SELF-REGULATION OF THE PACKAGE PLANTS/SWWTW INDUSTRY

Volume 1: Development of Proposed Framework of Standards, a Conceptual Model for a Test Facility and an Accreditation System for Each "New" Technology Provided by Suppliers

PN Gaydon

WATER RESEARCH COMMISSION

TT 620/14

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VOLUME 1

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PN Gaydon

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Water Research Commission

by

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Water Research Commission
Private Bag X03
Gezina, 0031
Republic of South Africa

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

The publication of this report emanates from a project entitled *Self-Regulation of the Package Plants/SWWTW Industry: Development of Framework of Standards, a Conceptual Model for a Test Facility and an Accreditation System for Each Technology Provided by Suppliers* (WRC Report No. K5/2193)

This report forms part of a series of two reports. The other report is *Self Regulation of the Package Plants/SWWTW Industry. Development of a "Green Droplet" Accreditation System* (WRC Report No. TT 621/14)

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EXECUTIVE SUMMARY

Small wastewater treatment (inclusive of package plants) are a common form of service utility in sewage treatment for smaller communities and are needed where sewerage reticulation is absent due inadequate space, difficult terrain, remoteness of areas in need and where standards set are higher than the effluent quality obtained from simple septic tank systems. The SWWTW industry in South Africa has grown rapidly from a small base and is currently unregulated in terms of process design, construction materials, etc. Most of the suppliers are not process experts but rather entrepreneurs who have funded the development of their product using limited resources. In this industry, traditionally maintenance contracts were not required by purchasers. Furthermore, some property developers provided the SWWTW suppliers with incorrect flow and strength data on which to base the design, or added extra housing units onto an existing plant without expanding it and Body Corporates also tended to neglect the operation and maintenance of the plants.

The aims of this project were as follows:

- Develop a framework of standards for small wastewater treatment technologies, which is practical for South Africa.
- Assess and recommend how the framework of standards will work within the sector ensuring that duplication is eliminated in the process.
- Develop a conceptual model with key criteria for an independent testing facility of the different technologies, including the evaluation of the feasibility of using existing in South Africa.
- Develop an accreditation system for technologies by the various suppliers which will encompass technical and managerial aspects, including a modification of the Green Droplet System that is currently used by DWA. This should also include the institutional and cost issues pertaining to implementation.

This study resulted in the compilation of 2 reports: SELF-REGULATION OF THE PACKAGE PLANTS/SWWTW INDUSTRY:

VOLUME 1 – Development of Proposed Framework of Standards, a Conceptual Model for a Test Facility and an Accreditation System for Each "New" Technology Provided by Suppliers

VOLUME 2 – Development of a "Green Droplet" Accreditation System

Volume 1 provides an introduction to the framework of standards which could be adapted for use in South Africa, and discusses their strengths and weaknesses, together with the feasibility of scaling them up for use on larger Works. It draws from current industry know-how as well as Australian, European and the United States NSF standards used internationally.

It examines the current South African legislative standards for discharge of treated effluent to the environment, together with the corresponding monitoring requirements. It continues to further examine the current General Authorisation Discharge Requirements, and makes strong recommendations with respect to:

•The quality of water to be used for lawn irrigation

•The issue of satisfactory compliance which needs to properly defined, including the method of calculation and the percentage compliance.

A categorization framework for SWWTW sizes was discussed and a three-tier system recommended after consultation with the industry body SEWPACKSA and the WISA SWWTW Division. Furthermore, a proposed SWWTW Treatment Efficiency Testing Standard was formulated inclusive of proposed process design standards. The intention behind this proposed SWWTW Treatment Efficiency Testing Standard is that it would serve as a national standard and would obviate the need for various municipalities to publish their own individual standards or by-laws. The study did a brief evaluation of the concept of a SWWTW evaluation facility making recommendations with respect to the requirements, funding of the facility and its operation.

Volume 2 of this study examines the development of the Green Droplet System for Small Wastewater Treatment Works (SWWTW). The Green Droplet System for SWWTW was borne out of a number of stakeholders needs such as:

Stakeholder	Needs
General Public	Desire for clean, unpolluted, aquatic environment
Regulators	Desire for a self-regulated system in place of a command and control system
SWWTW Owners	Desire for compliance with legislative requirements
SWWTW Suppliers	Desire for compliance with legislative requirements for SWWTW supplied

The behavioural change espoused in this concept of self-regulation is that the various stakeholders (owner, designer, supplier, operator, and regulator) would see fit to take the right actions proactively to minimize risk to environment, health and reputation. Thus, the system proposed is a simplified and graded system applicable to different Categories of SWWTW.

ACKNOWLEDGEMENTS

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Ms V Naidoo	Water Research Commission	(Chairperson)
Mr G Brown	Tecroveer Group	
Ms P Dama-Fakir	Golder and Associates	
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Mr M Ross	Lilliput Treatment Technologies International	
Mr David Light	Biobox South Africa	

Aspects of the document were completed in consultation with SEWPACKSA, the small wastewater treatment supplier association as well as the sector during opportune events like the WISA biennial conference.

TABLE OF CONTENTS **PAGE NO**

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION AND LITERATURE REVIEW

1.1 Background Information

This report is a sequel to two other reports commissioned by the Water Research Commission:

- Evaluation of Sewage Treatment Package Plants for Rural, Peri-Urban and Community Use, Gaydon et al., 2006.

- Guideline Document: Package Plants for the Treatment of Domestic Wastewater, van Niekerk et al., 2009.

The aims of this project were defined as follows:

- Develop a framework of standards for small wastewater treatment technologies (including package plant) as defined in the “DWA Package Plant Guidelines” and by the SWWTW Division. The standards should consider the EU CEN standardization (e.g. CEN std 12566-3:2005) and US NSF/ANSI 40 approaches. It should be practical for South Africa.

- Assess and recommend how the framework of standards will work within the sector ensuring that duplication is eliminated in the process.

- Develop a conceptual model with key criteria for an independent testing facility of the different technologies. Note: the conceptual model must evaluate the funding model, test facility standardization model, skills model.

- Evaluate the feasibility of using existing facilities in selected locations in South Africa.

- Develop an accreditation system for technologies by the various suppliers which will encompass technical and managerial aspects. This could be a modification of the green drop system that is currently used by DWA but which takes into consideration the fragmented roles of the sector stakeholders. The study should also look to who will manage and audit the accreditation process, the cost of the process and who bears the cost.

This report is split into two volumes with the first four aims being reported on in Volume 1 while the last aim is reported on in Volume 2.

1.2 Introduction

For the purposes of this report the definition of a Small Wastewater Treatment Works (SWWTW) is limited to those works designed to treat a daily Average Dry Weather Flow (ADWF) of less than 2000 m^3/d.

The SWWTW industry in South Africa has grown rapidly from a small base and is currently unregulated in terms of process design, construction materials, etc. Most of the suppliers are not process experts but rather entrepreneurs who have funded the development of their product using limited resources.

Furthermore, SWWTW's are often purchased on the basis of purchase cost which means that in some cases product costs have to be minimized. This can be achieved by omitting redundancy in the plant such as aerators and pumps, overloading media, and using optimistic upflow rates in settlers. Traditionally maintenance contracts were not required by purchasers.

An aggravating factor has been that many in the past some property developers provided the SWWTW suppliers with incorrect flow and strength data on which to base the design, or added extra housing units onto an existing plant without expanding it. Body Corporates also tended to neglect the operation and maintenance of the plants (Gaydon et. al., 2007).

SWWTW are a common form of service utility in sewage treatment for smaller communities. SWWTW are also needed where regional sewerage reticulation is absent and inadequate space for a common septic tank system (especially in multiple housing units) (Laas and Botha, 2004).

SWWTW are often viewed as a cheaper operating option in the long term than conservancy tanks (Laas & Botha, 2004). It has been observed that since the 1980s, SWWTW have made significant advances in addressing the sewage treatment needs of small communities.

Prefabricated systems are have been found to be less costly and quicker to construct than individually designed systems, and are thus often selected despite their occasional lack of flexibility in meeting site specific needs (Bucksteeg. 1990). It has been noted that in new building projects, developers tend to utilise the maximum possible area for housing to best realise profits. This often does not allow sufficient space for common septic tank systems (especially in multiple housing units), so where regional sewerage reticulation is absent the only alternative is to make use of SWWTW employing more sophisticated technologies (Laas & Botha, 2004).

As previously observed, the reduced capital costs and space efficiencies are factors favouring the employment of SWWTW. This positive aspect is further advanced when the units make use of commercially available components, so allowing for both construction and maintenance to be extremely quick and cost effective (Hulsman &. Svsartz, 1993).

SWWTW lend themselves to standardisation of motors, pumps, timers, etc. across a range of plant sizes, which can greatly assist in reducing maintenance complexity and cost (Stoodley. 1989). Supplementary advantages include ease of transport for units that can be housed in containers, the potential for reduced noise and odours and year round heat conservation for colder climates (Hulsman & Swartz, 1993).

For a more detailed account on the background to the use of SWWTW and the accompanying challenges – see Gaydon et al., 2007).

1.3 Introduction to Framework of Standards

It has previously been suggested that a set of standards be drawn up to regulate the design and construction of SWWTW to ensure that they are capable of producing a compliant effluent under the difficult conditions these works operate (marked flow peaks, low flows which are difficult to pump with the available pumps without expensive controls or modification, and toxicity due to over-zealous use of disinfectants).

Typical examples of such standards are the European EU-CEN 12566-3:2005 standard, the American NSF/ANSI 40 standard and the AUS/NZ standards

1.3.1 European Standard EU-CEN 12566-3:2005

European Standard EU-CEN 12566-3:2005 – Small Wastewater Treatment Systems for up to 50 PT – Part 3: Packaged and/or site assembled domestic wastewater treatment plants.

Part 1 and Part 2 relate to septic tanks and soil infiltration systems. Part 3 relates to "Packaged and/or site assembled domestic wastewater treatment plants" and specifies the "requirements and test methods used to evaluate packaged wastewater treatment plants which are required to treat sewage to a pre-determined standard."

50 PT means 50 inhabitants or population equivalents, which can be estimated to be approximately 25 m^3/d.

The Standard is available on the weblink: http://www.ecowa.ro/download/cen12566.pdf

1.3.2 EPA NSF/ANSI 40-2005 and NSF/ANSI 245-2007

EPA NSF/ANSI 40-2005 and NSF/ANSI 245-2007 National Sanitation Forum certification system for Residential Wastewater Treatment Systems (including nitrogen reduction).

The standard contains minimum requirements for residential wastewater treatment systems having rated treatment capacities between 1.5 m^3/d and 5.7 m^3/d.

The Standard is available on the weblink: http://www.techstreet.com/nsf/?

1.3.3 Australian/New Zealand Standard AS/NZS 1547:2000

Australian/New Zealand Standard AS/NZS 1547:2000 – On-site domestic wastewater management

The objective of this standard is to provide for the requirements for primary and secondary-treatments for all persons and agencies involved with sustainable and effective on-site domestic wastewater management.

The standard is limited to systems treating a maximum of 2 m^3/d, a population equivalent of 10 persons.

1.3.4 Design Guidelines for Small Waste Water Treatment Works

During the period that this report was being finalised, the National Department of Public Works published a document entitled "Design Guidelines for Small Waste Water Treatment Works" which, although not a Standard, is an extremely useful resource with respect to these Works.

The document is available on the weblink:

http://www.publicworks.gov.za/PDFs/consultants_docs/Design_guidline_for_Small_Waste_Water_Treatment_Works.pdf

1.3.5 Applicability of Standards to Larger Capacity SWWTW's

The maximum treatment capacity limitations provided in the three Standards mentioned above range between 2 and 25 m^3/d, which is very low. The reason for this is not given in the respective standards which is unfortunate and poses the question as to whether they are applicable to larger SWWTW's.

In the author's experience it appears to be reasonable that the test can be extended to larger maximum capacities, since it is generally accepted that large capacity plants generally produce better quality effluents than there smaller counterparts, and are simpler to design in terms of pumping, hydraulics and clarification.

This will however require larger testing facilities and equipment, which may have been the reason for the limitation by the above authorities.

1.4 Current South African Legislative Standards for Discharge of Water to the Environment

1.4.1 Discussion of Current Regulations

There are currently two sections in the General Authorisation (GA – GN 1191 of 1999 in terms of the National Water Act (Act No. 36 of 1998) which govern the quality of the effluent produced by SWWTW. They are the:

- Discharge Standard – applies to the discharge of effluent directly to a watercourse
- Irrigation Standard – applies to the irrigation of water on crops and pastures.

These are discussed below.

1.4.2 Discharge Standard

The discharge standard which is currently applicable to the discharge of SWWTW effluents (2000 m^3/d) directly into water bodies is shown in Table 2.1 below.

Table 1-1: General Authorisation Standards

Substance/Parameter	General Limit	Special Limit
Faecal Coliforms (per 100 mℓ)	1 000	0
Chemical Oxygen Demand (mg/ℓ)	75*	30*
pH	5.5-9.5	5.5-7.5
Ammonia (ionised and un-ionised) as Nitrogen(mg/ℓ)	6	2
Nitrate/Nitrite as Nitrogen (mg/ℓ)	15	1.5
Chlorine as Free Chlorine (mg/ℓ)	0.25	0
Suspended Solids (mg/ℓ)	25	10
Electrical Conductivity (mS/m)	70 mS/m above intake; max 150 mS/m	50 mS/m above receiving water, max of 100 mS/m
Ortho-Phosphate as phosphorous (mg/ℓ)	10	1 (median) and 2.5 (maximum)
Fluoride (mg/ℓ)	1	1
Soap, oil or grease (mg/ℓ)	2.5	0
Dissolved Arsenic (mg/ℓ)	0.02	0.01
Dissolved Cadmium (mg/ℓ)	0.005	0.001
Dissolved Chromium (VI) (mg/ℓ)	0.05	0.02
Dissolved Copper (mg/ℓ)	0.01	0.002
Dissolved Cyanide (mg/ℓ)	0.02	0.01
Dissolved Iron (mg/ℓ)	0.3	0.3
Dissolved Lead (mg/ℓ)	0.01	0.006
Dissolved Manganese (mg/ℓ)	0.1	0.1
Mercury and its compounds (mg/ℓ)	0.005	0.001
Dissolved Selenium (mg/ℓ)	0.02	0.02
Dissolved Zinc (mg/ℓ)	0.1	0.04
Boron (mg/ℓ)	1	0.5

* After removal of algae

1.4.3 Irrigation Standard

In terms of the General Authorisation a user can irrigate up to 500 cubic metres of domestic or biodegradable industrial wastewater on any given day, provided the-

(a) Electrical conductivity does not exceed 200 milliSiemens per metre (mS/m);

(b) pH is not less than 6 or more than 9 pH units;

(c) Chemical Oxygen Demand (COD) does not exceed 400 mg O_2/ℓ after removal of algae;

(d) faecal coliforms do not exceed 100 000 counts per 100 mℓ; and

(e) Sodium Adsorption Ratio (SAR) does not exceed 5 for biodegradable industrial wastewater.

1.4.4 Monitoring Requirements

The monitoring requirements for SWWTW are also set out in terms of the General Authorisation, and are shown in Table 2.2.

Table 1-2: Monitoring requirements for domestic wastewater discharges

Daily Discharge Volume	Monitoring Requirements
< 10 cubic metres	None
10 to 100 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ)
100 to 1000 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ) Chemical Oxygen Demand (mg/ℓ) Ammonia as Nitrogen (mg/ℓ) Suspended Solids (mg/ℓ)
1 000 to 2 000 cubic metres	pH Electrical Conductivity (mS/m) Faecal Coliforms (per 100 mℓ) Chemical Oxygen Demand (mg/ℓ) Ammonia as Nitrogen (mg/ℓ) Nitrate/Nitrite as Nitrogen (mg/ℓ) Free Chlorine (mg/ℓ) Suspended Solids (mg/ℓ) Ortho-Phosphate as Phosphorous (mg/ℓ)

The frequency of monitoring is also set out in the General Authorisation and depends upon the nature of the wastewater. For domestic sewage it is monthly, with the exception of plants with a treatment capacity of less than 10 cubic metres per day, where no monitoring is required.

1.5 Discussion of General Authorisation Requirements

1.5.1 General Authorisation Discharge Standard

The Project Team believes that the General Limit Values of the General Authorisation Discharge Standard should be able to be met by properly designed SWWTW on a regular basis, but that the Special Limit will require a purpose designed plant.

It is thus recommended that the Discharge Standard General Limit Values be retained as it is (with the ammonia amendment to 6 mg N/ℓ).

1.5.2 Irrigation Standard

The project team strongly recommends that wherever possible the treated effluent from SWWTW should be irrigated. The reasons for this are as follows:

- Water is a scarce commodity in South Africa, and its reuse will reduce demand for potable water for gardening purposes.
- This reduction in demand will assist strongly in municipal demand management schemes and should result in reduced capital infrastructure.
- The effluent being discharged from the SWWTW is seldom being returned to a point close to its original abstraction, and this has the potential to significantly change the stream flows if it is directly discharged into a local watercourse. This would be particularly significant in the case of housing developments.
- The nutrients contained in the effluent are also a valuable resource. It seems pointless to waste this, instead supplementing with commercial fertilizers which often contain heavy metals, and whose production results in environmental degradation.

Although this quality is suitable for agricultural use, and is currently limited to thereto (there is currently a move to include irrigation of domestic gardens in this Standard), it is doubtful that that the high COD is suitable for irrigation in urban and peri-urban areas where the unstabilised organics may cause odour and fly nuisances. The high faecal coliform concentration is also a concern as this may result in the infection of lacerations when children, or adults, play on the grass. Obviously some disinfection will take place due to UV radiation, but it is our experience that there is still a health threat.

It is thus recommended that for the irrigation of lawns the following standard be applied:

- Maximum COD of 75 mg/ℓ to prevent nuisances
- Maximum *E. coli* count of 1000 counts/100 mℓ where the lawn is for general use
- No disinfection required for lawns of restricted use

As lawns have a high nitrogen uptake rate it is recommended that there should be no nitrogen concentration restriction. It should be noted however that it may be difficult to achieve disinfection (as in the case of general use) where ammonia concentrations are high due to the formation of chloramines and the resulting slower disinfection rate. This may be able to be compensated for by using a higher chlorine dose and chlorine contact time.

It is the opinion of the Project Team that the irrigation of high ammonia effluent will not cause a nuisance as the smell is rapidly diluted by the atmosphere. This is borne out by experience at wastewater works, where the main smells tend to be hydrogen sulphide and volatile fatty acids (in the case of irrigation of stored waste activated sludge mixed with anaerobically digested sludge). No complaints have been received regarding the high ammonia concentrations.

1.5.3 Monitoring requirements

It is recommended that the monitoring requirements set out in the General Authorisation be retained as they are deemed to be reasonable.

However, accurate measurement for small flows is difficult, and the equipment required is costly and thus for the purposes of recordkeeping it is suggested that the sewage volume be taken as 70% of the water consumed in the system discharging to the SWWTW.

1.5.4 Compliance

One of the main problems with the General Authorisation is the issue of compliance which is singularly neglected in both the National Water Act (Act No. 36 of 1998) and the General Authorisation (GN 1191 of 1999).

Neither the calculation of compliance, not the level of acceptable compliance is included in the definitions of either of the two legislative articles above. No figures are provided for the percentage compliance which is deemed to be satisfactory.

Wastewater treatment works receive highly variable quality influent and as such can produce a varying effluent quality. The smaller the works, the greater the influent quality variation, and the greater the variance in the effluent quality. Coupled to this is that compliance samples are still taken by means of grab sampling which can be largely unrepresentative of the effluent quality over a longer period.

1.6 Summary of Recommendations with Respect to General Authorisation Requirements

The following recommendations are made:

- It is recommended that the Discharge Standard General Limit Values be retained as it is (with the ammonia amendment to 6 mg/ℓ).
- It is thus recommended that for the irrigation of lawns the following standard be applied:
  - Maximum COD of 75 mg/ℓ to prevent nuisances
  - Maximum *E. coli* count of 1000 counts/100 mℓ where the lawn is for general use
  - No disinfection required for lawns of restricted use
- It is recommended that the monitoring requirements set out in the General Authorisation be retained as they are deemed to be reasonable. However it is suggested that the sewage volume be taken as 70% of the water consumed in the system discharging to the SWWTW.
- The subject of determining a satisfactory level of compliance is too intricate to form part of this study, and should be dealt with in detail in a separate dedicated WRC project.

1.7 Proposed Materials and Construction Standards

An examination of the SANS standards applicable to SWWTW in South Africa resulted in the formulation of Table 2.3 below. It is suggested that these standards be adopted for the purposes of this Framework. The majority of these will in all likelihood already be in used by manufacturers.

Table 1-3: Applicable SANS construction standards

Standard No.	Description
SANS 10400	Plumbing and Drainage Code
SANS 10142	Electrical
SANS 3001	Soil testing
SANS 10100	Concrete – slabs
SANS 10100	Concrete – structural
SANS 966	PVC pipes
SANS 10112	PE pipes
SANS 310	PE Tanks
SANS 53121	GRP tanks
SANS 10162/1	Steel Fabrication
SANS 12944/4 paints/varnishes SANS 121 hot dip galv	Steel Coating
SANS 9001	Pumps
SANS 1186	Safety signs

1.8 References

Bucksteeg, K. 1990. Suitability of different biological sewage treatments. *Wat. Sci. Tech*. Vol. 22. No.3/4, 187-194.

Gaydon P; McNab N; Mulder G; Pillay I; Sahibdeen M; Thompson P; 2007. Evaluation of sewage treatment package plants for rural, peri-urban and community use; Research Report No.1539/1/06, Water Research Commission, Pretoria, South Africa.

Hulsman, A. and Swartz, C.D. 1993. Development of an improved compact package plant for small community wastewater treatment. *Wat. Sci. Tech*. Vol. 28. No. 10, 283-288.

Laas, I., and Botha, C. 2004. Sewage package plants: a viability or a liability for new developments? *Proceedings of the 2004 Water Institute of Southern Africa (WISA) Biennial Conference*, 1486-1494.

Stoodley, A.E. 1989. Operational experiences with package filter sewage-treatment plants. *J. IWEM*. 3. December, 583-587.

2 SWWTW EVALUATION FACILITY

In order to establish a successful SWWTW Evaluation Facility the following requirements need to be met:

2.1 Physical amenities

- Engineered concrete slab on which to place SWWTW including septic tank.
- Pumping apparatus including control to dose sewage according to the specified regime.

2.2 Sewage supply

- A sewage supply which fits the required specification, i.e. COD of 300-1000 mg/ℓ and NH_3 20-80 mg/ℓ.
- Sewage should not have more than 10% industrial contribution, and no toxic shocks should have been recorded.
- Sewage should not suffer from extreme, protracted dilution due to rainfall events.

2.3 Manpower

Manpower will be required to:

- Take samples as per specification
- Ensure that pumping is according to specification
- Clear any screens that may be installed
- Monitor the plant operation for problems and breakdowns

2.4 Altitude and Climate

Waste water treatment works require greater aeration at high altitudes and the kinetics of bacteria decrease as the sewage temperature decreases until 10°C whereupon they are generally accepted to cease altogether. Fortunately South Africa does not have any extreme altitudes or climatic conditions, and thus SWWTW should be able to operate satisfactorily at any altitude and under any normal climatic conditions. The time of the year and hence the seasonal variation for a works to be evaluated will depend upon the time slot available for testing, and will not be able to be booked.

As such it would be prudent to situate the testing site at a reasonable altitude where the winters would be cooler to ensure that they will work at altitudes and climates other than the coast.

2.5 Ideal Site

An ideal site would provide the following:

- An altitude of >1500 m, with a warm summer and cold winter
- A waste water treatment works with a reliable supply of raw sewage which is not unduly diluted during rainfall events.
- A stable electricity supply
- Sufficient human resources available to conduct sampling and monitoring.

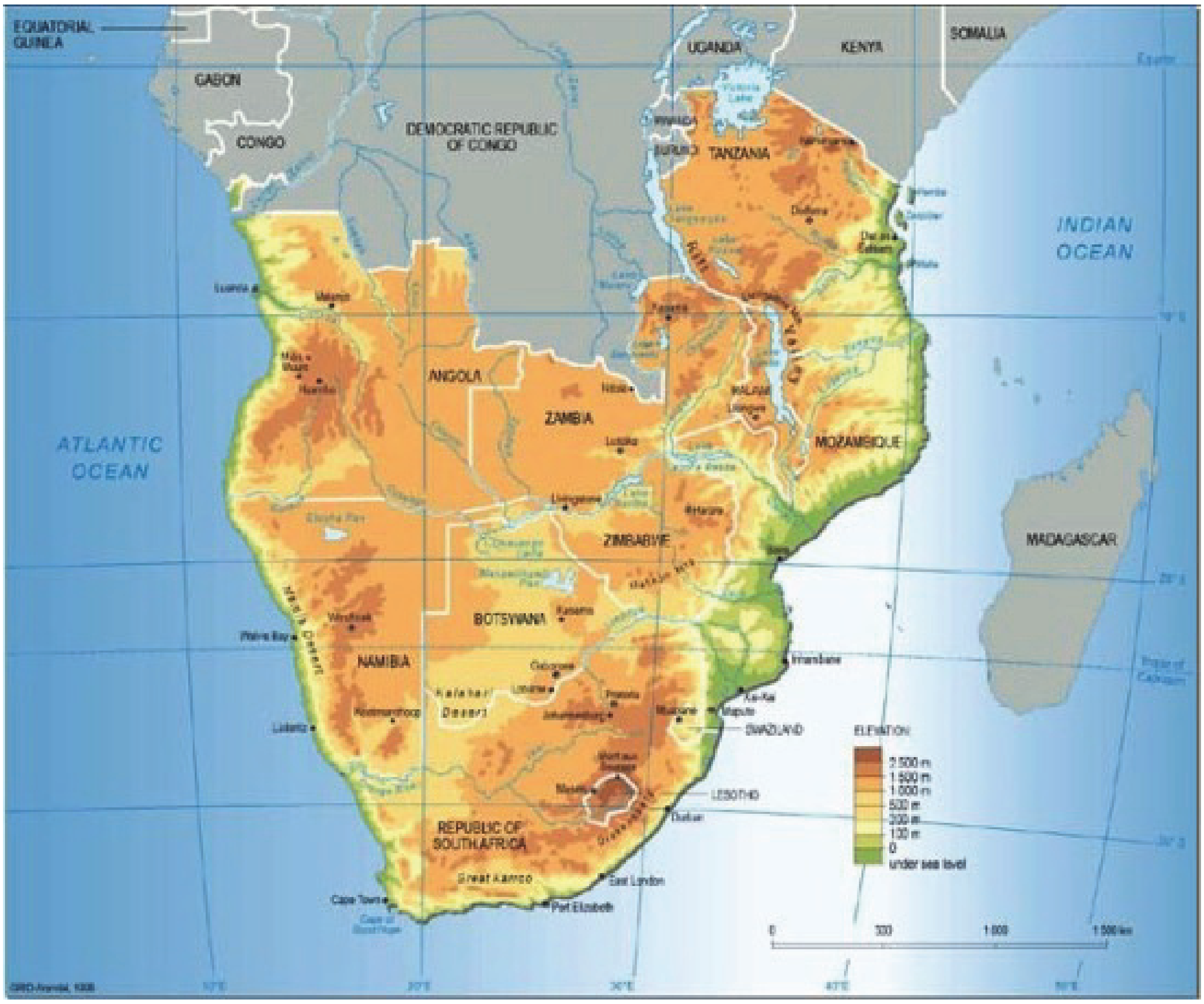

Figure 2-1: Map of South Africa indicating altitude

(Acknowledgement to: http://www.grida.no/graphicslib/detail/southern-africa-topographic-and-political-map_1515)

2.6 Instrumentation Required

- Maximum / minimum thermometer (if not available at works)
- Rain gauge to monitor for rainfall events
- Downloadable voltage monitor or voltage monitoring device indicating max/min voltage since last reset
- Submersible pump fitted with stainless steel strainer to prevent ingress of solids which would possibly foul the impeller. The pumping capacity of the pump would be such that it could pump the required maximum feed rate together with the minimum federate require when controlled by a VSD.
- Pumping control (Option A) – VSD coupled to magflow coupled to manually set timer which will be manually adjusted
- Pumping control (Option B) – PLC controlled VSD coupled to magflow meter which will be pre-programmed to cater for the different pumping regimes that will be required throughout the testing. Desired average flow would be entered followed by the pumping regime (Figure 3.2).
- Electrical protection against power surges and lightening

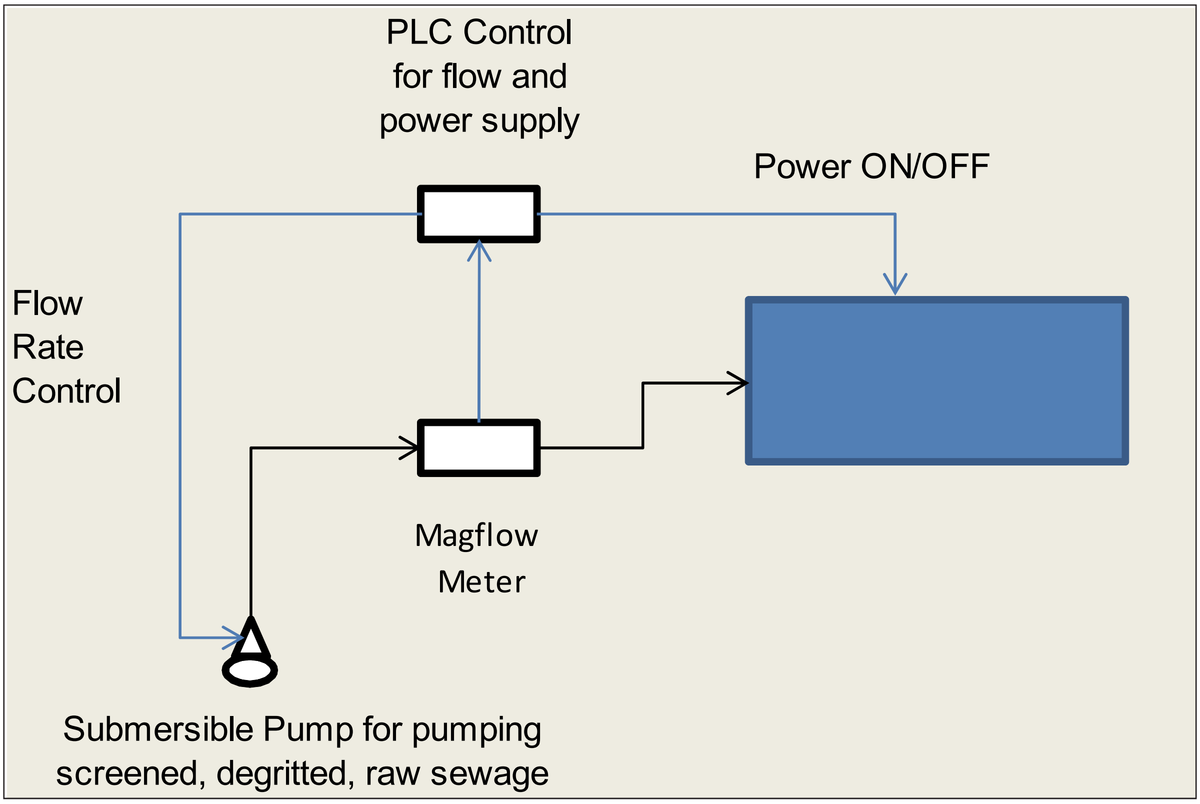

Figure 2-2: Control circuitry for SWWTW Evaluation Facility

2.7 Capex Funding Arrangements

It is recommended that a capital grant be requested from DWA to cover the initial costs for the installation of the testing facility. A public private partnership would make such an initiative viable and credible.

2.8 Operational Funding Arrangements

It is recommended that the SWWTW suppliers be responsible for the operational costs of the testing.

3 FRAMEWORK FOR SWWTW TESTING

The Proposed SWWTW Treatment Efficiency Testing Standard and the rationale behind it can be found in Appendix A and B respectively.

During discussions at the SEWPACKSA/WISA SWWTW Division Workshop held at the WISA Head Office on 29 January 2014 it was obvious that neither body supported the immediate implementation of the Accreditation Standard developed in Appendix A below. It was overwhelmingly apparent that a phased approach would need to be implemented with various options available.

One of the reasons for this is that the proposed test facility would only comprise a limited number of test bays which would result in a situation where it would take a number of years to actually test all the plants available as the proposed testing in the standard takes considerable time. It would be unjust to have some suppliers accredited while others wait in line as this would affect the marketability of the plants awaiting accreditation.

In view of this it was recommended that a staged approach be adopted with a number of options for accreditation. The following recommendations are made regarding the options available for Accreditation:

- Type I – Design Guidelines and Construction Materials/Installation Standards Accreditation – The first stage of Accreditation should be an undertaking to abide by the Design Guidelines contained in Chapter 6 of the WRC Report "Guideline Document: Package Plants for the Treatment of Domestic Wastewater", Research Report K5/1869, 2009 together with the Construction Materials/Installation Standards included in the SWWTW Testing Standard in this document. This will result in the purchaser being assured of a plant constructed and installed in accordance with SANS Standards and having an acceptable process design.
- Type II – Accreditation according to the Treatment Efficiency Test Procedure in Appendix A, but conducted in the field with a plant running at 80% of capacity or more
- Type III – Accreditation according to the Treatment Efficiency Testing Procedure in Appendix A.

4 CONCLUSIONS AND RECOMMENDATIONS

This report provides an introduction to the frameworks of standards which could be adapted for use in South Africa, and discusses their strengths and weaknesses, together with the feasibility of scaling them up for use on larger Works.

It examines the current South African legislative standards for discharge of treated effluent to the environment, together with the corresponding monitoring requirements. It continues to further examine the current General Authorisation Discharge Requirements, and makes strong recommendations with respect to:

- The quality of water to be used for lawn irrigation
- The issue of satisfactory compliance which needs to properly defined, including the method of calculation
- The percentage compliance required for satisfactory compliance to be achieved needs to be carefully and reasonably defined, taking into consideration the variable nature of influent sewage

Leading out of the three recommendations made above, it was specifically recommended that a future Water Research Commission project be established with the dedicated purpose of examining the matter of compliance and developing a suitable system that is acceptable both to the regulator, and the industry stakeholders.

The concept of a SWWTW evaluation facility was examined, and its location requires discussion. Clear recommendations with respect to altitude, climate and quality of available sewage were made. Recommendations were also made with respect to the funding of the facility and its operation.

A framework for SWWTW was discussed and a three tier system recommended after consultation with the industry body SEWPACKSA and the WISA SWWTW Division.

Lastly, a proposed SWWTW Treatment Efficiency Testing Standard was formulated together with the rationale behind its development (Appendices A&B), and the proposed process design standards included as Appendix C.

APPENDIX A: PROPOSED SWWTW TREATMENT EFFICIENCY TESTING STANDARD AND PROCEDURE

PROPOSED SWWTW TREATMENT EFFICIENCY TESTING STANDARD

The manufacture, construction and installation of the SWWTW shall comply with the following SANS standards:

Standard No.	Description
SANS 10400	Plumbing and Drainage Code
SANS 10142	Electrical
SANS 3001	Soil testing
SANS 10100	Concrete – slabs
SANS 10100	Concrete – structural
SANS 966	PVC pipes
SANS 10112	PE pipes
SANS 310	PE Tanks
SANS 53121	GRP tanks
SANS 10162/1	Steel Fabrication
SANS 12944/4 paints/varnishes SANS 121 hot dip galvanising	Steel Coating
SANS 9001	Pumps
SANS 1186	Safety signs

Safety Signage: SWWTW to have adequately signage indicating electrical and bio-hazard status. The Signage is to include manufacturers name, contact details, and capacity.

TREATMENT EFFICIENCY TEST PROCEDURE

[Note: this procedure is based on the CEN 12566-3:2005(E) test procedure which is simpler than the NSF 40-2005]

1. RESPONSIBILITY AND TESTING LOCATION

The plant shall be tested by a laboratory.

The test shall be performed either in the test house of the laboratory or on a user site under the control of the laboratory.

The selection of the test location is the manufacturer's choice but with the agreement of the laboratory.

The test conditions at the location are the responsibility of the laboratory and shall comply with the following conditions.

2. PLANT SELECTION AND PRELIMINARY EVALUATION

2.1. General

Before testing starts, the manufacturer shall provide the laboratory with plant and process design specifications including a complete set of drawings and supporting calculations. Full information concerning the installation and operation and maintenance requirements of the plant shall also be provided.

The manufacturer shall provide the laboratory with information detailing the mechanical, electrical and structural safety of the plant installation to be tested.

2.2. Installation and Commissioning

The plant shall be installed in a way that is representative of the normal conditions of use.

Test conditions, including environment and wastewater temperatures, and compliance with the manufacturer's manual, shall be monitored and recorded and agreed upon by the laboratory. The plant shall be installed and commissioned in accordance with the manufacturer's instructions. The manufacturer shall install and commission all items of the plant prior to testing.

2.3. Operation and Maintenance Procedures During Testing

The plant shall be operated in accordance with the manufacturer's operating instructions. Routine maintenance shall be carried out in strict accordance with the manufacturer's maintenance instructions. Sludge shall only be removed from the plant when specified by the manufacturer in his operating and maintenance instructions. All maintenance work shall be recorded by the laboratory.

During the test period no unauthorised access shall be permitted to the test site. Authorised access shall be supervised by the laboratory.

2.4. Data to be Monitored

The following core parameters shall be monitored in all plants to be tested for both the influent and the effluent:

a. Faecal coliforms;

b. chemical oxygen demand (COD);

c. pH;

d. ammonia;

e. nitrate/nitrite;

f. chlorine (free);

g. suspended solids (SS);

h. electrical conductivity;

i. ortho-phosphate;

Note: fluoride and soaps oils and greases are purposely omitted from the list as fluoride is not removed in biological processes, and the method for soaps oils and greases gives spurious results for final effluents – many laboratories detection limits are higher than 2.5 mg/ℓ.]

The following parameters shall also be measured for reporting purposes:

a. temperature (liquid phase);

b. ambient air temperature (min/max);

c. alkalinity;

d. daily hydraulic flow;

e. dissolved oxygen concentration;

f. total power consumption of the product if applicable

3. PROPOSED TEST PROCEDURE

3.1. Time for Establishment

The manufacturer shall indicate to the laboratory the X-value defined in Table 2.

3.2. Influent Characteristics

Raw domestic wastewater shall be used. The laboratory shall not use grinding equipment on the raw wastewater supply. It is acceptable to coarse screen and remove grit prior to use as long as the influent is of the following quality:

- COD between 300 and 1000 mg/ℓ
- TSS between 200 and 700 mg/ℓ
- Ammonia between 22 and 80 mg N/ℓ

3.3. Daily Flow Pattern for Testing

The daily flow used for testing purposes shall be measured by the laboratory. The daily flow pattern shall comply with Table 1 with a tolerance of ±5%.

Table A-1: Daily flow pattern

Period (hrs)	Percentage of Daily Flow Volume (%)
3	30
3	15
6	0
2	40
3	15
7	0

Where influent is introduced, it shall be done regularly throughout the entire period.

3.4. Test Procedure

3.4.1. General

Routine monitoring shall take place throughout the period of the test procedure. The test schedules listed in Table A-2 shall apply.

Measurements shall be regularly made during each sequence avoiding the day when stress takes place.

The full test shall be carried out during a period of (38 + X) weeks.

After desludging, a period of 1 day shall be allowed for recovery before the programme of tests and sampling is continued.

Table A-2: Test Schedules

Stress Condition	Implication	Notes
Biomass establishment	Nominal flow	X weeks – to be stipulated by supplier
Nominal*-design flow ± 5%	Nominal flow	6 weeks
Under-loading	50% nominal flow	2 weeks
Nominal*	Nominal flow	6 weeks
Power breakdown	24hrs power off	6 weeks – 24 hour power breakdown after 2 weeks of sequence
Nominal*	Nominal flow	6 weeks
Low occupation	No flow	No sampling
Nominal*	Nominal flow	6 weeks
Overloading	Nominal flow plus overload	Capacity ≤ 1.2 m^3/d – 150% Capacity > 1.2 m^3/d – 125%
Power breakdown	24hrs power off	6 weeks – 24 hour power breakdown after 2 weeks of sequence
Under-loading	50% nominal flow	2 weeks
Nominal*	Nominal flow	6 weeks
**Peak flow discharge – conducted once a week during nominal operation*	*200 litres over 3 minutes = a peak flow unit* *Peak flow introduced during period of 40% flow*	*Capacity ≤ 0.6 m^3/d – 1* *Capacity 0.6 – 1.2 – 2* *Capacity 1.2 – 1.8 – 3* *Capacity > 1.8 – 4*

3.4.2. Overload

The laboratory shall adjust the hydraulic daily flow in order to establish the extra load during 48 hours, as shown in Table A-3, at the start of the 2 weeks overloading phase.

Table A-3: Definitions of overloads

Nominal hydraulic flow	Total flow (%)
Capacity ≤ 1.2 m^3/d – 150%	150
Capacity > 1.2 m^3/d – 125%	125

3.4.3. Peak Flow Discharge

A peak flow discharge shall be executed once a week only during the NOMINAL sequences according to the conditions given in Table 4. This peak flow discharge shall not be done during the day used for power breakdown.

One peak flow discharge consists of a volume of 200 ℓ of test influent which shall be discharged, in addition to the daily flow, over a period of 3 minutes, at the beginning of the period with a flow equal to 40% of the daily flow.

Table A-4: Number of peak flow discharge

Nominal hydraulic flow	Total flow (%)
Capacity ≤ 0.6 m^3/d	1
Capacity 0.6 – 1.2 m^3/d	2
Capacity 1.2 – 1.8 m^3/d	3
Capacity > 1.8 – 4 m^3/d	4

3.4.4. Power Breakdown/Machine Breakdown

Where applicable, a power breakdown test shall simulate loss of electric power/mechanical breakdown for 24 hours for the plant equipment. During this power breakdown, influent input shall be maintained according to the daily flow pattern.

This test shall not be done during the day used for peak flow.

When there is optional electrical discharge equipment, the test shall be done with this equipment.

3.5. Influent and Effluent Samplings

The laboratory shall collect and analyse influent samples to determine compliance with the influent characteristics (see 3.2). Effluent sample shall be analysed to determine efficiency ratio.

Inlet and outlet samples shall be flow-based composites over 24 hours taken according to Table 2. Samples shall be taken regularly.

4. SAMPLE ANALYSIS

The determinants specified in 2.4 shall be analysed at an accredited laboratory, or laboratory participating in the Proficiency Testing System (PTS), using Standard Methods for the Examination of Water and Wastewater, published by the American Public Health Association or EPA accredited methods in the case of test kits.

5. COMPLIANCE

The General Authorisation requires compliance of the discharged effluent with the values given in the said document. It does not however define compliance. Under the Green Drop system satisfactory compliance is defined as 95% per parameter.

Both the NSF and CEN testing systems make some allowance for either averaging (NSF) or cleaning (CEN – best 20 of 26 results) of results prior to determining compliance and it suggested that this approach be adopted in South Africa.
As such, if satisfactory compliance is defined as 95%, and the statistical percentage is defined as best 80% of results, then an overall compliance can be defined as 95% of 80% which is equivalent to the 76 percentile of all the analytical results being compliant.

This approach is considered simpler and less likely to result in disputes from spurious analytical results than the NSF method.

In order for accreditation to be conferred the 76 percentile of all the results generated for the parameters indicated under section 2.4 (a-i), with the exception of 2.4 (h) shall be less than or equal to the General Limit Values given in the General Authorisation (as amended). Conductivity is omitted as it is not removed during conventional wastewater treatment processes.

6. TEST REPORT

The report shall contain at least the information specified below:

a. details of the plant tested including information regarding the nominal daily load;

b. information on the conformity of the plant tested with the information provided prior to testing;

c. all data obtained during testing (see 2.4), including the 76 percentile of all the results generated for the parameters indicated under section 2.4 (a-i), with the exception of 2.4 (h), for nominal loading;

d. information on all maintenance and repairs carried out during the test period, including details of desludging frequency, quantity and the volume removed;

e. information on the electrical energy absorbed during the test period;

f. information on any problems, physical or environmental, occurring during the test period. Deviations from the manufacturer's maintenance instructions shall be reported in this section;

g. information detailing any physical deterioration of the plant that has occurred during the test period; e.g. the clogging behaviour of the plant where applicable;

h. information concerning deviations from the test procedure;

i. scaling rules used by the manufacturer to assess the same treatment efficiency and structural behaviour for all the products in the range.

APPENDIX B: RATIONALE BEHIND PROPOSED PERFORMANCE TESTING STANDARD

1. INTRODUCTION

In order to ensure that a SWWTW will work under "real life" conditions it has been suggested that plants be tested or evaluated under such conditions, or, in order to ensure uniformity, simulated "real life conditions".

Performance testing can thus be defined as follows:

"testing of the performance of a specific SWWTW under uniform, simulated "real life" conditions, where "real life" includes stress periods, power outages, washday stress, working parent stress and holiday stress."

Both the NSF (40&245) and CEN standards make provision for performance testing while the AUZ/NZ standard is confined to materials of construction. Since the NSF is an international organisation, it is recommended that their testing regime structure or layout be adopted as a ***basis or template*** for performance testing in South Africa.

The NSF has two applicable standards – NSF 40 for Residential Wastewater Treatment Systems, and NSF 245 for Wastewater Treatment Systems – Nitrogen Reduction. NSF 245 serves as a kind of addendum to NSF 40 in order to include nitrogen removal which is neglected in NSF 40.

The NSF standards use BOD_5 as a measure of organic pollutant oxygen demand. In South Africa Chemical Oxygen Demand would be used, as the measurement of BOD_5 is complicated and prone to errors. The analysis of BOD_5 is also only available in a limited number of laboratories.

The CEN standard includes both BOD_5 and COD. As such BOD_5 will be omitted as per reasons given above. Structure/Layout of the Proposed Standard

The structure/layout from the CEN Standard has been adopted as it is simple and easily understood when compared to that of the NSF Standard.

2. QUALITY OF INFLUENT SEWAGE

Influent wastewater characteristics

- *Discussion:* 30 d average COD shall be between 154 and 461 mg/ℓ *(CEN: COD between 300-1000 mg/ℓ)*
- *Discussion:* 30 d average TSS shall be between 100 and 350 mg/ℓ *Remove reference to TSS – what is the point of it? (CEN: SS between 200 and 700 mg/ℓ)*
- *Discussion:* 30 d average TKN shall be between 35 and 70 mg/ℓ *Appears to be high for domestic sewage (CEN: KN between 25 and 100, or ammonia as N between 22 and 80 mg/ℓ)*
- Alkalinity > 175 mg/ℓ as $CaCO_3$
- Temperature 10-30°C
- pH 6.5 to 9

3. DAILY FLOW PATTERN AT NORMAL DAILY FLOW

Table B-1 provides a comparison of the daily flow pattern to be dosed to the SWWTW according to the two Standards.

Table B-1: Comparison of Daily Flow Patterns

CEN Standard		NSF Standard	
Time	**Percentage of Daily Flow Volume**	**Time**	**Percentage of Daily Flow Volume**
0h00-6h00	0	0h00 -6h00	0
6h00-9h00	30	6h00-9h00	35
9h00-12h00	15	9h00-11h00	0
12h00-18h00	0	11h00-14h00	25
18h00-20h00	40	14h00-17h00	0
20h00-23h00	15	17h00-20h00	40
23h00-24h00	0	20h00-24h00	0

The CEN daily flow pattern is considered to be more representative of normal South African flows.

4. COMPARISON OF TEST SCHEDULES AND SAMPLING REQUIREMENTS

Both Standards test the performance of the SWWTW over a period of time during which the SWWTW is subjected to a nominal flow which is periodically supplemented by a number of stress events to simulate “real life” events.

A comparison of the test schedules is given in Table 2 below, while a comparison of the sampling requirements is given in Table 3.

From Table 2 it can be conclude that the NSF stress loading regime is very complex and difficult to simulate unless an advanced PLC/SCADA system is used. The CEN system is much simpler to adopt.

Table 3 shows that the NSF testing regime sampling provides for a minimum of 96 data days (in a maximum of 34 weeks evaluation) while the CEN testing regime sampling provides for 26 data days (in a maximum of 38 weeks evaluation). This equates to 192 samples (i.e. influent and effluent) for the NSF testing, while the CEN testing equates to 52 samples (i.e. influent and effluent).

Table B-2: Comparison of Test Schedules

CEN Standard			NSF Standard		
Stress Condition	**Implication**	**Notes**	**Stress Condition**	**Implication**	**Notes**
Biomass establishment	Nominal flow	X weeks – to be stipulated by supplier	Design loading	• 35% flow during 6-9am • 25% flow during 11am - 2pm • 40% flow during 5-8pm • Individual doses should not exceed 38 litres	16 weeks of design loading
Nominal*-design flow ± 5%	Nominal flow	6 weeks	Wash day stress	• 3 wash days in a 5 day period • Wash days separated by 24 hour period	5 days
Under-loading	50% nominal flow	2 weeks	Design loading		1 week
Nominal*	Nominal flow	6 weeks	Working parent stress	• 5 consecutive days • 40% of daily hydraulic capacity between 6-8am • Remaining 60% between 5-8pm including one wash cycle and two rinse cycles	5 days
Power breakdown	24hrs power off	6 weeks – 24 hour power breakdown after 2 weeks of sequence	Design loading		1 week
Nominal*	Nominal flow	6 weeks	Power equipment failure stress	• 40% of daily hydraulic capacity between 5-8pm on day of power failure • Power switched off at 9pm and left off for 48h with no dosing • After 48 hours switch back on and dose with 60% of daily hydraulic capacity over 3 hours including one wash cycle and two rinse cycles	3 days
Low occupation	No flow	No sampling	Design loading		1 week

CEN Standard			NSF Standard		
Nominal*	Nominal flow	6 weeks	Vacation stress	• Dose 35% of daily hydraulic capacity between 6-9am and 25% between 11am - 2pm • Cease dosing between for 8 consecutive days • On 9th day dose 60% of daily hydraulic capacity between 5-8pm, including 3 wash cycles and 6 rinse cycles	9 days
Overloading	Nominal flow plus overload	Capacity ≤ 1.2 m^3/d – 150% Capacity > 1.2 m^3/d – 125%	Design loading		2.5 weeks
Power breakdown	24hrs power off	6 weeks – 24 hour power breakdown after 2 weeks of sequence			
Under-loading	50% nominal flow	2 weeks			
Nominal*	Nominal flow	6 weeks			
**Peak flow discharge – conducted once a week during nominal operation*	*200 litres over 3 minutes = a peak flow unit* *Peak flow introduced during period of 40% flow*	*Capacity ≤ 0.6 m^3/d – 1* *Capacity 0.6 – 1.2 – 2* *Capacity 1.2 – 1.8 – 3* *Capacity > 1.8 – 4*			

Table B-3: Comparison of Sampling Requirements

CEN Standard			NSF Standard		
Stress Condition	**Samples**	**Time elapsed**	**Stress Condition**	**Samples**	**Time elapsed (wks)**
Design loading	5/week	16 weeks	Biomass establishment	None	X
Wash day stress	Influent and effluent samples on day stress initiated	5 days	Nominal	4	6
Design loading	6/week*	1 week	Under-loading	2	2
Working parent stress	Influent and effluent samples on day stress initiated	5 days	Power breakdown	5	6
Design loading	6/week*	1 week	Low occupation	0	2
Power equipment failure stress	Influent and effluent samples on day stress initiated	3 days	Nominal	3	6
Design loading	5/week*	1 week	Overloading	2	2
Vacation stress	Influent and effluent samples on day stress initiated	9 days	Power breakdown	5	6
Design loading	6/week* for one week then 5 per week for 1.5 weeks	2.5 weeks	Under-loading	2	2
			Nominal	3	6

*24 hours after completion of stress influent and effluent samples taken for 6 consecutive days

5. ANALYTICAL REQUIREMENTS OF THE NSF 40/245 AND CEN STANDARDS

The analytical requirements and costs for the NSF and CEN standards are indicated in Tables 4 and 5 below, together with an indication of the associated costs (2012). It is immediately evident that the costs of analysis for the NSF Standard are almost double those of the CEN Standard.

Table B-4: NSF standard analytical requirements and costs (2012 prices)

Analysis	Influent	Effluent	Total no of analyses	Cost	Total Cost
BOD5	96		96	R 99.00	R 9,504.00
COD5		96	96	R 99.00	R 9,504.00
TSS	96	96	192	R 72.00	R 13,824.00
pH	96	96	192	R 29.00	R 5,568.00
Temperature	96	96	192		R 0.00
Dissolved oxygen		96	96	R 70.00	R 6,720.00
Alkalinity	96	96	192	R 55.00	R 10,560.00
TKN	96	96	192	R 200.00	R 38,400.00
Ammonia	96	96	192	R 78.00	R 14,976.00
Nitrate/nitrite		96	96	R 99.00	R 9,504.00
Grand Total					R 118,560.00

Table B-5: CEN standard analytical requirements and costs

Analysis	Influent	Effluent	Total no of analyses	Cost	Total Cost
BOD5	52		52	R 99.00	R 5,148.00
COD5		52	52	R 99.00	R 5,148.00
TSS	52	52	104	R 72.00	R 7,488.00
pH	52	52	104	R 29.00	R 3,016.00
Temperature	52	52	104		R 0.00
Dissolved oxygen		52	52	R 70.00	R 3,640.00
Alkalinity	52	52	104	R 55.00	R 5,720.00
TKN	52	52	104	R 200.00	R 20,800.00
Ammonia	52	52	104	R 78.00	R 8,112.00
Nitrate/nitrite		52	52	R 99.00	R 5,148.00
Grand Total					R 64,220.00

6. SATISFACTORY COMPLIANCE

A comparison of the CEN and NSF Satisfactory Compliance Calculation is given in Table 6 below. The CEN method is simpler than that of the NSF method which comprises a little more statistical understanding.

Table B-6: Comparison of CEN and NSF Calculation of Satisfactory Compliance

CEN Standard	NSF Standard
"Treatment efficiency declaration" for COD, BOD, and SS, where the efficiency ratio, R is calculated as R = Pi – Po/Pi, where Pi is the influent and Po is the effluent value	**$CBOD_5$** • 30 d average shall not exceed 25 mg/ℓ Equivalent COD – 33 mg/ℓ • 7d average shall not exceed 40 mg/ℓ Equivalent COD – 61.5 mg/ℓ
The mean value of R is calculated for the 20 efficiency ratios obtained during the NOMINAL flow sequences and reported	**TSS** • 30 d average shall not exceed 30 mg/ℓ • 7 d average shall not exceed 44 mg/ℓ
The individual values for *UNDERLOADING* sequences (4 efficiency ratios) and *OVERLOADING* sequences (2 efficiency ratios) shall be stated in the report.	**Total nitrogen** • Average total nitrogen concentration of all effluent samples shall be less than 50% of the average total nitrogen concentration of all the influent samples
	pH • pH of individual effluent samples shall be between 6.0 and 9.0
	Effluent concentration excursions • During first calendar month of performance testing and evaluation the above value should be multiplied by a factor of 1.4 for compliance purposes
	Colour • Report, but no criteria
	Odour • Overall rating of each of the three diluted (1:1000) composite samples shall not be offensive
	Oily film and foam • Oily films and foaming shall not be visually detected in any of the undiluted composite effluent samples

7. COMPARISON OF STANDARDS

Table 7 below gives a summarised comparison of the South African General Limit Values and the NSF and CEN standards for purposes of comparison.

Table B-7: Comparison of Standards

Parameter	South African General Authorisation (GLV)	NSF 40/245	CEN
pH	5.5-9.5	6.0-9.0	Not stipulated
TSS (mg/ℓ)	25	30	Efficiency ratio
COD (mg/ ℓl	75	33/61.5	Efficiency ratio
NH3 (mg/ ℓl	6	Not specified	Not stipulated
Nitrate/Nitrite (mg/ ℓl	15	Not specified	Not stipulated
Total N (mg/ℓ)	21*	<50% of influent TKN	Not stipulated
Faecal Coliforms (cfu/100 mℓ)	1000	Not specified	Not stipulated
Chlorine (free) (mg/ℓ)	0.25	Not specified	Not stipulated

***DERIVED EX AMMONIA + NITRATE/NITRITE**

APPENDIX C: PROPOSED PROCESS DESIGN STANDARDS

1. PROPOSED PROCESS DESIGN STANDARDS

The following design criteria must be adhered to.

1.1. System Location

In the case of domestic homes the system should be located within 15 m of the dwelling. This is to ensure that the owner of the dwelling is aware of any nuisances being generated by the system and that the unit is nearby in terms of operational and maintenance requirements.

In the case of other applications such as housing developments, hotels and hostels the system should not be situated more than half-way between the closest building it serves and the property boundary. Where this is not possible a motivation will need to be forwarded to the relevant authority for approval.

1.2. System installation

Below ground units, such as septic tanks shall be installed such that:

- They do not "float" out of the ground when emptied if the soil is waterlogged; and
- All inspection covers are above ground level for easy access.

Above ground units shall be installed such that:

- The plant will be installed on a suitably sized and reinforced concrete slab that will support the weight of the unit when full;
- The plant should have reasonable access to allow operations and maintenance work to be performed;
- The plant should be constructed (supported, jointed, and protected) to avoid the likelihood of damage from superimposed loads or normal ground movement; and
- Access to the site is restricted in order to prevent theft and vandalism.

1.3. System Specification Plate

Every plant shall have a permanently fixed aluminium plant specification plate (similar to that affixed to electric motors) bearing the following information stamped or printed on it:

- Name and contact details of supplier;
- Year of manufacture;
- Total installed power;
- Power supply requirements (voltage, phase, frequency);
- Design influent flow rate; and
- Design COD and TKN concentrations.

A clearly displayed warning sign (e.g. “DO NOT DRINK WATER) must also be included.

1.4. Operational and Maintenance Manual (O & M)

An O & M Manual including at least the following sections must be supplied with the installed plant:

- A system specification sheet containing the following:
  - Date of manufacture;
  - Serial and batch number;
  - Design flow;
  - Power rating;
  - Phase rating;
  - Design COD and TKN concentration;
  - Design effluent concentrations;
  - Disinfection method;
  - Discharge method for treated water; and
  - Sludge production and disposal.
- A process flow diagram;
- A description of how the process operates,
- A description of any limitations the system may have (e.g. what toxic household chemicals should be avoided, exclusion of stormwater from the system);
- A section describing the various operational and maintenance procedures required (e.g. regular checks on pump and blowers, disinfection chemicals, etc.);
- A matrix indicating the frequency of each operational and maintenance procedure;
- A section indicating the monitoring requirements and frequency of sampling/analysis required;
- A table of acceptable discharge limits;
- A “FAQ” (frequently asked questions) section dealing with common questions asked and resolving common misperceptions;
- A troubleshooting matrix;
- Contact details of the supplier;
- A signed copy of the initial maintenance contract; and
- A declaration by the owner that the operational and maintenance procedure was explained and understood.

1.5. Technology Selection

The most commonly available technologies are:

- Activated sludge
- Rotating bio-contactors
- Submerged bio-contactors

1.6. Capacities

The majority of plants are designed as modular systems. Each plant will be made-up to the specifications supplied by the consumer. Refer to design parameters for criteria for determining the capacity of the plant.

1.7. Design parameters

In terms of ease of design, large plants are always much simpler to design than small plants. There are a number of reasons for this, including:

- The smaller the plant the less likely it will get regular maintenance;
- Components which are small in capacity such as pumps and aerators are more difficult to source, and tend not to be as durable as larger components;
- Piping is difficult to design – a small bore pipe is easily blocked while solids tend to deposit in larger pipes due to low velocities;
- The variance in flows is more acute in a smaller plant – this is one of the most marked problems;
- Disinfection is more difficult for a small flow and it is very difficult to get an accurate dose of disinfectant in a small system; and
- There is less buffering volume in terms of a toxic or inhibitory event.

This is meant to be a guide to assist with the design of plants. Each situation will be unique and must be treated as such when proposing a suitable technology.

1.7.1. Domestic Wastewater Quality

It is important to ensure that the quality of the domestic wastewater inflow is properly evaluated. The quality of domestic wastewater is dependent on the source. In the case of office block /supermarket complexes the domestic wastewater quality tends to be stronger than normally expected due to a lack of low COD flows such as bathing and laundry. Total Kjeldahl Nitrogen (TKN) and thus ammonia concentrations are generally very high (100-200 mg N/ℓ) due to the use of “un-flushed” urinals in the toilets.

These high TKN’s (ammonia in the influent of a works is generally only half the value of the TKN) require special treatment both in terms of process configuration and alkalinity requirements (if the alkalinity is too low, which it generally will be, then the pH will drop below 7 and nitrification will cease, resulting in a high ammonia content effluent).

In most cases the high ammonia domestic wastewater will need to be treated in a two stage aerobic process to achieve full nitrification, and de-nitrification to below 15 mg N/ℓ may well be impossible, without the addition of supplementary COD after nitrification.

It is thus highly recommended that comprehensive sampling be performed at any installation which is not simply a domestic home to check on the COD and TKN concentrations.

Supermarkets and restaurants also tend to have high oils and fats due to the indiscriminate dumping of cooking oil and scrapings down the drains. Experience has shown that if oil/fats enter SWWTW they will cause serious problems in the process, if not total failure. Conventional grease traps are not effective in dealing with the high loads of oil/fats from these enterprises and thus their sewer lines need to be fitted with septic tanks with at least a full day's retention time in order to retain the oils/fats. The accumulated oils and fats will need to be removed from time to time by vacuum tanker for suitable disposal. Smaller grease traps are seldom serviced and when they are they are, the contents is often simply emptied into the discharge compartment, negating their whole purpose. This cannot be as easily achieved in a septic tank.

Commercial premises may also suffer from toxicity problems from time to time due to the liberal use of disinfectants in drains to combat odour problems.

Typical South African municipal domestic wastewater quality is given in Table C-8 below, adapted from Ekama, 1984.

Table C-8: Typical domestic wastewater characteristics for South Africa

Determinant	Unit	Typical domestic wastewater characteristics
COD	mg $O_2.\ell^{-1}$	500-800
SS	mg.ℓ^{-1}	270-450
TKN	mg.ℓ^{-1}l	35-80
TP	µg.ℓ^{-1}	8-18

Where actual sampling is not conducted in a programmed manner, then the domestic wastewater concentration should be taken as the upper limit of the range indicated above. If an unusual domestic wastewater is suspected, it is important to obtain a sample of domestic wastewater from a similar source and evaluate the quality in terms of a minimum of COD, Total Kjeldahl Nitrogen and Total Phosphate. The size of the reactor vessels will then need to be adjusted accordingly.

1.7.2. Domestic wastewater flow rates

Any domestic wastewater treatment plant operates optimally under steady flow and load. The smaller the domestic wastewater collection system feeding a plant, the more marked flow spikes and diurnals will be. This is well documented. Where the characteristics of the flow is uncertain, and suspected to be unusual it is imperative to evaluate the flow regime, either of the facility in question, or a similar one.

Measuring actual domestic wastewater flows is a difficult and not particularly pleasant job, and a simpler method is to measure the potable water supply to the facility by means of the water meter supplying the facility. The meter should be read hourly for 24 hours on a typical usage day and the flow per hour calculated. The results can then be evaluated to determine the volume of the balancing or equalisation tank.

As a general rule an equalisation tank of 12 hours holding capacity should be installed. Where a two stage septic tank of total hydraulic retention time of 24 hours precedes the plant, then the second compartment can serve as the holding tank. Three stage tanks are preferred.

In the case of normal domestic accommodation, the minimum volumes of wastewater generated should be calculated as in Table C-9.

Table C-9: Minimum volumes of wastewater

Type of accommodation	Minimum volume per capita (ℓ/day)	Minimum volume per unit (ℓ/day)
Normal domestic including townhouse, duplexes, etc.	200	[(Number of bedrooms -1) x 200] + 400
Retirement villages (stand-alone units)	200	500
Schools, hostels or hotels	75	-

For higher volume domestic installations such as schools, hostels or hotels, a per capita volume of 75/ℓ day has been suggested. However, volumes should be established from similar institutions, and should be validated against water use according to the supply meter (sewer return being in the range of 70-90% depending on the degree of gardening or washing down of driveways, etc. taking place).

If a reduced flow is used in some unusual application, then the flow must be fully substantiated to the applicable authority and documented in the O&M manual.

1.7.3. Flow Equalisation Tank or Septic Tank

Since flows are highly variable through the day it is important to install a flow equalization tank. This usually takes the form of a 12 hour retention tank from which the domestic wastewater is fed to the plant at a constant rate using a pump.

The flow equalization tank will often take the form of a two stage septic tank. This is ideal in that the first compartment usually retains most of the debris and grit in the domestic wastewater, ensuring that it does not reach the pump that feeds the aerobic treatment section. Where a septic tank is installed it should have a capacity of not less than 24 hours flow. This should increase at institutions where the debris content of the domestic wastewater is expected to be high, e.g. clinics, hospitals and schools.

An equalisation tank of 12 hours holding capacity or a double chamber 24 hour retention septic tank should be installed ahead of the plant.

1.7.4. Screening

Screenings are a problem in all domestic wastewater treatment plants with major implications in terms of pipe and pump blockages. As the flows diminish and pipe sizes are reduced the problems become more exaggerated. It is thus important to remove as much debris as possible. The international trend is towards the use of fine screens (5 mm) even in large plants, as the savings in maintenance costs are significant.

Septic tanks appear to provide an excellent form of screening and degritting, but where these are not fitted it is essential to provide some form of screening to prevent blockages and pump breakages. This is even more crucial where the system is serving a communal or institutional building where unusually large amounts of debris are often deposited into the toilet (e.g. rags and plastic packets).

Traditional bar screens such as those found on small conventional wastewater works are not particularly suitable in many applications as they require daily cleaning, and may cause odours. A useful alternative is the use of a suitably sized bucket made from stainless steel with 5 mm perforations at the inlet to equalisation tank. The perforated stainless steel is available from steel merchants.

The bucket screen system worked extremely well in a WRC research project, and tends to be self-cleaning for faecal matter as the turbulence of the incoming domestic wastewater breaks up any organic solids. Obviously the size of the bucket depends on the flow. The regularity of cleaning the bucket will need to be assessed according to the application, but will obviously be more frequent in communal applications.

One big advantage of the bucket screen is the small perforation size, and the prevention of the screenings being "raked over the top" of the screen by maintenance staff – a fairly common problem. Access to the bucket screen should be designed in such a way that it is convenient to remove and replace the bucket.

1.8. Design Criteria

1.8.1. Septic Tanks

The design parameters for septic tanks preceding SWWTW are simple and can, for the purposes of this report, be condensed to a minimum of one day's hydraulic retention time, split into a minimum of two equal sized chambers. Three chamber septic tanks are preferred. There are a number of important design details for the tanks, and there are numerous texts which detail these.

It is also advisable to consider building a simple anaerobic baffled reactor (ABR) which gives enhanced COD reduction for the same space requirement. Obviously this is highly desirable as the lower the COD reaching the aerobic stage the less the organic load and the better the performance. The ABR will however require a final storage chamber of 12 hours retention time to achieve equalisation for the subsequent aerobic treatment stage.

1.8.2. *Activated Sludge*

Activated sludge is an excellent technology for domestic wastewater treatment, providing a good quality effluent if properly designed constructed and operated. The chief problems incurred with activated sludge are maintaining the correct sludge age; and preventing solids carryover from the secondary clarifiers.

Carryover of solids from the clarifier is a big problem in activated sludge and needs to be controlled in two ways. The mixed liquor concentrations must be maintained in the correct range by careful control of sludge wasting, and flow peaks must be eliminated as far as possible. The reason for this is that the solids-liquid separation in the clarifier is a difficult one due to the small density difference between the two phases due to the activated sludge floc comprising mainly water.

Thus settling is slow and upflow rates have to be limited. This is largely achieved by having an equalisation tank upfront, and the feed pump being controlled by a variable speed drive (VSD).

Activated sludge plants can be preceded by a septic tank, the advantage being that screenings are well removed, and the anaerobic process will remove up to 50% of the COD with minimal solids yield, thus lowering the COD entering the activated sludge plant and reducing the sludge wasting. The septic tank also serves to attenuate flows.

Simple design parameters for activated sludge are given in Table C-10.

Table C-10: Activated sludge design parameters.

Sludge Age (days)	Sludge loading rate (kg COD applied per day per kg MLSS)*	Recommended sludge density (mg MLSS/ℓ)
30	0.12	5 200
25	0.13	4 400
20	0.15	3 900
15	0.19	3 500
10	0.24	3 300
5	0.39	3 200

*These loadings originate from WISA (1988), but have been converted to COD from BOD assuming BOD = 0.65 x COD

The reactor volume (V_r) in m^3 can then be calculated as:

$$V_r = \frac{M_t}{X}$$

Where: M_t = Total mass of sludge required (kg); and

X = Recommended sludge consistency (kg/m^3)

The sludge age is usually 20 days or greater in order to ensure the sludge is properly stabilised, and will not cause a fly or odour problem while drying.

The volume of sludge to be wasted on a daily basis is the volume of the reactor divided by the sludge age (when wasting from the reactor). When wasting from the return sludge line, the volume is calculated as follows:

$$Volume\ wasted = \frac{Volume\ of\ reactor * MLSS}{Sludge\ age * RSS}$$

Where: RSS = Concentration of sludge in return sludge

One of the biggest problems in activated sludge is the management of the sludge, which is either dried on drying beds or dewatered by mechanical means. A novel method is the use of Bidem or similar synthetic sacks to pump the sludge into, capturing the solids, but allowing the water to pass through.

A simple means of simplifying the sludge handling problem is to switch off the aeration and feed pump and allow the sludge to settle for say an hour or two. During this time the sludge concentration at the base of the reactor should concentrate approximately five fold, meaning that the wasting volume can be reduced by a factor of 5.

Worked Example

Consider an activated sludge system with a reactor dimension of 5 m x 5 m x 4 m and a settler volume of 18 m^3. Calculate the volume of:

a) Un-thickened sludge to be wasted per day to achieve a 25 day sludge age

b) Un-thickened sludge to be wasted per week day (i.e. 5 days a week) to achieve a 25 day sludge age

c) Thickened sludge to be wasted per day to achieve a 25 day sludge age

d) Thickened sludge to be wasted per week day (i.e. 5 days a week) to achieve a 25 day sludge age

e) Thickened sludge to be wasted once a week to achieve a 25 day sludge age

Answer

Generally the settler contains a negligible amount of sludge which means that it can be neglected in the calculations. The reactor sludge volume is then calculated as:

Reactor volume	=	length x breadth x depth
	=	5 m x 5 m x 4 m
	=	100 m^3

The sludge volumes to be wasted can then be calculated as follows:

a) *Daily sludge volume to be wasted*	= sludge volume of reactor / 25[1]
	= 100 m^3 / 25 days
	= 4 m^3 /day

[1] The defined sludge age in days

b) *Weekday sludge volume to be wasted*	= 4 m^3/day X 7 / 5
	= 5.6 m^3/weekday
c) *Daily thickened sludge volume to be wasted*	= 4 m^3 /day / 5[2]
	= 0.8 m^3 /day
d) *Weekday thickened sludge to be wasted*	= 0.8 m^3 /d x 7 / 5
	= 1.12 m^3/weekday
e) *Once weekly thickened sludge to be wasted*	= 0.8 m^3 /d x 7
	= 5.6 m^3/week

The above calculations (a-e) shows the advantage to be gained by allowing the sludge to settle before wasting. The example also gives the volumes for wasting for weekdays and once a week. The latter is useful in that wasting is probably the most important operating function and can be done by a more skilled person who visits the plant weekly. This is ideal as the aeration must be switched off together with the feed pump. The sludge wasting may well be risky in the hands of an un-skilled labourer, particularly when the aeration and feed pump need to be switched off simultaneously. Potential exists for an improper restart to occur.

To determine the exact sludge thickening factor fill a 1 litre volumetric cylinder to the 1 000 mℓ mark with well aerated mixed liquor and leave for 1 hour. Measure the volume of the sludge at the end of the hour and then divide 1 000 by the settled volume.

If the sludge to be wasted above is to be dried on drying beds then a once weekly wasting scenario is also an advantage in that fewer beds are required. A typical scenario would be as follows:

Sludge volume to be wasted once weekly	= 5.6 m^3/week
Initial drying bed depth of sludge	= 0.3 m
Required area of bed	= 18.7 m^3
Bed dimensions	= 3 m x 6.2 m

How many drying beds are needed? In a dry, Highveld, climate the bed will take about five days to dry, and can then be cleared and left to “air” for a day or two before the next use – so theoretically one bed may be enough. However if it rains for a number of days or there is a labour problem, then one gets behind in wasting and can’t catch up. Thus an extra bed is needed.

At the coast rain can be experienced daily for a week and the humidity is often high. This can result in a bed taking two weeks to dry in inclement weather and thus two beds will be enough to “cope”. However, if the weather remains wet for a month then there will be problems so it is advisable to have a third. Where it rains frequently it is advisable to have the drying beds under a waterproof shelter to prevent the sludge from re-wetting and smelling.

[2] The factor by which the sludge generally thickens

Greater efficiencies can be obtained in clearing drying beds if they are either paved or the sand covered with shade-cloth. When paving drying beds, the bricks used should have holes through them (e.g. the normal 3 hole type). Reject face-bricks are excellent for this use as they are harder. The holes through the bricks should be in the vertical plane with sand swept into them until they are filled. A little extra sand is then raked over the brick surface. Cement pavers are totally unsuitable for this use as they do not have holes through them and their pores blind rapidly.

Where shade cloth is used to cover the sand the dried sludge can be "shaken" into the centre of the cloth and then dragged to a disposal point.

Drying beds should wherever possible be placed on a north facing slope.

The return sludge flow rate from the clarifier should be the same as the peak inflow rate. Flow equalization will thus decrease the pumping capacity needed.

The oxygen requirement for COD removal and nitrification must be calculated separately. The oxygen requirement for COD removal depends upon the sludge age, and is shown in Table C-11.

Table C-11: Oxygen requirements to achieve nitrification in activated sludge

Sludge Age (days)	**Sludge loading rate (kg COD applied per day per kg MLSS)**	**Dissolved oxygen required for COD oxidation (kg O_2.kg COD^{-1} removed)***
30	0.118	1.092
25	0.134	1.066
20	0.154	1.021
15	0.186	0.949
10	0.24	0.845
5	0.389	0.689

*These oxygen requirements originate from WISA (1988), but have been converted to COD from BOD assuming BOD = 0.65 x COD

The oxygen requirement for nitrification is simpler to calculate, being 4.5 mg O_2 per 1 mg ammonia as N nitrified. It is also important to note that nitrification consumes alkalinity as it generates nitric acid. For every 1 mg as N ammonia that is nitrified 7 mg of alkalinity as $CaCO_3$ is consumed. If de-nitrification takes place half of this is recovered.

It is important to realize that the pH will drop with nitrification, especially in soft waters, and this will inhibit further nitrification. When this occurs slaked lime must be added to compensate for the alkalinity loss. The simplest way of achieving this is to flush a calculated amount of slaked lime down the toilet once or twice a week in the case of small plants.

For larger plants slaked lime can be added twice a week directly into the reactor. The authors have had good success with this in larger SWWTW. The addition of slaked lime also produces a more robust floc.

Worked example:

Domestic wastewater from an office block and shopping centre enters an aerobic treatment plant (all technologies have the same alkalinity requirements) with an alkalinity of 200 mg $CaCO_3$/ℓ, a TKN of 155 mg N/ℓ, and an ammonia of 70 mg N/ℓ. A final effluent alkalinity of 80 mg/ℓ is recommended to protect the concrete work and keep the pH stable. Is there sufficient alkalinity in the domestic wastewater, and if not how much slaked lime will need to be added to the influent per day if the flow is 100 m^3/day. Assume that total nitrification and de-nitrification takes place.

Answer

The ammonia entering a works is only part of the total nitrogen which is given by the TKN – thus one only uses the TKN in calculations. If only ammonia figures are available then double them to get an approximate TKN value.

Alkalinity consumed in nitrification	= 155 mg N/ℓ x 7
	= 1 085 mg $CaCO_3$/ℓ
Alkalinity returned by de- nitrification	= 155 mg N/ℓ x 3.5
	= 542.5 mg $CaCO_3$/ℓ
Net requirement for alkalinity	= 1 085-542.5 mg $CaCO_3$/ℓ
	= 542.5 mg $CaCO_3$/ℓ
Alkalinity available in domestic wastewater	= 200 mg/ℓ
Alkalinity shortfall	= 542.5-200 mg $CaCO_3$/ℓ
	= 342.5 mg $CaCO_3$/ℓ
Alkalinity required	= 342.5 + 80[3] mg $CaCO_3$/ℓ
	= 422.5 mg $CaCO_3$/ℓ
Slaked lime addition required	= 422.5 x (74/100)[4] mg $CaCO_3$/ℓ
	= 312.6 mg/ℓ slaked lime

So far the mass (mg) of slaked lime per litre has been calculated – this now needs to be converted to kilograms per 100 m^3 or kilograms per day. This is done as follows:

Slaked lime needed per day (100 m^3/d)	= 312.6 mg/ℓ x 100 m^3/d x 0.001[5]
	= 31.3 kg/d

Slaked lime is calcium hydroxide and should be handled with care. Agricultural lime, which is calcium carbonate, cannot be used as it does not readily dissolve. Slaked lime is available at most builders' merchants. The total amount of lime can be added in one daily dose, or a slurry fed into the reactor over a day.

Aeration of activated sludge can be achieved through surface aeration, or submerged aeration. Surface aerators tend to be more robust and simple to install and are easily removed for repairs using a crane if necessary on bigger plants. They are also available in floating or fixed configuration – the fixed configuration gives better energy efficiency, but requires extra construction to give a solid mounting point.

[3] Residual alkalinity required to protect concrete structures and stabilize alkalinity
[4] Conversion from mg $CaCO_3$/ℓ to $Ca(OH)_2$
[5] 0.001 is a conversion factor to give kg/d

Submerged aeration is neater in terms of aesthetics than surface aeration and is also quieter if radial blowers are installed in a sound proof blower room. They require very little maintenance apart from air-filter changes, while surface aerators often require the gearbox oil level to be checked which can be difficult in the case of a floating aerator. Submerged aeration is generally more complicated to install, but the air supply is useful for air-lift pumping operations such as the return activated sludge.

Aeration equipment performance is usually specified in terms of kg $O_2.kWh^{-1}$, which allows one to calculate the power of the aeration equipment. There are however adjustments which need to be made for motor efficiency, alpha factor (related to impurities in domestic wastewater such as salts and detergents) and beta factor (related to altitude, temperature and residual dissolved oxygen). Care must also be taken with submerged aeration to consider the path length of the bubbles through the domestic wastewater (i.e. the depth of the domestic wastewater).

Clarifier design must be carried out with much care, and a conservative attitude should be adopted as carryover of solids is detrimental to the quality of the final effluent, giving high suspended solids and COD. Problems will also be experienced in disinfection as the result of increased chlorine demand from the organics and the shielding of bacteria within flocs.

Few SWWTW clarifiers will have motorized sludge scrapers. In view of this, the walls must not be less than 60 degrees to the horizontal for the sludge to slide/fall to the bottom of the clarifier. The walls should also be as smooth as possible, with concrete structures being painted, after thorough sanding with a carborundum block, with epoxy type paint to reduce the roughness.

The inlet to the clarifier must be carefully designed to prevent short circuiting of the sludge to the weir. It is usual to have a stilling chamber in the centre of the clarifier which reduces the turbulence of the mixed liquor being fed into the clarifier. This stilling chamber should extend below the surface of the water in the tank to approximately one third of the total depth of the tank. It is important to minimise turbulence in the clarifier to ensure good settling.

Care should be taken in designing the weir to ensure that it can be properly levelled to prevent short circuiting. Where an annular ring pipe is used always have the holes on the outside of the pipe in order to capture debris on the inside of the ring and thus prevent blocking.

The most important design parameter for clarifier design is not to exceed 1 m/h upflow rate at peak dry weather flow. Solids flux loadings should not exceed 8 kg.m^2/h. Should either of these two parameters be exceeded it is likely that clarification will fail, solids carryover will occur and the effluent quality will fail.

Submersible pumps or airlift pumps can be used for the recycling of the return activated sludge.

1.8.3. Rotating Bio-contactors

These are plants in which the aerobic treatment takes place on disks arranged on a central shaft which rotates in a bath of domestic wastewater. The shaft remains above the surface of the domestic wastewater, and thus only about 40% of the disk surface is in the domestic wastewater at any one time. The disks vary in diameter between 1 and 3.5 m and rotate at 1 to 5 rpm.

A bio-film is formed on the disk as it rotates and this contains the organisms required to treat the domestic wastewater. As the disk rotates through the domestic wastewater it picks up a film of domestic wastewater and aerates it as it passes through the air. When the bio-film growths too thick the inner layer dies off due to a lack of nutrients and it "sloughs" off. The bio-film has an aerobic layer on the outside, but this becomes anoxic as one goes deeper into the layer due to a lack of dissolved oxygen. The disks are usually made of fibreglass or polyethylene. Some contemporary designs comprise a large cylindrical cage filled with packing as found in bio-towers.

With proper design rotating bio-contactor's can remove COD, ammonia and denitrify. They usually have some kind of fibreglass or plastic cover which is prone to damage from veld fires, so it is wise to surround the plant with a few meters of stone chips to prevent it going up in smoke. It must also be ensured that the cover is well ventilated to ensure that sufficient oxygen is present.

The wetted area is shown as a function of influent and effluent COD in Figure C-1.

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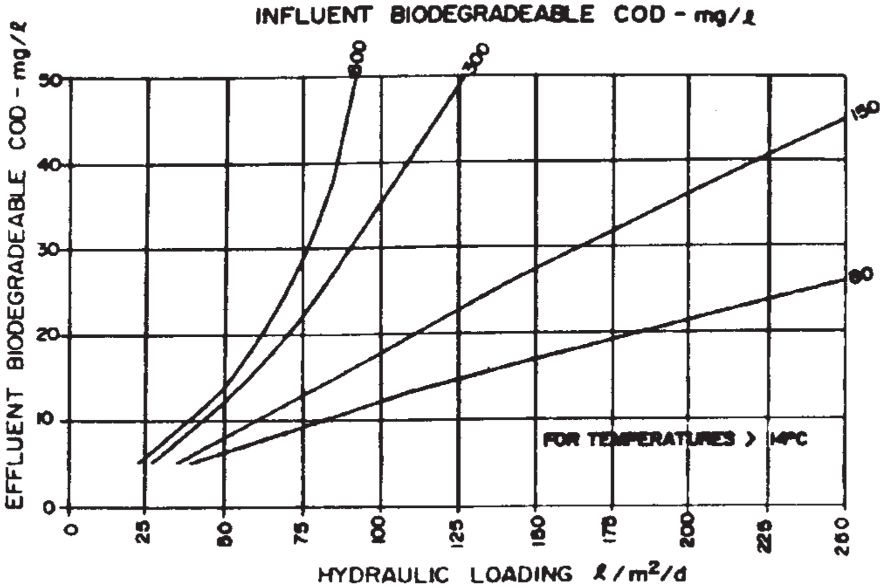

Figure C-1: COD removal curves

Rotating bio-contactors have been shown to perform significantly better when staged in series, i.e. one after the other. If more than 2 stages are used then the wetted surface area can be reduced, as shown in Table C-12.

Table C-12: Correction factor for staging

No. of stages	Correction Factor
3	0.95
4	0.90
>4	0.86

When using staging, the load on the initial stages can become very high, causing odours. Due to this an organic loading of 100g biodegradable COD.m^{-2}.day^{-1} should not be exceeded.

Nitrification will only take place once the COD of the domestic wastewater has been reduced to 25 mg/ℓ. If sufficient alkalinity is present to ensure that the pH does not go lower than 7 (for alkalinity requirement see the relevant discussion under activated sludge) then

Figure C-2 can be used for design to remove ammonia.

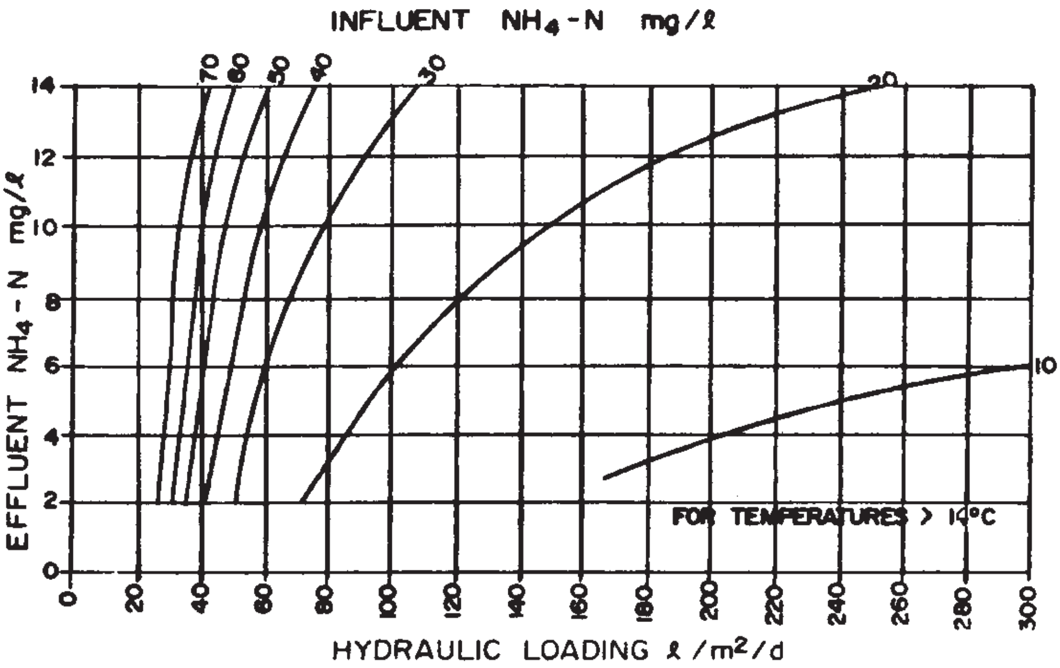

Figure C-2: Nitrification parameters in rotating bio-contactors.

The above design criteria are for a steady state system, i.e. constant flows. In view of the diurnal flow of domestic wastewater the following correction factors should be adopted (C-13).

Table C-13: Correction factor for diurnal

Population	Correction factor
400	1.3
400-1500	1.3-1.1
1500-5000	1.1-1.0

The above design parameters work well in the temperature range of 14-30°C. If colder temperatures are experienced then the correction factors provided in C-14 should be adopted.

Table C-14: Correction factors for low temperature

Temperature (°C)	Correction factor
14	1.00
12	1.05
10	1.15
8	1.30
6	1.40

Rotational speed of the disks is usually between 1 and 5 rpm. Little improvement in treatment has been observed at higher speeds, but the power consumption is somewhat higher.

The settling tank after the bio-contactor is designed using criteria for a humus tank. Where a Dortmund type tank (no mechanical scraper) is used then the design parameters in Table below should be adopted.

Table C-15: Dortmund tank design parameters.

Upward velocity at average dry weather flow	≤ 1 $m.h^{-1}$	Use whichever gives the larger surface area
Average velocity at peak dry weather flow	≤ 1.5 $m.h^{-1}$	

Table C-15 data was formulated assuming that any recycle will take place from the bio-contactor trough back to the septic tank, or from the bottom of the clarifier to the septic tank. If the recycle is from after the settling tank then the recycle must be added to the normal flow. Recycling from the settling tank to the septic tank means that drying beds and conventional desludging of the settling tank is not needed as the solids are removed to the septic tank where they are treated anaerobically. Such a recycle is imperative for de-nitrification.

For a flat-bottomed scraped settling tank the design parameters are provided in Table C-16.

Table C-16: Flat bottomed scraped settling tank parameters.

Retention period of peak dry weather flow plus recirculated flow	≤ 1.5 $m.h^{-1}$	
Upward velocity at average dry weather flow plus recirculated flow	≤ 1 $m.h^{-1}$	Use whichever gives the larger surface area
Average velocity at peak dry weather flow plus recirculated flow	≤ 1.5 $m.h^{-1}$	

Table C-16 recommendations are made assuming that any recycle will take place from the bio-contactor trough back to the septic tank, or from the bottom of the clarifier to the septic tank. If the recycle is from after the settling tank then the recycle must be added to the normal flow. Recycling from the settling tank to the septic tank means that drying beds and conventional desludging of the settling tank is not needed as the solids are removed to the septic tank where they are treated anaerobically. Such a recycle is necessary for denitrification.

The recycle needed for denitrification should be in a ratio of 3:1 to the average dry weather flow. Denitrification halves the alkalinity requirement for nitrification which is desirable. Where necessary, slaked lime should be added to boost the alkalinity.

1.8.4. Submerged bio-contactors

Submerged bio-contactors are normally preceded by a septic tank which provides a screening and degritting role, as well as lowering the COD markedly while producing little biomass due to the nature of anaerobic digestion. The final chamber of the septic tank can also be used as an anoxic reactor for nitrification.

Few design criteria are available in the references commonly used by the authors. These include the "Manual on the design of Small Domestic wastewater Works" and Metcalf and Eddy (Tchobanoglous & Burton, 1979). This paucity of "recognized" design criteria makes it difficult to design these plants on anything other than empirical data. It also makes it difficult to check the design if a plant does not operate satisfactorily.

One approach to solving the lack of published design criteria is to base their design on the design parameters for trickling filters. The rationale for this approach is that they are essentially the same in operation, with the submerged bio-contactor merely being "flooded" and having forced aeration. Certain trickling filters under high COD load are in fact force aerated, usually from the bottom.

There are advantages of using submerged bio-contactors over trickling filters, especially for small scale plants. These are:

- Distribution of the influent is not a problem as it is in trickling filters;
- The media always remains submerged thus eliminating the problem of "drying out" under no flow or low flow conditions; and
- Forced aeration makes it simple to ensure that sufficient oxygen is supplied.

In activated sludge it is well known that for nitrification to occur the dissolved oxygen concentration must be maintained above 2 mg/ℓ. In submerged bio-contactors this alone does not produce nitrification. The efficiency of nitrification depends on the organic loading. When a bio-film forms there are both heterotrophs (COD removers) and nitrifiers (ammonia removers), the nitrifiers tend to attach themselves onto the heterotrophic biofilm, because the heterotrophs grow much faster at higher COD loading rates the nitrifiers become overgrown and nitrification ceases (EPA Technology Fact Sheet 9, USEPA, 2000a).

This is fairly well documented in the literature, and was confirmed with Ekama (1984).

The EPA in their Technical Fact Sheet for trickling filter nitrification (USEPA, 2000b) give a range of organic loadings for plastic media biofilters which will provide 75 to 85% nitrification. These are summarized in Table C-17.

Table C-17: Organic loading rates to ensure nitrification

Loading rate basis	**Loading rate requirement**
Loading rate based on BOD	192-288 $gBOD.m^{-3}.day^{-1}$
Loading rate based on COD	295-443 $gCOD.m^{-3}.day^{-1}$

* based on BOD = 0.65 x COD

A critical assumption which needs to be made here is that the specific area (m^2/m^3) of the packing quoted in the EPA document is the same as that used by the local manufacturers. This is considered to be a valid assumption in that the most commonly used packing in the local SBC's would be ideal for use in trickling filters.

As nitrification is temperature dependant, the more conservative loading rates should be used in cooler climates (minimum monthly average domestic wastewater temperature <18°C) to ensure nitrification.

The pH is also extremely important, and should be maintained above 7. A pH level under 6.5 can result in nitrification failure. Since nitrification causes the pH to drop it must be carefully monitored. Slaked lime is used to maintain a suitable pH and can be added once or twice a week, either by flushing it down the toilet, or into the septic tank, depending on the size of the plant and thus the amount of lime (see the discussion on lime addition in the activated sludge section).

Where the effluent is to be discharged to a water body a recycle from the outlet of the aeration tank (or the base of the settling tank – where fitted) back to the final compartment of the septic tank must be instituted to achieve de-nitrification. This has the added advantage of recovering half the alkalinity lost in nitrification, and hence halving the addition of slaked lime. The recycle ratio should be 1:1 with the average dry weather flow.

Where high TKN domestic wastewaters are to be treated the SBC should be designed with two tanks in series to enhance nitrification in the second tank due to the lower COD loading in the that reactor.

The SBC systems all use submerged aeration, or air entrainment via a pumped venturi, which is essentially the same. The air supply for submerged aeration can have a number of forms, e.g. diaphragm pump, vein pump, or a blower radial or rotating lobe.

Of great importance with submerged aeration systems, both for SBCs and activated sludge, is the depth of submersion of the aerator panel, sock, or venturi return. The deeper the reactor the greater the depth of liquid through which the oxygen bubbles pass and the lower the amount of air required. Conversely, the shallower the liquid the shorter the bubble path, and the more the air required.

It is thus advantageous in these systems to have reactors with the maximum possible depth.

When bio-film breaks off the packing material there are one of two routes it can go. The one route is to settle to the base of the reactor where it will continue to digest both aerobically (and thus increasing oxygen demand) and, depending on the reactor design, anaerobically (where a thicker layer forms the underneath sludge will turn anaerobic). The other route is that the solids can leave the reactor with the effluent. Which particular route a particle takes will depend on its size: large particles will tend to settle, while smaller particles will tend to be lifted upwards by the bubble stream and leave the reactor in the final effluent.

The “stray” solids give rise to two important design elements:

- An outlet should be fitted to the base of the aerobic reactor to allow the accumulated solids to be drained out from time to time, and returned to the septic tank inlet; and
- The treated solids exiting the aerobic reactor need to pass through a settling tank in the case of larger flows, while that exiting smaller units should pass through a sieve type screen which will retain the solids (1-2 mm mesh). This sieve will need to be cleaned on a weekly basis. If the sieve is housed in the chlorine contact tank it should not make contact with the chlorinated effluent in the tank. Any solids in a chlorine contact tank will exert a high chlorine demand which will result in no chlorine being available for disinfection (an analogous situation occurs when the leaf catcher in a swimming pool is left full of leaves – the organics absorb all the chlorine, and the algae thrive, turning the pool green much to the owners astonishment as he adds more and more chlorine).