		Electrical energy	
CH_4 as renewable energy	50.1	MJ/kg	18.3	GJ/y	5.080	MW/y	2.032	MWe/y

CO_2 footprint of IAPS AD

	Value	Units
Estimated CO_2 production	1.539	t CO_2/y
Estimated CH_4 production	13.047	t CO_2/y
Estimated CO_2 produced if CH_4 burnt	1.709	t CO_2/y
Estimated CO_2 reduction if electricity recovered	4.282	t CO_2/y
Net CO_2 emissions from AD	-1.035	t CO_2/y

Estimated GHG reduction if methane used to generate electricity

	Thermal energy		Electrical energy	
Estimated energy recovery	0.174	MW/ML	0.0696	MWe/ML
Estimated fossil fuel CO_2 reduction	4.282	t CO_2/y	0.1467	t CO_2/ML
Estimated CO_2 produced if CH_4 burnt	1.709	t CO_2/y	0.0585	t CO_2/ML
Estimated GHG reduction if CH_4 burnt	11.338	t CO_2/y	3.8830	t CO_2/ML

Effects of different operating scenarios on GHG emissions

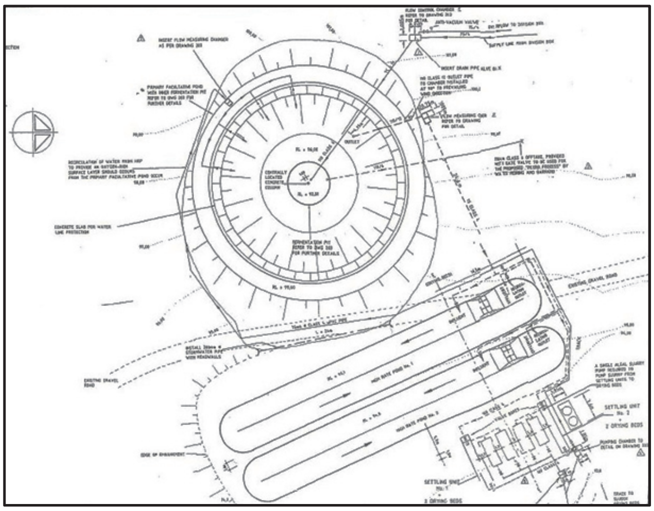
Options	Value	Units
Scenario 1 - CH_4 is emitted	14.586	t CO_2/y
Scenario 2 - CH_4 combusted	3.248	t CO_2/y
Scenario 3 - CH_4 combusted and energy generated	-1.035	t CO_2/y

Appendix E: Operator and Maintenance Guide

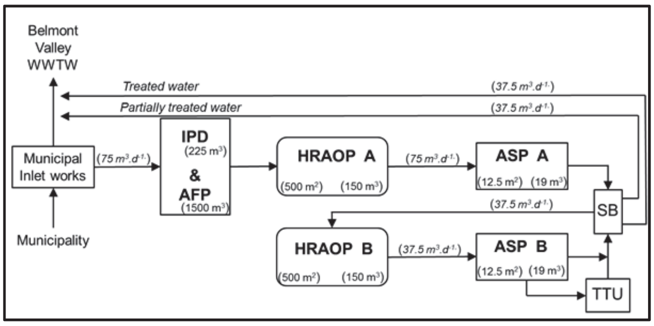
System and Process Flow

Aerial view

Surface plan



Process flow



LEGEND

AFP=Advanced Facultative Pond;
AD=Anaerobic Digester;
HRAOP=High Rate Algae Oxidation Pond;
ASP=Advanced Settling Pond;
SB=Splitter Box; C/F=Coagulation/Flocculation;
MP=Maturation Pond;
MMF=Multimedia Filtration;
SSF=Slow Sand Filter;
UV=Ultraviolet.

Standard Operating Procedure

Daily operation and maintenance is carried out by following the steps below:

Task	Action	Check
1. **Inlet** (flow of raw sewage):		
a) Remove inlet cover	**- Remove Debris and Clean -**	
b) Check flow in splitter box	**- Replace Cover and Measure Flow -**	
2. **Digester**:		
a) Check inlet pipe for obstructions	**- Clear and Clean -**	
b) Check digester activity	**- Observe Gas Bubbles in Water -**	
3. **Facultative Pond** (off-take pipe):		
a) If below water surface	**- Stop Flow and Remove -**	
b) Remove the pipe from its shaft	**- Clean -**	
c) Replace pipe and restart flow	**- Replace and Restart -**	
d) Check the flow	**- Measure Flow -**	
e) Check off-take pipe above position	**- Adjust Off-Take Pipe -**	
4. **Raceway** inflow:		
a) Remove splitter box cover	**- Remove Debris and Clean -**	
b) Check flow into Raceways	**- Replace Cover and Measure Flow -**	
5. **Paddlewheels**:		
a) Check paddlewheel shafts	**- Check Position -**	
b) Check gearbox	**- Check Rotation and Speed -**	
c) Clear obstructions	**- Remove stones from raceway floor -**	
d) Check raceway colour is green	**- Observe water colour and turbidity -**	
6. **Algae Settling Ponds**:		
a) Determine amount of settled algae	**- Measure -**	
b) Divert outflow to another settler	**- Change Flow -**	
c) Transfer biomass to drying bed	**- Empty Algae Settler -**	
c) Clean emptied settler	**- Clean -**	
d) Estimate next cleaning cycle	**- Date of Next Clean -**	

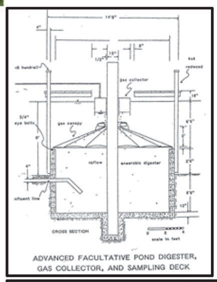
For detail of unit operations see "*Integrated Algae Ponding System: Technical Description*" by AK Cowan and DS Render (attached)

Component Parts

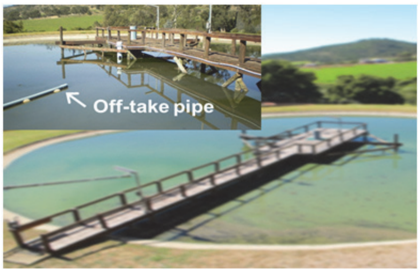
Inlet

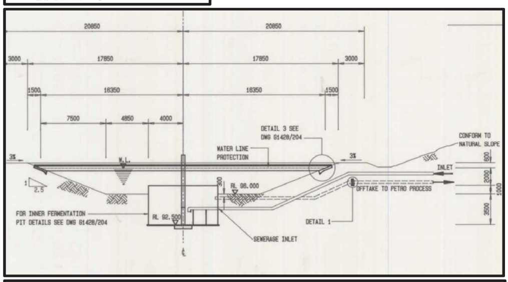
Note: In commercial systems a standard inlet works is in operation

Digester

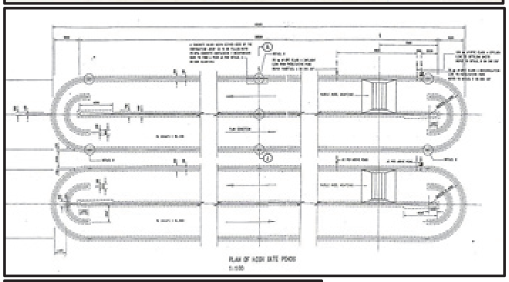


Facultative Pond

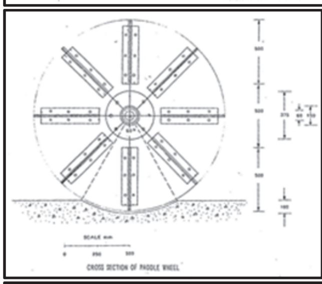




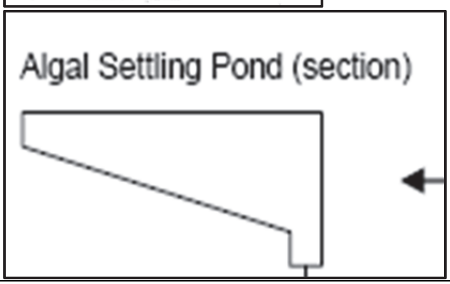
Raceways



Paddlewheel



Settling Pond



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