PERFORMANCE OF WASTE STABILISATION PONDS IN THE EASTERN CAPE PROVINCE

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ABSTRACT

Water is a scarce natural resource, which requires to be treated with much care and importance. It is a finite resource and should be used sparingly. The process of treating domestic wastewater varies from ponds to the more advanced system, namely the activated sludge system. The main purpose of wastewater treatment is the reduction of pathogenic contamination, coliform bacteria, suspended solids, oxygen demand, and nutrient enrichment. The application or use of stabilisation ponds, as a part of the wastewater treatment process, depends on, among other factors, the influent loading and climate conditions.

Waste Stabilisation Ponds (WSPs) are used to biologically treat domestic wastewater or industrial wastewater. The present study focuses on the treatment of domestic wastewater by using the WSPs in the absence of mechanical and electrical equipments.

Different countries use different methods of pond design or WSP sizing and different parameters to ensure that the effluent discharge guidelines of the Department of Water and Sanitation (DWS) and World Health Organisation (WHO) are met.

There are insufficient literature studies focusing on the design models and water quality data that can be used for sizing the WSPs in South Africa. There is a requirement for a study that can compare the existing WSP design models in different countries and check their suitability for South Africa, particularly their applicability to provinces with respect to climate and domestic wastewater quality. The comparison between the WSP design models will assist the process designers in the early stages of projects, particularly in the feasibility study stages (Scenario 1).

The objective of the present study is to perform a comprehensive review of the use of WSPs in domestic wastewater treatment, their design and operating requirements for optimal performance, and the existing mathematical models used to virtually replicate the WSP treatment processes. Also considered is the development of a simplified model to demonstrate its application as a tool for the effective design of WSPs, including a case study of a WSP in the Eastern Cape (EC).

The present study investigates the factors that can lead to poor effluent quality other than those caused by operation and maintenance (O&M). This includes conceiving quick process sizing models to investigate the performance of the existing WSPs, without the use of the existing (as-built) drawings for Scenario 1.

The information, including the pond mechanism of operation and geometry (depth, bottom and top widths, side slope, and others), water quality, and characteristics are used for pond sizing and performance study. They are primarily based on the research and literature studies by recognised European, African, British, and North American scientists.

Different models are used to compute the design or sizing of the WSP units (anaerobic, facultative, and maturation ponds), from which a suitable model for the EC WSP is chosen at the feasibility stage (Scenario 1) and subsequently used at the conceptual stage (Scenario 2). The water quality assessments are done by independent laboratories (IDZ and Monitor laboratories). The water test results are used for modelling the WSP for Scenario 2, whereby the design inputs or parameters used for Scenario 1 are the typical domestic wastewater characteristics obtained from the literature. In the present study, only the bacteria reduction Faecal Coliform (FC) and Biochemical Oxygen Demand (BOD) loading are considered for the WSP process design. The ratio COD:BOD of 2:1, in domestic wastewater, is used to calculate the BOD of the WSP influent (van Niekerk *et al.*, 2009).

During the Scenario 1 stage, the information, namely the water quantity (flow rate to and out of the pond system), water quality results, and pond depths are selected based on data found in the literature. This information, made for Scenario 1, is confirmed and refined during the analysis of Scenario 2. Hence, the information is used as a design input to compute the WSP sizing to ensure that the requirements of the DWA and/or WHO are met.

The two types of WSP configuration, namely the A-F-M and F-M systems, are considered during the Scenario 1 stage, and the F-M system is only considered during the Scenario 2 stage. The A-F-M system is the WSP containing an Anaerobic Pond (AP), Facultative Pond (FP), and Maturation Pond (MP), whereas the F-M system is the A-F-M system without AP. The summary of the models used for the A-F-M and F-M systems for Scenario 1 are presented in Table 2.2.

Table 1: The summary of the models used for the A-F-M and F-M systems for Scenario 1

		A-F-M System		
Pond Unit	**Model 1**	**Model 2**	**Model 3**	**Model 4**
AP (A-F-M system)	Detention time of 2 days	volumetric BOD loading of 250 g BOD/m^3.d	volumetric BOD loading of 140 g BOD/m^3.d	
Secondary Facultative Pond (SFP) – (A-F-M system)	Mara Empirical Equation	First-order kinetics	McGarry & Prescod Empirical procedure	Marais design sequence
MP (A-F-M system)	The retention time of about 3 days	The retention time of 10 days to all the MPs	The retention time of 7 days to all the MPs	
		F-M System		
Primary Facultative Pond (PFP) – (F-M system)	Mara Empirical Equation	First-order kinetics	McGarry & Prescod Empirical procedure	Marais design sequence
MP (F-M system)	The retention time of about 3 days to all MPs	The retention time of 10 days to all the MPs	The retention time of 7 days to all the MPs	

The analyses and results of Scenario 1, for both the A-F-M and F-M systems, are computed and compared. The comparison of the water quality at the inlet and outlet of each pond unit, namely AP, FP, and MP, is done. The best model from the Scenario 1 stage is used during the analysis of Scenario 2 stage to ascertain the pond sizes and effluent characteristics for each pond unit. The designs for Scenario 2 are done based on the existing pond configuration, i.e., the WSP without AP (F-M System).

During Scenario 2, the Excel spreadsheet is used to determine the minimum, average, median, and maximum concentrations of the wastewater parameters that are based on field measurements at the influent and effluent points of the WSP. The average influent COD concentration is used for modelling the PFP size during the Scenario 2 stage. The summary of the models used for the F-M system for Scenario 2 is presented in Table 2.2.

Table 2: The summary of the models used for the F-M system for Scenario 2

Pond Unit	Model used	Comments
PFP	Marais design sequence	The calculations on this model do not consider the surface loading (λ_s) and/or arial loading rate (λ_d).
MP	Retention time of 7 days for all the MPs	The design of two or more maturation ponds in cases where the effluent Faecal Coliforms (FC) from the primary facultative meets the DWS general limit.

The as-built information is to be kept safe in order to be used for modelling the performance of the WSPs, should the possibility of an upgrade of the existing WSPs be required.
This can assist in determining the poor performance at an early stage before the WSP is overloaded. The operator of the WSP should pay attention to the performance of the pond rather than assess compliance with the DWS discharge guidelines.

The literature has indicated that different countries, particularly due to different climatic zones, use different wastewater parameters and methods of pond design. The Marais design sequence is found to be the simplest model which can be used for both sizing the pond units and checking the performance of the WSPs.

The pond sizing performed during the Scenario 1 stage indicated no major changes in pond sizing required during the Scenario 2 stage, more especially when the influent domestic characteristics from the literature have been used.

In present study, the modelling of a WSP in the Eastern Cape region of South Africa was considered as a case study. It was noted that the maximum temperature should not be used when sizing the FPs, as this can result in a smaller pond size, and subsequently in an effluent quality not complying with the required discharge guidelines, particularly during the winter season. Also, this would influence the performance of the subsequent pond units.

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SYMBOLS AND ACRONYMS

SYMBOLS

A_f	Facultative pond mid depth area, m^2
A_{sfp}	SFP mid depth area, m^2
A_{pmp}	PMP mid depth area, m^2
A_{smp}	SMP mid depth area, m^2
a	Factor equal to $(1 + 4 \text{ kt } D)^2$.
B_v	Volumetric loading, g BOD/(m^3 day)
b_p	Breadth of the pond, m
c	Capita, No.
C_e	Effluent BOD_5 concentration, mg/l
C_0	Influent BOD_5 concentration, mg/l
d	Total depth, feet
d	Day
d	Depth of the pond, m
D	Dimensionless dispersion number = H / vL = Ht / L^2
D_{pmp}	Depth of the PMP, m
e	Base of natural logarithms = 2.718282
f	Algal toxicity factor
f '	Sulfide oxygen demand
g	Gram
H	Axial dispersion coefficient, area per unit time
K	Monomolecular or velocity constant measured in log S-day units
K	Degradation rate constants, day^{-1}
k	First-order reaction rate, $days^{-1}$
K_e	Equivalence rate constant, day^{-1}
K_b	First order rate constant for FC removal, day^{-1}
K	Decay constant dependent on temperature, day^{-1}
K_T	Decay constant at T ^{0}C
K_{20}	Decay constant at 20 ^{0}C
k_{1T}	First order rate constant for facultative pond, day^{-1}
$k_{1(20)}$	Facultative pond first order constant, day^{-}

k_p	Plug flow first-order reaction rate, days^{-1}
kl	Kilolitre
km	Kilometre
L_i	Untreated wastewater strength, mg BOD/l
L_e	Unfiltered effluent strength, mg BOD/l
L_a	Ultimate influent BOD or COD, mg/l
L	Length of the travel path of a typical particle
l	Litre
l_p	Length of the pond, m
l/cap/d	Litres per capita per day
mlS/m	Milli-Siemens per meter
mg	Milligram
ml	Millilitre
m	Metre
m^3	Cubic Metre
m^3/c/d	Cubic metre per capita per day
m^2	Square metre
N	Concentration of faecal organisms per unit volume
N	Number of bacteria per millilitre in the pond
N	Period
N_e	Number of E. coli/100ml of effluent
N_i	Number of E. coli/100ml of influent
N_0	Number of bacteria per millilitre in the pond influent
P	Contribution population
P	Total BOD concentration in the pond, mg BOD/l
P_o	Influent BOD concentration, mg BOD/l
P_0	Present population (base population)
P_N	Population of Nth year
pH	Potential of hydrogen
Q	Average flow, m^3/d
Q	Flow per capita per day, m^3/c/d
Q	Influent flow rate, l/d
Q_1	Influent flow to the pond at any time, t
Q_2	Effluent flow from the pond at any time, t
Q_{ADWF}	Average dry weather flow, kl/day
Q_{PDWF}	Peak dry weather flow, kl/day
Q_{PWWF}	Peak wet weather flow, kl/day
R	Retention time (V/Q), days
R_1	Inflow retention time, days
R_2	Outflow retention time, days

r	Rate of population growth
S	Years between desludging (maximum of 10 years allowed), years
S	Concentration in the pond and in the effluent from the pond at time, t.
S_0	Concentration in the influent to the pond, t.
t	Time (measure from any arbitrary instant), days
t	Retention time, day
t	hydraulic retention time, days
T	Mean air temperature in the design month, ^0C
T	Pond design temperature, ^0C.
V	Volume of the pond, m^3
V	Fluid velocity, length per unit time
V_{ap}	AP volume, m^3
V_f	FP volume, m^3
V_{pmp}	PMP volume, m^3
V_{smp}	SMP volume, m^3
V_{ST}	Septic Tank volume, m^3
V_{sfp}	SFP volume, m^3
λ	Maximum BOD loading, kg/ ha/ day
λ_d	Design loading, kg/ ha/ day
λ_s	Surface loading, kg/ ha/ day
λ_s	Areal loading rate, kg BOD_5 / (ha day)
λ_v	Volumetric loading, g/ (m^3 day)
θ	Constant
θ	Temperature correction coefficient =1.085
θ_f	Retention time, day
Ω	Ohm
%	Percentage
^0C	degree Celsius
CH_3COOH	Acetic acid
$C_2H_{12}O_6$	Acetic acid tetrahydrate
NH_3	Ammonia
NH_4^+	Ammonium
NH_3 -N	Ammonia-Nitrogen
NH_4^+ -N	Ammonium-Nitrogen
CO_2	Carbon Dioxide
Cr^{+3}	Chromic ion
H_2	Hydrogen
CH_4	Methane
NO_3^-	Nitrate
N	Nitrogen
O_2	Oxygen

P	Phosphorus
K	Potassium
S	Sulphur
SO_4	Sulphate

ACRONYMS

ASM	Activated Sludge Model
AeP	Aerobic Pond
ADM	Activated Digestion Model
AP	Anaerobic Pond
AS	Activated Sludge
BH	Borehole
BOD	Biochemical Oxygen Demand
CWM	Constructed Wetland Model
CAS	Conventional Activated Sludge
COD	Chemical Oxygen demand
CBD	Central Business Centre
CFD	Computational Fluid Dynamics
DO	Dissolved Oxygen
DoH	Department of Housing
DWA	Department of Water Affairs, now called DWS
DWS	Department of Water and Sanitation
DESA	Department of Economic and Social Affairs
EC	Eastern Cape
E. coli	Escherichia coli
FCC	Fort Cox College
FP	Facultative Pond
FC	Faecal coliforms
F. strep	faecal streptococci
LA	Local Authorities
LS	Laboratory Scale
Lit	Literature
MLS	Mixed liquor suspended solids
MP	Maturation Pond
Max	Maximum
Min	Minimum

O&M	Operation and Maintenance
PDWF	Peak Dry Weather Flow
PWWF	Peak Wet Weather Flow
PFP	Primary Facultative Pond
p.e.	Population equivalent or unit per capita loading
pH	Potential of hydrogen or power of hydrogen
PMP	Primary Maturation Pond
RWQM	River Water Quality Model
sect.	Section
SAR	Sodium Adsorption Ratio
SA	South Africa
SAI	South African Infantry
SANAS	South African National Accreditation System
SAWS	South African Weather Services
SFP	Secondary Facultative Pond
SS	Suspended Solids
TARDI	Tsolo Agricultural and Rural Development Institute
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorus
USA	United States of America
USEPA	United States Environmental Protection Agency
WHO	World Health Organization
WRC	Water Research Comission
WSA	Water Services Authorities
WSP	Waste Stabilisation Pond
WWTP	Waste Water Treatment Plant

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND OF RESEARCH

Water is a scarce natural resource, which requires to be treated with much care and importance. It is a finite resource and should be used sparingly. This also includes wastewater, particularly domestic wastewater, which is reusable. There is a saying by the Department of Water and Sanitation (DWS) stating: “Water is life. Sanitation is dignity”. This is in agreement with the concept of treating water with care to improve our longevity. The process systems for treating domestic wastewater vary in complexity from ponds to more advanced systems, namely the activated sludge systems.

The main purpose of wastewater treatment is to bring the wastewater to an acceptable quality level for it to be discharged into the surrounding environment, particularly in receiving waters (e.g. rivers, lakes, and others) (Edokpayi, *et al.*,2021). The wastewater treatment process generally involves the reduction of pathogenic contamination, coliform bacteria, suspended solids, oxygen demand, and nutrient enrichment (Ukpong, 2012 and Nozaic *et al.*, 2009). Pescod and Mara (1988) added that the application or use of the stabilisation ponds, as wastewater treatment processes, depends on the investments in a collection system, which are the location of system either in rural areas or developing countries, availability of land, local habits and social practice including values. In addition, Nozaic *et al.* (2009) stated that the choice of the suitable wastewater treatment process is influenced by the proposed plant size, type of waste to be treated, and degree and consistency of treatment required.

Waste Stabilisation Ponds (WSPs) are used to biologically treat domestic wastewater or industrial wastewater. WSPs have been used for the treatment of wastewater for over 3 000 years (USEPA, 2011), and the first pond system was constructed in the United States of America (USA) in San Antonio in 1901 (Gloyna, 1971). Today, there are large pond systems found in New Zealand, Australia, and Africa, which are used to treat a variety of wastewaters, from domestic to complex industrial effluents (Mara, 2003).

The research studies on ponds in Southern Africa mostly date from the period between 1970–2016, and therefore there is a research gap, on WSP modelling, from 2016 to this day; the design theories of WSP modelling have to be researched further (Ho, et al., 2017). The verification of the models to suit real life must be done to ensure that the literature represents the real life situation. The design models are based on the literature performed at small-scale laboratory levels. The objective of this study is to carry out the review of the WSP design models to be used for sizing the WSPs in the Eastern Cape (EC) region. This will provide confidence in the type of design model that suits the conditions of the EC region.

1.2 QUESTIONS AND HYPOTHESIS OF RESEARCH

South Africa lacks a suitable design model for sizing the WSPs. Design practitioners are using different models for pond sizing or to ascertain the performance of the existing WSPs. Detailed studies were carried by Marais in Southern Africa, but they primarily focused on Botswana. There is insufficient study and literature dealing with the design models that can be used for sizing the WSPs in South Africa. This includes the lack of water quality test data. A study is required to compare the design models from other countries and check their suitability for South Africa. The comparison between the design models for ponds will assist the process designers in the early stages of projects, particularly during the feasibility study stages (Scenario 1). The model must be easy to use by the operators to check the performance of the pond during the design and operation.

The validation of the WSP models and water quality data for ponds, at certain periods or during the lifetime of the WSP, must be done, including the performance checking of the existing ponds. The validation is done by using different types of WSP models. This is achieved by using the same input data, including, but not limited to, temperature, Chemical Oxygen demand (COD), Biochemical Oxygen demand (BOD), Number of Escherichia coli (E. coli) and Faecal coliforms (FC), and compare the calculated pond sizes and predicted system performance variables (e.g. effluent quality). This will also assist the operators in determining the time period until the pond is overloaded and subsequent desludging required.

1.3 OBJECTIVES OF RESEARCH

The main objective of the present study is to:

- Perform a comprehensive review of the use of ponds, i.e. WSPs, for domestic wastewater treatment, their design and operation requirements for optimal performance, and the currently developed mathematical models used to virtually replicate the WSP treatment processes.
- Review a simplified model to showcase its application as a tool for the effective design of ponds, with a case study performed on a WSP in the EC Province of South Africa.
- Provide recommendations for a protocol used in checking WSP performance.

1.4 SCOPE AND LIMITATIONS OF RESEARCH

The scope of the present study project is to investigate the impact of wastewater loading and wastewater quality with respect to the performance of the WSP. Also considered is an investigation of how these factors affect the design and performance of the WSP. A major portion of the present study includes the rigorous review of WSPs and their application for domestic wastewater treatment. The focus of the present study is the investigation of the reasons for poor effluent quality, excluding the Operation and Maintenance (O&M) issues.

For the requirements of the intended Master Degree, it was decided to exclude extensive experimental work, hence, no laboratory or field testing was done. The present study is primarily a desktop study, which involved the utilisation of historical data, concepts obtained from literature, and analytical design calculations, using mass balanced steady state equations, where necessary. The method of analysis, including the models used, is discussed in Section 3.6.

The present study does not include complex dynamic simulations of WSPs, nor the use of computational fluid dynamics (CFD) in the model development or process application. The experimental data obtained by independent laboratories were subject to descriptive statistical analysis, and the graphs were obtained using Microsoft Excel 2013.

Seven full scale pond systems in the Eastern Cape Province were initially chosen to be the study areas for the present work. Since the geometry of the ponds was unknown, the writer refined the selection to Tsolo Agricultural and Rural Development Institute (TARDI), as the study area, for which the pond geometry was known. The assumptions considered during the design stage are discussed in Section 3.7.

The following are the limitations of the present study:

- The ponds at TARDI, the study area, were upgraded and commissioned in September 2019, and there were few water tests records available from the date of commissioning of the pond to the time of writing the study.
- The water quality results at the inlet works of the WSP, including the characterisation and influent flows, were not obtained due to the insufficient data for comprehensive characterisation of the wastes. The data available was reconciled with the observed trends and parameters from literature to determine the comprehensive characteristics of the influent. This was deemed sufficient for comparative study purposes, and subsequently the same influent characteristics were used in the comparative design studies.
- The water quality tests were not conducted at the influent and/or effluent of each pond unit (septic tank, facultative and maturation ponds) due to the financial constraints of the present study.
- The specific climatic conditions at the study area were not measured and were unknown.
- The geometry of the existing ponds was unknown during the feasibility study stage (Scenario 1), including the test records for assessing or evaluating the performance of WSP. These parameters are used for the pond sizing of Scenario 1 conditions, and they are refined and confirmed during the pond sizing and analysis of Scenario 2. The availability of the pond measurements (length, width, and depth) and water test records for Scenario 1 assist in comparing the on-site WSP water quality test results with the calculated theoretical water quality from the mathematical models.

1.5 STRUCTURE OF THESIS

Chapter 1 outlines the background and objectives of the present study, and its scope and limitations. **Chapter 2** presents a review of the literature, including the applicability and suitability of WSPs, domestic wastewater characterization, brief review of anaerobic and aerobic biological processes, and types and mechanisms of operation of WSPs. This chapter also includes comparisons of the wastewater processes, which are generally used for treating the domestic wastewater; and considerations of design theories for WSPs, including the factors that affect the performance of WSPs as discovered by Marais, Mara, and other authors. Furthermore, it makes references to various WSP models, including the South African effluent guidelines, as specified by the DWS.

Chapter 3 provides a comprehensive description of the materials and methods used to collect data, including the parameters required to carry out the study, overview of the case study, and data processing and analyses for design modelling purposes. Also included are the problems encountered during the data collection, preparation and analyses, and design criteria and assumptions, including the approach followed in the WSP study. **Chapter 4** analyses the data collected in TARDI and evaluates the different mathematical models to compare the effluent water quality, including the required WSP. The best model is selected, thereof, to determine the design and performance of ponds. The chapter also discusses the applicable design model or approach to the WSP.

Chapter 5 presents the conclusions on the applicability of the pond design.

Figure 1.1 gives the structure of the present study.

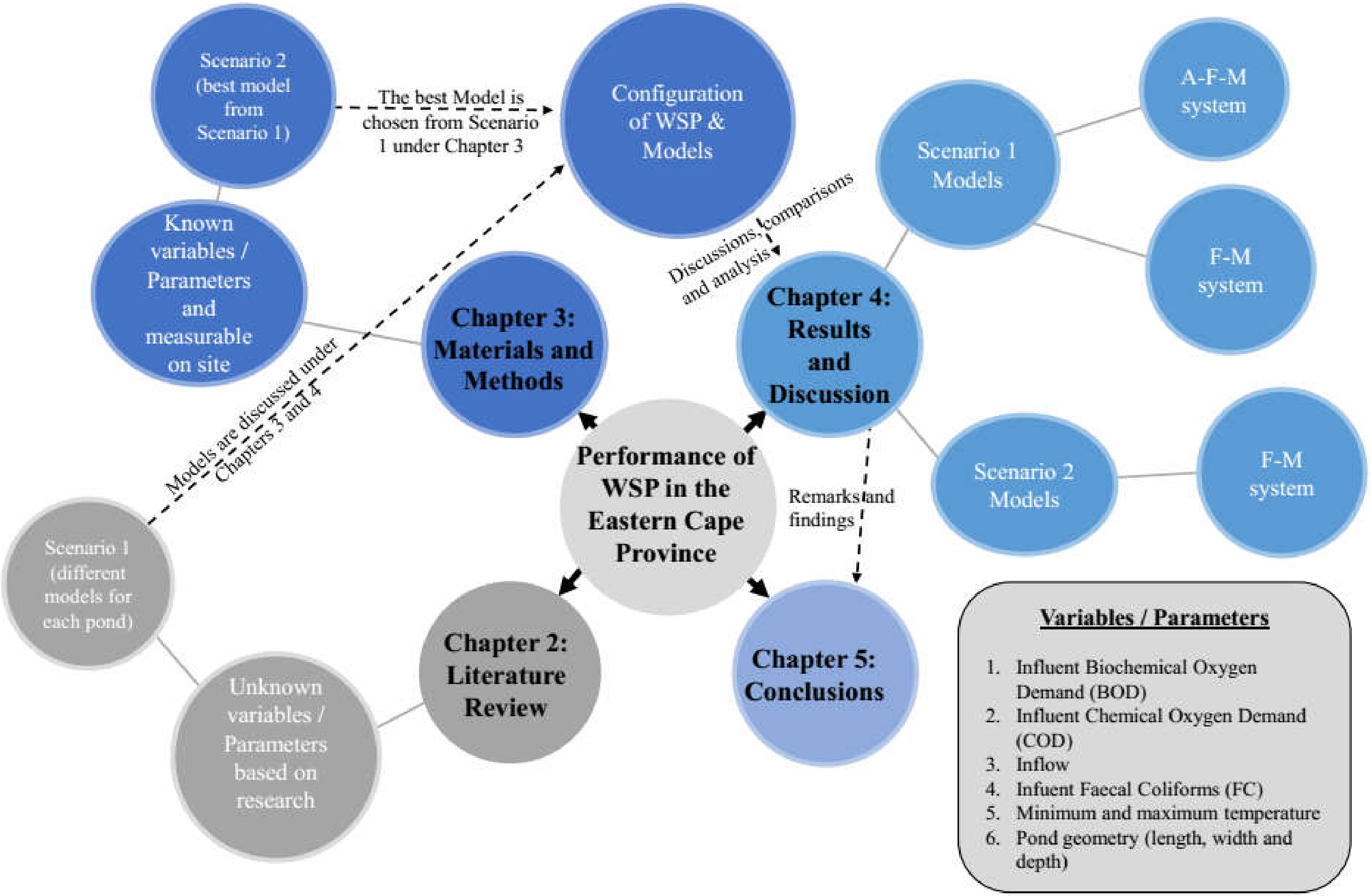


Figure 1.1: The structure of the present study.

CHAPTER 2: LITERATURE REVIEW

2.1 INTRODUCTION

Waste Stabilisation Ponds (WSPs) are earth-made dams or ponds which are constructed with the aim of treating wastewater, particularly domestic wastewater, to ensure that the treated effluent meets the regulatory discharge guidelines before it is released into the surrounding environment. The regulatory effluent discharge guidelines vary from one country to another. This literature review gives a brief overview of the aforementioned guidelines below.

This chapter will review and/or discuss the following:

(i) the applicability of the WSPs with respect to the geographic and climate zones;
(ii) the suitability of the WSP systems with respect to the sewage quality and quantity to be treated;
(iii) the characteristics of domestic wastewater for both treated effluent and untreated influent;
(iv) the biological processes involved during treatment of domestic wastewater, including the types of units used for WSP;
(v) the comparison between the WSP units and systems;
(vi) the characteristics and mechanisms of operation of ponds, and the design concepts of WSPs, including an overview of WSP modellings; and
(vii) the effluent discharge guidelines.

The literature review will conclude by giving the method used in population forecasting to determine the design flow of WSPs and a conclusive summary of the parameters used for sizing the WSPs, which are based on the literature review models.

2.2 APPLICABILITY OF THE WSP SYSTEM WITH RESPECT TO THE GEOGRAPHIC AND CLIMATE ZONES

Marais (1966) and Arthur (1983) stated that the environmental conditions in tropical and subtropical regions are favourable for attaining a high level of pond performance.

The map in Figure 2.1 below indicates the geographic and climate zones. Based on the statement by Marais (1966) and Arthur (1983), the WSPs are suitable for the Eastern Cape Province of South Africa given that the climate in this province is a subtropical climate.

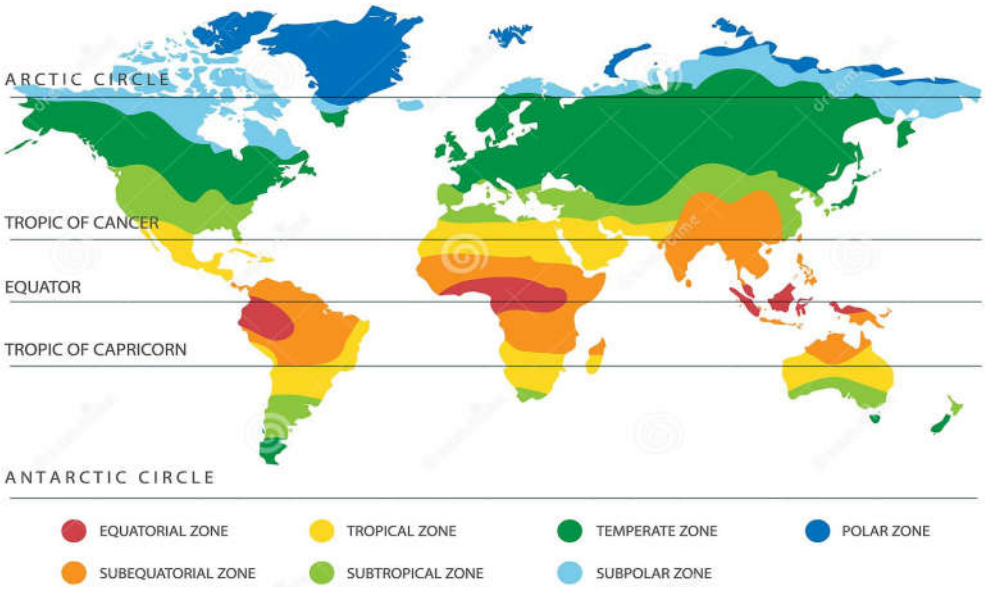


Figure 2.1: The geographic and climate zones as indicated on the map, extract from "World climate zones…" (n.d.).

2.3 APPLICABILITY AND SUITABILITY OF THE WSP SYSTEMS

Waste Stabilisation Ponds (WSPs) are found to be the most cost-effective solutions for treating wastewater in terms of capital, operating, and maintenance costs (Nameche and Vasel, 1998; van Niekerk *et al.*, 2009; Ukpong, 2012, and Edokpayi, *et al.*,2021). They have been known to be a cheap and effective systems to treat wastewater in situations where the cost of land is not a factor (Ukpong, 2012). Added to the economic benefits, the WSPs have distinct advantages in terms of the simplicity of construction, operation, and maintenance over conventional wastewater treatment processes (Pescod *et al.*, 1988).

Mara (2009) identified the two suitable situations for the WSP systems, which are for small and large communities, respectively. The author suggested that the WSPs are a superior method for treating wastewater given that they make use of natural resources, whereas the electrified wastewater treatment systems, e.g. the Activated Sludge (AS) systems, do not. Butler et al. (2015) added that the oxidation pond process is a natural process due to the use of microorganisms (bacteria and algae) during the treatment process. The difference between the Conventional Activated Sludge (CAS) systems and WSPs are summarised in Table 2.1 below (von Sperling, 2007a and Genderen, 1995).

Table 2.1: The main biological wastewater treatment systems - comparison between the CAS systems and WSPs (von Sperling, 2007a and Genderen, 1995)

Item No.	WSPs	CAS systems
1	Good pathogen removal efficiencies.	Poor pathogen removal efficiencies.
2	The wastewater remains in the ponds for many days.	High capital, Operation and Maintance costs.
3	Low level of operating and maintenance skills	High level of operating and maintenance skills
4	Large land area is required.	Small land area is required.
5	The photosynthesis process and algae are used for supplying oxygen to the water body	The mechanical aerators and diffused air are used for supplying oxygen to the water body.
6	Tolerance to deal with both hydraulic and organic shock loads.	

Mara (2009) recommended the use of the wastewater storage and treatment reservoirs for small and large communities to reuse the treated water from the WSP, this particularly as water scarcity increases worldwide.

The performance of the WSP depends on climatic conditions (sunlight, temperature, wind, and rain), hydraulic loading, and influent wastewater quality. Tharavathy *et al.* (2014) and Edokpayi *et al.* (2021) indicated that when wastewater is overloaded with pollutants, it can cause shock loads, which can degrade the effluent quality of the WSP. This is also in agreement with Butler *et al.* (2015) who indicated that the four major mass transport mechanisms acting in the oxidation ponds are diffusion, advection, gravity, and interception, depending on the type of wastewater being treated. Arthur (1983) indicated the major treatment processes in a WSP to be:

- Absorption of both organic and hydraulic shock loadings;
- Settling of solids to form the benthal sludge layer; and

- Treatment of the organic wastes by both the aerobic bacterial oxidation and anaerobic digestion.

On the contrary, Arthur (1983) highlighted that the pond system responds well even when overloaded beyond its theoretical design loads. This can be achieved by the minor modifications to the plant, to include baffles, which can lead to discharging an effluent of high quality (Nameche and Vasel, 1998, Mara, 2009 and Sah *et al*., 2011). The characteristics and mechanism of operation of the WSPs are briefly discussed in detail in Section 2.8.

Mara (2009) expressed his views on the suitability of the WSP system, including the components of a WSP for small and large communities as follows:

a) For small communities of up to approximately 500 p.e. (Population equivalent or unit per capita loading): the WSP should consist of two septic tanks followed by a baffled secondary facultative pond and a rock filter, with the rock filter being aerated if ammonia- nitrogen (NH_3 -N) removal is required.
b) For large communities of up to approximately 2000 p.e.: the WSP should consist of Imhoff tanks instead of the septic tank.

The WSP, as mentioned by Edokpayi, *et al*., (2021), Nomache *et.al.* (1998), and Pescod *et al.* (1988), does not require highly skilled labour and is easy to operate and maintain. Most of the Local Authorities (LA) and institutions (colleges, hospitals, clinics, and others), particularly those that are located in remote areas in South Africa (SA), prefer to treat their domestic wastewater by using the WSP. In developing rural areas of South Africa, the choice of WSPs can be influenced by the following:

- Scarcity of skilled operators and complexity of operation of the WSP systems, which are generally known to be robust;
- Issues regarding the availability of land; and
- Low operating and maintenance costs compared with Activated Sludge (AS) systems.

2.4 WASTEWATER CHARACTERISTICS FOR WSP SYSTEMS

Wastewater contains various organic and inorganic components with different physical and biochemical (biological and chemical) characteristics, as shown in Table 2.2. In addition, Henze *et al.* (1997), von Sperling (2007a), and Henze *et al*. (2008) indicated that the domestic wastewater consists of microorganisms, biodegradable organic materials, non-biodegradable organic materials, nutrients, metals, and inorganic materials, and it can have odour, taste, thermal effects (variability of temperature) and even radioactivity.

It is important to understand the origin of the wastewater being treated, which can be from domestic wastes (which, according to Arthur [1983], comprise faeces, urine, and sullage, and consist of 99% of water and 0.1% of solids), wastes from institutions (e.g. hospitals), industrial wastes (e.g. from tanneries), and others. Various types of wastewaters have different compositions that essentially depend on the sources of pollutants. Furthermore, the design and sizing of the treatment process are not only affected by the concentrations of the pollutants, but also by the amounts of wastewater being treated (i.e. the daily pollutant loads).

Table 2.2. presents the different domestic wastewater characteristics related to the physical, chemical, and biological properties of domestic wastewater.

Microorganisms are important in wastewater because they are the primary agents of biological treatment (Spellman & Drinan, 2014). The microorganisms indicated in Table 2.2 are found in wastewater, and they include, but are not limited to, bacteria, protozoans, viruses, and algae (von Sperling, 2007a). The typical concentrations of microorganisms, as reported for untreated domestic wastewater in developing countries, are presented in Table 2.2.

Henze *et al.* (2008) indicated that it is necessary to characterize the wastewater physically and biologically for the AS system. The approach of wastewater characterisation is vital when modelling the WSP due to its performance, which is impacted by the wastewater constituents. The typical concentration values of pollutants found in untreated domestic wastewater in developing countries, sourced from various authors, are presented in Table 2.2.

Table 2.2: The organisms and characteristics of domestic wastewater (adopted from von Sperling, 2007a, Henze et al., 2008, van Niekerk *et al.* , 2009 and Nozaic *et al*., 2009)

Physical Characteristics

Temperature * ++
a) It influences the microbial activity, solubility of gases, and viscosity of the liquid.
b) It is marginally higher than drinking water.
c) It varies according to the seasons of the year, and it is more stable than the air temperature.

Colour *
a) It looks slightly grey for fresh wastewater and dark grey or black for septic wastewater.

Odour *
a) It has characteristic odours, if industrial wastewater.
b) It has an oily odour or is relatively unpleasant for fresh wastewater.
c) It has a foul odour (unpleasant) due to hydrogen sulphide gas and other decomposition by-products for septic wastewater.

Turbidity *
a) It is a fresher or more concentrated wastewater with a generally greater turbidity.
b) It is caused by great variety of suspended solids.

Chemical Characteristics

Total Solids (TS) * ++
TS are subdivided into:
a) Suspended organic and inorganic solids;
b) Dissolved organic and inorganic solids; and
c) Settleable solids.

Organic matter *
It is the heterogeneous mixture of various organic compounds.
The main components are protein, carbohydrates and lipids

Total Nitrogen (TN) *
It includes organic N, ammonia, nitrite, and nitrate. This is briefly discussed under anaerobic and aerobic biological processes in Section 2.5.

Total phosphorus *
It exists in organic and inorganic forms.

Biological Characteristics

The main parameters defining the quality of wastewater are solids, organic matter, Nitrogen (N), Phosphorus (P), and indicators of faecal contamination.

Solids * ++
The solids are classified by size and state, chemical characteristics, and settleability, namely suspended (non-settleable), dissolved (settleable), volatile (organic), fixed (inorganic), suspended settleable, and suspended non-settleable solids.

	Total Solids *	Suspended Solids (SS)		Fixed (inorganic) Suspended Solids *	Volatile Suspended Solids *	Dissolved Solids *	Fixed (inorganic) Dissolved Solids *	Volatile Dissolved Solids *	Settleable Solids (SetS)	
Range (mg/l)	700 – 1 350	200 – 450 *	200 – 350 **	400 – 100	165 – 350	500 – 900	300 – 550	200 – 350	10 – 20 *	8 – 10 **
Typical (mg/l)	1 100	350 *		80	320	700	400	300	15 *	

Carbonaceous organic matter *
It is classified in terms of structure and size, and in terms of biodegradability, namely suspended (particulate), dissolved (soluble), inert (unbiodegradable), and biodegradable organic matter. Direct and indirect methods are used to quantify the organic matter.
a) Indirect methods (measurement of oxygen consumption) – Biochemical Oxygen Demand (BOD), Ultimate Biochemical Oxygen Demand (BODu), and Chemical Oxygen Demand (COD).
b) Direct methods (measurement of organic carbon) – Total Organic Carbon (TOC).

	BOD_5		COD		BOD ultimate *
Range (mg/l)	250 – 400 *	250 – 350 +	450 – 800 *	500 – 700 **	350 – 600
Typical (mg/l)	300 *		600 *		450

Nitrogen *
The predominant forms of nitrogen in untreated domestic wastewater are organic nitrogen and ammonia. These two forms are determined by the Kjeldahl method, at laboratory scale, to yield the Total Kjeldahl Nitrogen (TKN).
TKN is the sum of ammonia and organic nitrogen, whereas the Total Nitrogen (TN) is the sum of TKN, Nitrogen Dioxide or nitrite (NO_2^-) and Nitrate (NO_3).
Thus the TN includes the organic nitrogen, ammonia, nitrite, and nitrate. It is found to be the essential nutrient for growth of the microorganisms in biological wastewater treatment.

	Total Nitrogen *	Organic N *	Ammonia			Nitrite *	Nitrate *	Total Kjeldahl Nitrogen	
Range (mg/l)	35 – 60	15 – 25	20 – 35 *	40 – 60 +++	40 – 50 +	0	0 – 2	35 –85 +++	60 – 85 +
Typical (mg/l)	45	20	25 *			0	0		

Phosphorus *
The Total phosphorus exists in organic and inorganic forms.
It is the essential nutrient in biological wastewater treatment.

	Phosphorus		Organic Phosphorus *	Inorganic Phosphorus
Range (mg/l)	4 – 15 *	10 – 13 **	1 – 6	3 – 9
Typical (mg/l)	7 *		2	5

pH (Potential of Hydrogen) *
It is the indicator of the acidic or alkaline conditions of the wastewater.
The biological oxidation processes normally tend to reduce the pH.
The pH ranges between 6.7–8.0, and the typical value is 7.

Alkalinity *
The alkalinity in wastewater is the indicator of the buffer capacity of the medium, which is the resistance to variations in pH. It is caused by the presence of bicarbonate, carbonate, and hydroxyl ions. The Alkalinity ranges from 100 to 250, and the typical value of alkalinity is 200.

<table>
<tr><th>Physical Characteristics</th><th>Chemical Characteristics</th><th colspan="2">Biological Characteristics</th></tr>
<tr><td></td><td></td><td>Chlorides *</td><td>Chlorides originate from drinking water and human and industrial wastes.</td></tr>
<tr><td></td><td></td><td>Oils and Grease *</td><td>The oils and grease together represent the fraction of organic matter, which is soluble in hexane. They originate from oils and fats used in foods found in domestic wastewater.</td></tr>
<tr><td></td><td></td><td>Main organisms in domestic wastewater *</td><td>The main organisms present in domestic wastewater are bacteria, archaea, algae, fungi, protozoa, viruses, and helminths.</td></tr>
<tr><td></td><td></td><td>Pathogenic organisms and indicators of faecal contamination *</td><td>The major groups of pathogenic organisms are bacteria, protozoa, viruses, protozoans, and helminths.
<table>
<tr><th></th><th>Total Coliforms (TC),</th><th>Faecal coliforms (FC) or thermotolerant coliforms,</th><th>Escherichia coli (E. coli)</th><th>Faecal Streptococci</th><th>Protozoan cysts</th><th>Helminth eggs</th><th>Viruses</th></tr>
<tr><td>Range (No./100 mml)</td><td>$10^7 – 10^{10}$</td><td>$10^6 – 10^9$</td><td>$10^6 – 10^9$</td><td>$10^4 – 10^7$</td><td>$<10^4$</td><td>$10^0 – 10^3$</td><td>$10^2 – 10^4$</td></tr>
</table>
The TC, FC, and E. coli are the indicators of faecal contamination, which are normally used.</td></tr>
</table>

* von Sperling, 2007a ** van Niekerk *et al* , 2009 and Nozaic *et al*., 2009
\+ Nozaic *et al*., 2009 ++ Henze *et al*., 2008 +++ van Niekerk *et al* , 2009

2.5 BIOLOGICAL PROCESSES IN WASTEWATER STABILISATION PONDS

There are three possible biological processes, namely aerobic, anoxic, and anaerobic processes, which occur in WSPs. The anoxic biological process occurs inside the FP as a transitional layer between the aerobic and anaerobic zones. These processes (aerobic and anaerobic processes) occur due to the microorganisms, which predominate in the given environment, and the processes are briefly discussed in the following Sections.

2.5.1 Aerobic biological process

There are two organisms involved during the aerobic biological process which are the algae that provide oxygen and the heterotrophs that use oxygen to break down the organics. The aerobic process requires oxygen and nutrients, namely nitrates and phosphates, for metabolism. The growth rate of the aerobic bacteria depends on sufficient organic matter to nutrient ratios, pH, and temperature. The rate of oxygen produced by algae is proportional to solar intensity, which means that larger surface areas result in greater total oxygen production. The oxygen is used by bacteria to break down the organic matter (wastewater) into simple compounds (Ashworth *et al.*, 2011). The dissolved or suspended organic matter is also metabolized by heterotrophic bacteria within the process of the consumption of oxygen by algae that take up the wastewater nutrients.

Ashworth *et al.* (2011) noted that algae play a significant role in treating wastewater, and that it is imperative to understand the trade-off between daytime and night time, including the effect of wind on algae, as discussed in Section 2.8.3.

2.5.2 Anaerobic biological process

In anaerobic conditions, the anaerobic bacteria (methanogenic) break down the organic matter in wastewater and convert them to biogas, mainly methane and carbon dioxide (Nozaic and Freese, 2009). As noted by various authors (Batstone *et al.*, 2002; Sotemann *et al.,* 2005, USEPA, 2011), the major pathways of methane formation processes are:

- The breakdown of acetic acid to form methane and carbon dioxide. This process is called acetogenesis and is represented by Equation 2.1; and
- The reduction of carbon dioxide by hydrogen gas to form methane. This process is called methanogenesis and is represented by Equation 2.2 below.

$$CH_3COOH \rightarrow CH_4 + CO_2 \qquad \text{Equation 2.1}$$

$$CO_2 + 4H_2 \rightarrow CH_4 + 2H_2O \qquad \text{Equation 2.2}$$

These two major pathways of methane formation are presented in Figure 2.4 of Section 2.6.2.

If the retention time is too short, algae cannot grow (Marais and Shaw, 1961). This situation commonly happens, in practice, when anaerobic conditions develop in the primary pond (Marais, 1966).

Marais (1974) also discovered that the die-off rate of faecal organisms is slow under anaerobic conditions in summer and concluded that no or little destruction of faecal bacteria takes place under these conditions.

2.6 TYPES OF WSPs

WSPs are commonly used to reduce the concentration of biochemical oxygen demand (BOD), total suspended solids (TSS), and amounts of coliform to meet the water quality guidelines. They are designed to enhance natural ecosystems that are either anaerobic, aerobic, or facultative. The typical section of a WSP is indicated in Figure 2.2 below.

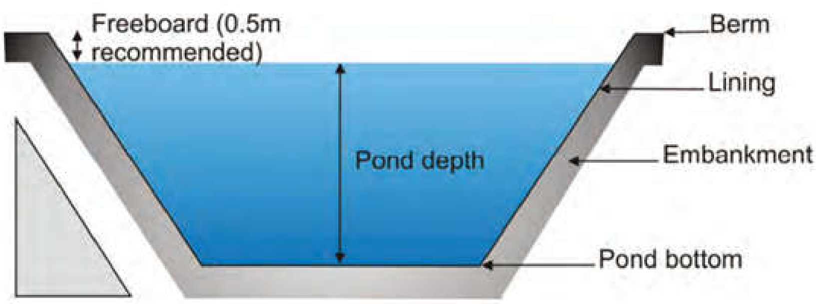


Figure 2.2: The section through a WSP, extract from de Souza and Jack (2010).

The design and modelling concepts of WSPs are discussed in Sections 2.9 and 2.10, respectively. Table 2.3 summarises the applications, loadings, and sizes of WSPs (USEPA, 2011 and von Sperling, 2007b). The comparisons of the main wastewater treatment processes, including their advantages and disadvantages, are discussed in detail in the subsequent Section 2.6, in Table 2.7.

WSPs primarily consist of Anaerobic Ponds (APs), usually followed by a series of aerobic and MPs (Jack *et al.*, 2006 and Ashworth, 2011). Aerobic ponds, normally called oxidation ponds or Maturation Ponds (MPs), are shallow in depth, as indicated in Table 2.3. These types of WSPs are discussed in detail under the following Sections, and the design aspects are indicated in Section 2.9.

The configuration of WSP systems is in agreement with Marais (1966), who stated that WSPs can be subdivided into the following types:

- APs, which are used as pre-treatment structures and help to reduce the size requirements of aerobic ponds.
- Facultative ponds (FPs), which have aerobic and anaerobic conditions maintained inside of them, and where the upper and lower layers are aerobic and anaerobic, respectively.
- High rate oxidation (aerated) ponds, which are aerobic throughout.
- MPs, which are used for treating effluents from conventional disposal works namely the AS systems.
- Mechanically assisted WSPs, which are supplied with oxygen by mechanical aeration, or by recycling the wastewater from one WSP to another for instance from MP to FP, or by mixing the wastewater within the FP.

Pescod and Mara (1988) indicated five types of ponds that are similar to those identified by Marais (1966). The mechanically assisted ponds are referred to as aerated lagoons in their paper. These lagoons are the activated sludge units, without sludge recycle, and they operate at low levels of Mixed Liquor Suspended Solids (MLSS), ranging from 200 to 500 mg/l, with relatively long retention times (Pescod and Mara, 1988).

Butler *et al.* (2015) identified four major types of oxidation ponds, which are aerobic (high-rate), anaerobic, facultative, and MPs. Nozaic and Freese (2009) added that anaerobic, facultative, and MPs are the three major types of WSPs that rely on natural processes, which is also supported by van Niekerk *et al*. (2009). These different types of WSPs are discussed further in the subsequent Sections. The present study will not further discuss mechanically assisted WSPs or aerated lagoons.

Table 2.3: The basic features of wastewater ponds

Type of Pond	Application and Process	Typical surface loading (BOD_5)	Typical Retention time (d)	Typical Depth (m)	Reference
Anaerobic Pond (AP)	• It treats the domestic, industrial, and agricultural wastewaters.		1 – 5	2 – 4	Frederick-van Genderen (1995)
	• It separates the solids from dissolved materials, whereby the solids settle at the bottom of the unit as bottom sludge.		3 – 5	2 – 4	De Souza and Jack (2010)
	• It breaks down the biodegradable organic matter and dissolve the organic matter further.	3000 kg ha/day	1 – 1.5	2 – 5	Butler *et al.* (2015)
	• It stores the undigested matter and non- biodegradable solids as bottom sludge.		1 – 4	2 – 5	Nozaic and Freese (2009)
	• It allows the partially treated effluent to pass to the sequential WSP unit.	280 – 4500 kg / 1000 m^2 /d	5 – 50	2.5 – 4.5	USEPA (2011)
			3 – 6	3 – 5	von Sperling (2007b)
				3 – 5	Ashworth and Skinner (2011)
Facultative Pond (FP)	• It receives the untreated municipal wastewater and, hence, is called Primary Facultative Pond (PFP).	22 – 56 kg / 1000 m^2 /d	7 – 50	0.9 – 2.4	USEPA (2011)
	• It receives the effluent from the primary treatment, trickling filter, aerated pond, or AP and, hence, is called Secondary Facultative Pond (SFP).	120 – 180 kg / ha.d	30 – 50	1.0 – 1.5	Nozaic and Freese (2009)
		100 to 400 kg /ha.day	14 – 21	1.0 – 2.0	Butler *et al.* (2015)
				1.5 – 1.8	Ashworth and Skinner (2011) for PFP
				1.5 – 2.5	Ashworth and Skinner (2011) for SFP
			10 – 14	1.5 – 2.0	Frederick-van Genderen (1995)
			14 – 28		De Souza and Jack (2010)
		100 – 350 kg / ha.d	15 – 45	1.5 – 2.0	von Sperling (2007b)
				1.8 – 2.5	Ashworth and Skinner (2011) for PFP if screenings and grit removal are to take place inside the pond. Refer to Figure 2.12.
Aerobic pond/ MP	• It is used to treat the effluent from other processes through separation, dissolution, and digestion of organic matter.	112 – 225 kg / 1000 m^2 /d	2 – 6	0.18 – 0.3	USEPA (2011)
	• It breaks down most of the remaining organic solids near the pond surface.	100 – 350 kg / ha.d	Function of the pond shape and the required coliform removal efficiency	0.8 – 1.2	von Sperling (2007b)
	• It stores residues of digested matter and non- biodegradable solids as bottom sludge.				
	• It produces effluent, which is low in soluble BOD_5 and high in algal solids.				
	• It allows the treated effluent to flow into waterways or additional treatment systems like wetlands.			0.9 – 1.5	Ashworth and Skinner (2011)
	• It removes the pathogenic microorganisms by solar radiation.	112 – 225 kg / 1000 m^2 /d	2 – 6	1 – 1.15	Butler *et al.* (2015)
				1 – 1.5	van Niekerk *et al.* (2009)
			± 12		De Souza and Jack (2010)
			3 – 15	1.0 – 1.5	Frederick-van Genderen (1995)

Ashworth, J. and Skinner, M. (2011) identified two WSP configurations, namely WSP with and without the AP, as shown in Figure 2.3 and Figure 2.4, respectively, below. The benefits of including the APs are discussed in depth in Sections 2.6.1 and 2.7.

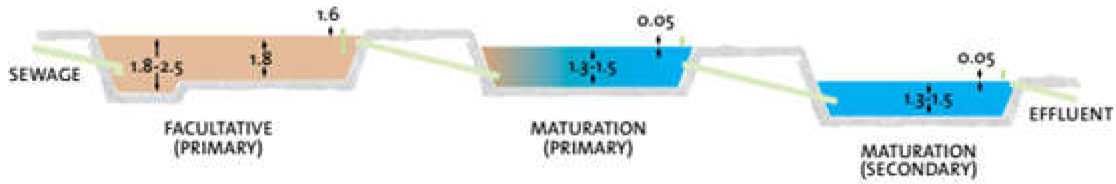


Figure 2.3: The WSP Configurations without the AP (F-M system), extract from Ashworth and Skinner (2011).

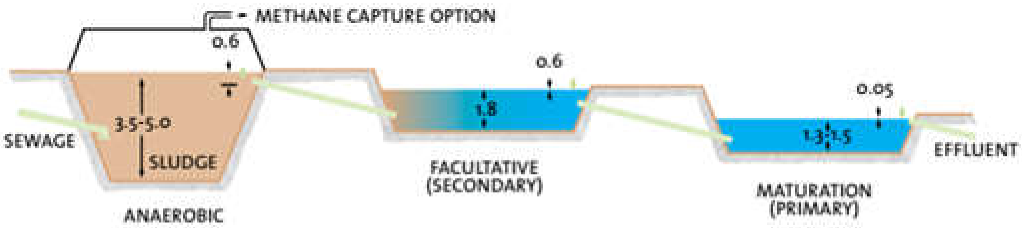


Figure 2.4: The WSP Configurations with the AP (A-F-M system), extract from Ashworth and Skinner (2011).

2.6.1 APs

Butler et al. (2015) indicated that the APs are incorporated alongside the facultative ponds, and that this is a standard practice. USEPA (2011) added that the AP is commonly used for the treatment of heavily polluted industrial and agricultural wastes as a primary treatment unit for the municipal treatment works. The AP operates in the absence of dissolved oxygen in wastewater. The other benefit of APs is to reduce the Nitrogen (N), Phosphorous (P), Potassium (K), and the pathogenic microorganisms by sludge formation and release of ammonia into the air. The N, P and K are associated with the non-biodegradable fraction of the algal cells a remain in sediments, and they settle to the bottom of the pond. The anaerobic biological process is discussed in detail in Section 2.5.2.

Reports from various studies, including Nozaic and Freese (2009), Ashworth and Skinner (2011) and Butler *et al*. (2015), regarding the parameters of APs viz. depth, retention time, BOD and sulphate loadings, and BOD removal efficiency, are summarised below.

2.6.1.1 Pond depth

Ashworth and Skinner (2011) recommended the depth of between 3.5–5 m for APs with pond outlets at a depth of 300 mm below the top water level of the pond. They advised an allowance of approximately 10 m for access around the pond to allow for the desludging of the AP. On the other hand, Butler *et al.* (2015) and Nozaic and Freese (2009) indicated that the APs are commonly 2 to 5 m deep.

2.6.1.2 Retention time

Butler *et al.* (2015) stated that APs are designed with a retention time between 1–1.5 days, for an optimum pH less than 6.2, and temperature greater than 15 ^{0}C. Pescod and Mara (1988) recommended the retention time of the AP to be greater than 1 day. This is also supported by Nozaic and Freese (2009) who suggested a retention time of not less than 24 hours.

Nozaic and Freese (2009) indicated that a retention time of up to 4 days can be required for "strong" wastewater (i.e. wastewater with high BOD loading). They also recommended the retention time of 1–2 days to be sufficient for an influent BOD of up to 300 mg/l; the retention time of 1 day is sufficient at temperatures greater than 20°C. To the contrary, de Souza and Jack (2010), citing Mara (2005), suggested the retention time between 3–5 days at temperatures greater than 20 °C.

2.6.1.3 BOD and sulphate loading

Nozaic and Freese (2009) also recommended the benefits of recycling the final aerobic effluent to the head of a highly loaded AP, at a ratio of approximately 0.5:1, when dealing with or treating wastewater with high BOD loading. In addition to the limit of loadings, Mara and Pearson (1986) proposed an alternative limit of sulphate volumetric loading rate of 500 g SO_4/m^3.d to avoid or minimise odour nuisance. Frederick-van

Genderen (1995), also citing Meiring *et al.* (1968), Gloyna (1971), and Mara *et al*. (1992a), stated that the Sulphate concencration of 500 mg/l is considered to be the maximum that can be tolerated in a pond system.

Butler et al. (2015) indicated that the APs are designed for an organic loading rate of 3 000 kg ha/day. Nozaic and Freese (2009) and Frederick-van Genderen (1995), citing Meiring *et al.* (1998) and Arthur (1998), recommended the practical volumetric loading to be between 100 g BOD / (m^3 day) to 400 g BOD / (m^3 day), whereas Frederick-van Genderen (1995), citing Mara *et al* (1992 a), indicated the upper limit of 300 g BOD / (m^3 day) as a safety margin for odours. However, Nozaic and Freese (2009) highlighted that the odour generation can occur when the loading is above 200 g BOD / (m^3 day).

2.6.1.4 **BOD removal – efficiency**

Butler *et al*. (2015) and Machibya and Mwanuzi (2006) indicated that APs can remove approximately 60 % of BOD. The removal rate depends on the climate conditions and driving force (gravity force applied on the particle and drag force the opposes or resists the movement of the particle) behind the treatment process, called sedimentation. Butler *et al*. (2015) added that the bacteria and viruses are removed by attaching themselves to the settling solids in the pond or perishing due to predators or loss of food sources. The observation of helminths settling to the bottom of the AP is in agreement with Butler *et al*. (2015). The authors also mentioned the minimum BOD removal of 60% and approximate BOD removal range of 60–85% in warm climates even for relatively short retention times.

Table 2.3 indicates the minimum retention time and depth of 1.5 day and 2 m, respectively, for the maximum retention time and depth of 50 day and 4.5 m, respectively. It is also indicated by von Sperling (2007b), Henze *et al*. (2008), van Niekerk *et al.*, (2009) and Nozaic *et al.* (200) that the performance of the AP is dependent on both the strength and temperature of the influent wastewater.

2.6.2 FPs

FPs are ponds for which the treatment process is both anaerobic and aerobic. According to Butler *et al.* (2015) and Shilton (2005), FPs are subdivided into three zones namely:

- The aerobic surface zone, consisting of aerobic bacteria and algae;
- The anaerobic bottom zone, consisting of anaerobic bacteria; and
- The zone between the anaerobic and aerobic zones where bacteria can thrive in both anaerobic and aerobic conditions. This region is also called the anoxic zone.

The process indicated above is presented in Figure 2.5 below, which is extracted from Shilton (2005). The aerobic and anaerobic biological processes are discussed in Section 2.5.

The sludge accumulation in ponds is between 0.01 and 0.04 m^3/ person year. Ashworth and Skinner (2011) also advised to allow approximately 20% of the volume of the FP for desludging. Nozaic and Freese (2009) added that desludging is possible once in every 10 to 15 years and several of these ponds can operate for much longer without being de-sludged. Mara (2009) recommended the septic and Imhoff tanks to facilitate desludging, particularly in winter where both tanks also protect algae in the FP.

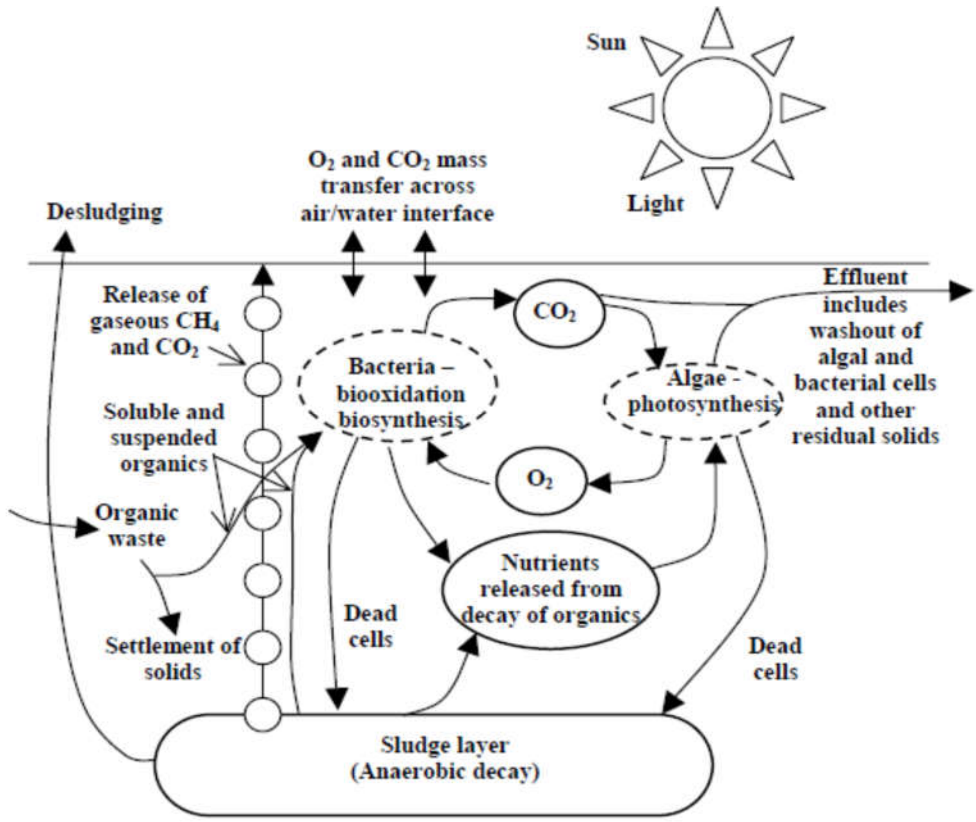


Figure 2.5: The basic biological activities in a FP, extract from Shilton (2005).

2.6.2.1 <u>Pond depth and length to breadth ratio</u>

Ashworth and Skinner (2011) indicated that the design of the FP depends on whether the screening and grit removal processes are required. When these are not required, the recommended depth is 2.5 m, for the first third of the FP, and 1.5 m, for the remaining two thirds, with the outlet located at approximately 600 mm depth from the water surface level. Ashworth and Skinner (2011) added that when the two processes are required, a depth of at least 1.8 m and length to breadth (L/B) ratio of 3 to 1 are recommended. However, von Sperling (2007b) recommended the L/B ratio of 2 to 4 for FPs and highlighted the fact that FPs with a L/B ratio greater than 1 tend to plug flow conditions, whereas those with a L/B ratio close to 1.0 approach the completely mixed conditions. Gopolang and Letshwenyo (2018) indicated that the L/B ratio of 10 to 20 can be achieved in order to have a pug flow conditions for secondary facultative and maturation ponds.

Nozaic and Freese (2009) recommended a depth between 1–1.5 m. To the contrary, von Sperling (2007b) acknowledged that the literature regarding the optimisation of FP depth is limited and adopted a depth ranging between 1.5–3 m. In addition, The author specified that the depth of 1.5–2.0 m is commonly used in practice. Butler *et al.* (2015), on the other hand, indicated the average depth of the FP to be between 1–2 m.

2.6.2.2 BOD loading, efficiency and retention time

Butler *et al.* (2015) indicated that the BOD removal efficiency in the FPs is approximately 95% when BOD loading is within a range of 100–400 kg BOD/ha/day. They also added that the treatment time can approximately be 2 to 3 weeks.

2.6.2.3 Design methods - modelling

Butler *et al.* (2015) indicated five design methods for the FPs, namely Areal loading rate, Gloyna equation, Plug flow model, Marais and Shaw, and Thirumurthi application. On the other hand, Nozaic and Freese (2009) indicated the Marais sequence (First order kinetics), McGarry and Pescod curve, and Mara global design empirical equation as the three design methods for the FPs. These design methods are explained and discussed in detail in Section 2.9.2, including the design approaches cited by Ho, *et al.* (2017), Umara, *et al.* (2010), and Pescod and Mara (1988).

2.6.3 Aerobic (high-rate) ponds

Aerobic Ponds (AePs) are also known as high rate algal ponds that can maintain the dissolved oxygen between 300–450 mm in the deep pond. These ponds are well known for having a high BOD removal due to the algal photosynthetic process or activity (Butler *et al.*, 2015). Algae and bacteria interactions are discussed in Section 2.8.5, and the overview of the aerobic biological process is given in Section 2.5.1.

Marais (1966) noted that the rule of thumb for designing AePs is adequate when the ponds receive a pre-treated effluent. The rule of thumb states the following:

- The primary pond should be 1.2 m deep and have 1.3 m^2 of pond surface per person; and

- The individual secondary and tertiary ponds should be 1.2 m deep and have 0.65 m^2 of pond surface per person.

Butler *et al.*, (2015) indicated that these ponds have specific characteristics viz. the retention time of approximately 2–6 days, BOD loading rate between 112–225 kg/(1000 m^3 day), and BOD removal efficiency of approximately 95%. It should be noted that the BOD removal efficiency of AePs is similar to the BOD removal efficiency of FPs.

2.6.4 MPs

MPs operate similarly to FPs whereby algae are used as a primary driving force for the treatment process (Butler *et al.*, 2015). The main purpose of these ponds is to remove the FC, pathogens, and nutrients by utilising the sun's UV light, hence they have to be shallow. MPs are used to polish the effluent quality of conventional Wastewater Treatment Plants (WWTPs). The MP can also be used as a tertiary treatment for a conventional WWTP.

2.6.4.1 Pond depth

The characteristic depth of MPs is between 1–1.15 m (Butler *et al.*, 2015). On the other hand, van Niekerk *et al.* (2009) recommended the water depth to be between 1–1.5 m. Ashworth and Skinner (2011) indicated that the depth of MPs varies from 1.3 m to 1.5 m. They recommended an optimum depth of 1.3 m and stated that the depth should not be less than 1 m.

A review of the literature revealed that different authors, particularly Butler *et al.* (2015), Ashworth and Skinner (2011), and van Niekerk *et al.* (2009), had different recommended MP depth values, as indicated in Table 2.3. Butler *et al.* (2015), van Niekerk *et al.* (2009), and Frederick-van Genderen (1995) all three agreed on the minimum pond depth of 1 m, while their recommended maximum MP depth varies between 1.15–1.5 m.

2.6.4.2 Organic loading

Ashworth and Skinner (2011) also stated that MPs tend to reduce the organic load by approximately 20% and their primary function is disinfection. However, Mara and Pearson (1987) suggested the 25% removal of filtered BOD in each maturation pond.

2.7 COMPARISON BETWEEN THE WSP

Comparative analyses of the liquid and solid phases between the main WSP units andsystem are presented in the Table 2.4 to Table 2.7 for domestic wastewater (von Sperling, 2007a).

The comparative analyses are summarised as follow:

- The average effluent concentrations and typical removal efficiencies: they refer to the capacities of the main wastewater treatment systems for reaching different quality levels of BOD, COD, SS, Ammonia, Total Nitrogen, Total Phosphorous, FC, and helminth eggs.
- The typical characteristics of the main wastewater system: they refer to the values of land requirement per capita, power for aeration and sludge volume, and production of sludge to be disposed, of the main wastewater treatment processes.
- The qualitative comparative analyses: they cover various aspects in the evaluation of the wastewater treatment systems, including the analyses of the problems related to their efficiencies, economies, processes, and environments.
- The basic equipments: they include all the necessary components used in the main wastewater treatment systems.
- The main advantages and disadvantages: they consider all the pros and cons of the main wastewater treatment systems.

Table 2.4: The average qualities of the effluent and corresponding average removal efficiencies (adopted from von Sperling, 2007a)

System/ unit process	BOD_5 (mg/l)	COD (mg/l)	SS (mg/l)	Ammonia (mg/l)	Total N (mg/l)	Total P (mg/l)	FC (FC/100ml)	Helminth eggs (eggs/ l)	BOD_5 (%)	COD (%)	SS (%)	Ammonia (%)	Total N (%)	Total P (%)	FC (log units)
Primary treatment (septic tanks)	200–250	400 – 450	100 – 150	> 20	>30	>4	10^7 - 10^8	>1	30–35	25 – 35	55 – 65	< 30	< 30	< 35	< 1
Conventional primary treatment	200–250	400 – 450	100 – 150	> 20	>30	>4	10^7 - 10^8	>1	30–35	25 – 35	55 – 65	< 30	< 30	< 35	< 1
FP	50–80	120 – 200	60 – 90	> 15	>20	>4	10^6 - 10^7	<1	75 - 85	65 – 80	70 – 80	< 50	< 60	< 35	1 – 2
AP and FP	50–80	120 – 200	60 – 90	> 15	>20	>4	10^6 - 10^7	<1	75 - 85	65 – 80	70 – 80	< 50	< 60	< 35	1 – 2
AP, FP, and MP	40–70	100 – 180	50 – 80	10–15	15 - 20	<4	10^2 – 10^4	<1	80–85	70 – 83	73 – 83	50–65	50–65	< 50	3 – 5
AP, FP, and High rate pond	40–70	100 – 180	50 – 80	5–10	10–15	3–4	10^4 – 10^5	>1	80–85	70 – 83	73 – 83	65 - 85	75 - 90	50 – 60	3 – 4
AP, FP, and Algae removal	30–50	100 – 150	<30	>15	>20	>4	10^4 – 10^5	>1	85 - 90	75 – 83	>90	<50	<60	<35	3 – 4
Constructed Wetlands (CWs)	30–70	100 – 150	20–40	>15	>20	>4	10^4 – 10^5	<1	80–90	75 – 85	87 – 93	<50	<60	<35	3 – 4
Septic tank and Anaerobic filter	40–80	100 – 200	30 – 60	> 15	>20	>4	10^6 - 10^7	>1	80–85	70 – 80	80 – 90	<45	<60	<35	1 – 2
Septic tank and Infiltration	<20	<80	<20	<10	<15	<4	10^3 – 10^4	<1	90–98	85 – 95	>93	>65	>65	>50	4 – 5

Table 2.5: The typical characteristics of the main domestic wastewater systems (adopted from von Sperling, 2007a)

System/ unit process		Power for aeration		Sludge Volume	
	Land requirements (m^2/ capita)	Installed power (W / capita)	Consumed power (kWh / capita. year)	Liquid sludge to be treated (L / capita. year)	Dewatered sludge to be disposed of (L / capita. year)
Primary treatment (septic tanks)	0.03 – 0.05	0	0	110 – 360	15 – 35
Conventional primary treatment	0.02 – 0.04	0	0	330 – 730	15 – 40
FP	2.0 – 4.0	0	0	35 – 90	15 – 30
AP and FP	1.2 – 3.0	0	0	55 – 160	20 – 60
AP, FP, and MP	3.0 – 5.0	0	0	55 – 160	20 – 60
AP, FP, and High rate pond	2.0 – 3.5	<0.3	<2	55 – 160	20 – 60
AP, FP, and Algae removal	1.7 – 3.2	0	0	60 – 190	25 – 70
Constructed Wetlands (CWs)	3.0 – 5.0	0	0		
Septic tank and Anaerobic filter	0.2 – 0.35	0	0	180 – 1000	25 – 50
Septic tank and Infiltration	1.0 – 1.5	0	0	110 – 360	15 – 35

Table 2.6: The relative evaluation of the main domestic wastewater treatment systems , liquid phase (adopted from von Sperling, 2007a)

System/ unit process	Removal efficiency			Economy					Resistance capacity to influent variations and shock loads			Reliability	Simplicity in O&M	Independence of other characters for good performance		Lower possibility of environmental problems			
	BOD	Nutrients	Coliforms	Requirements		Costs		Generation of sludge	Flow	Quality	Toxic			Climate	Soil	Bad odours	Noise	Aerosols	Insects and worms
				Land	Energy	Constr	O&M												
Primary treatment (septic tanks)	0	0	0	5	5	5	4	5	5	5	5	5	3	5	5	1	4	5	3
Conventional primary treatment	1	1	1	5	4	4	3	3	4	5	4	4	3	4	5	2	4	5	3
FP	3	2	4	1	5	3	5	5	4	4	3	4	5	2	3	3	5	5	2
AP and FP	3	2	4	2	5	4	5	5	4	4	3	4	5	2	3	1	5	5	2
AP, FP, and MP	3	3	5	1	5	3	5	5	4	4	3	4	5	2	3	3	5	5	2
AP, FP, and High rate pond	3	4	4	2	4	3	4	5	4	4	3	4	3	3	3	3	2	2	2
AP, FP, and Algae removal	4	2	4	2	5	3	4	5	4	4	3	4	5	2	2	2	5	5	2
Constructed Wetlands (CWs)	4	2	3	1	5	3	3	5	4	4	3	4	4	2	2	2	5	5	2
Septic tank and Anaerobic filter	3	1	2	5	5	3	3	4	3	3	2	3	4	2	5	2	4	5	4

5 – Most favourable, 1 – Least favourable, 2 to 4 – Intermediate grades in increasing order, 0 – Zero effect

Table 2.7: The minimum equipment of the main domestic wastewater treatment processes (adopted from von Sperling, 2007a)

Treatment process	Basic equipment
Preliminary Treatment	Screens, grit chamber, flow meter
FP	None
AP and FP	Effluent recycle pump (optional)
High rate pond	Rotors for fluid movement
Constructed Wetlands (CWs)	None
Septic tank and Anaerobic filter	None

Table 2.8: The comparisons of the main domestic wastewater treatment processes – advantages and disadvantages (adopted from von Sperling, 2007a)

Systems	Advantages	Disadvantages
WSP systems		
FP	• Efficient BOD removal. • Reasonable pathogen removal efficiency. • Easy construction, operation and maintenance. • Absence of mechanical equipment. • Reduced construction and operational costs. • Satisfactory resistance to load variations. • Requirement for sludge removal after 20 years. • No energy requirements.	• High land requirements. • Operational simplicity can lead to the disregard to maintenance (growth of vegetation). • Difficulty to achieve the restricted discharge guidelines. • Possible requirement for removal of algae from effluent to comply with the stringent discharge guidelines. • Possible insect growth. • Variable performance with climatic conditions.
AP and FP	Same as the above FP system, including: • Low land requirements than single FP.	• The same as the above FP system. • Random requirement for effluent recycling to control bad odours. • Requirement for periodic removal of sludge from AP. • Possibility of bad odours in the AP. • Requirement for a safe distance from surrounding neighbourhoods.
AP, FP and MP	• Lowest land requirements for all pond systems. • Greater independence from climatic conditions than the FP and AP-FP systems. • Relatively easy construction, operation and maintenance. • Reduced possibilities of bad odours. • Satisfactory resistance to load variations. • High pathogen removal efficiency. • Reasonable nutrients removal efficiency.	• Very high land requirements. • Introduction of equipment and relatively high energy requirements (optional). • Low coliform removal efficiency. • Marginally increase in the sophistication level. • Requirement for periodic removal of sludge from AP.
AP, FP, and High rate pond	• Same as the preceding ponds. • Good pathogen removal efficiency. • High nutrients removal efficiency.	• Same as the preceding ponds.

Systems	Advantages	Disadvantages
Land disposal Systems		
Constructed Wetlands (CWs)	• High BOD and coliforms removal efficiency. • No energy requirements. • Easy construction, operation and maintenance. • Good resistance to load variations. • Reduced construction and operational costs. • Possible using the produced plant biomass. • No sludge treatment required.	• High land requirements. • The influent to be previous treated before it goes to the CW. • Possible mosquitos in surface flow system. • Requirement for macrophytes handling. • Requirement for substrate, namely gravel or sand. • Susceptible to clogging.
Anaerobic reactors		
Septic tank and Anaerobic filter	• Reasonable BOD removal efficiency. • Low land requirements. • Low construction and operational costs. • No energy requirements or consumption. • Easy construction, operation and maintenance. • Tolerance to influents highly concentrated in organic matter. • Possible of energy use of biogas. • Sludge with good dewatering. • Very low sludge production. • Sludge stabilisation on the reactor itself. • Requirement for only dewatering and final disposal of sludge. • Rapid start up after periods of no use (biomass is preserved for various months). • Good adoption to different wastewater types and concentration. Good resistance to load variations.	• Difficulty in complying with restrictive guidelines. • Low coliform removal efficiency. • Practically no N and P removal. • Possibility of the generation of bad odours, although it is controllable. • Possible of the generation of an effluents with unpleasant aspects.

2.8 CHARACTERISTICS AND MECHANISM OF OPERATION OF PONDS

This section summarises the work done by Marais and Shaw (1961), Maris (1966 and 1974), Nemeche and Vasel (1998), Ukpong *et al.* (2006), and others regarding the characteristics and mechanism of operation of ponds in Southern Africa. The literature indicates the fundamentals to consider when dealing with the design of the WSPs, including the assumptions made when developing the design theory.

Marais and Shaw (1961) highlighted that the effluent from a single pond (anaerobic, facultative or maturation pond) does not meet the bacteriological and chemical requirements of South Africa. They advised and developed a system using two or more ponds in series so that the DWS requirements were met. In their paper, the focus was only on two pollutants, namely BOD and faecal bacteria. They found that the concentration in the pond is determined by the monomolecular constant K and physical characteristics of the system, namely the influent and effluent flows and concentrations, and the volume of the pond.

The wastewater in the WSPs is biologically treated by natural processes which involve, among others, the use of algae and bacteria. This is also in agreement with Tharavathy *et al.* (2014) who stated that the oxidation ponds consist of different groups of organisms, including bacteria, algae, protozoa, fungi, viruses, and insects. They mentioned that these organisms exist and compete with each other.

The bacteria decompose the biodegradable organic matter and release carbon dioxide (CO_2), ammonia (NH_3), and nitrates (NO_3^-) in the process. Additionally, these compounds or products are utilized by algae with the presence of sunlight and photosynthesis process, and algae release oxygen during the process. This enables the bacteria to break down more wastewater organic matter and achieve the reduction in BOD level, thus improving the effluent quality from the WSP. These biological processes, namely aerobic and anaerobic processes and the effect of algae, are discussed in detail in Sections 2.5 and 2.8.5.

2.8.1 Effect of temperature on the sludge layer of the WSP

Marais (1966) indicated that temperature has an important role in respect to the efficiency of anaerobic fermentation in the sludge. The author added that the degradation occurring in the sludge layer can be incorporated in the differential equation, particularly when it is assumed that the anaerobic degradation of the sludge is a first order reaction.

The theory of Marais (1966) proved that when the equilibrium conditions are reached, there is an annual cyclic variation of sludge accumulation whereby the amount of sludge in the WSP decreases in summer and increases in winter. The author added that there is little degradation that takes place in the sludge during the cold season, and the bottom of the pond becomes the sludge storage space. Marais (1974) also noted that the sludge increases its fermentation rate in summer.

The sludge accumulated during the winter season is off-loaded to the supernatant liquor during the hot season (Marais, 1966). On the contrary, during the summer season, the rapid degradation rate and low BOD equilibrium in the pond are established. The author further concluded that the BOD in the pond seemed to be insensitive to the seasonal variations in temperature. Mara (2003) indicated the mechanistic model for sludge accumulation in primary FPs to be the model that includes sedimentation of the influent settleable solids, digestion of the settled solids, and compaction of the sludge.

2.8.2 Effect of stratification in WSPs

Ukpong *et el.* (2006) indicated that the existence of stratification in ponds cannot be ignored as it has a lot of implications on aspects of the ponds, namely the pond design, sampling, and operation. They also noted that stratification can occur in ponds as shallow as 0.2 m. In addition, Nameche and Vasel (1998) noted that ponds that are deeper than 2 m or 3 m normally experience thermal stratification during the summer season.

Marais (1974) stated that the thermocline and mix (concentration) in the pond are disturbed under windy conditions, and the length of stratification is usually longer in summer than in winter season, for the same wind speed.

Marais (1966 and 1974) indicated that the periods of stratification and mixing can be identified by monitoring the temperature of the pond at the top and bottom levels. The author's study mentioned that when the temperatures of both the top and bottom of the pond are the same, mixing takes place, whereas when there is a difference in temperature, stratification takes place.

Ukpong, Agunwamba and Egbuniwe (2006) noted that the thermal stratification influences the distribution and variation of both the physicochemical and biological parameters inside the water body of the WSP. The authors performed tests and measurements of the Dissolved Oxygen (DO), pH, Biochemical Oxygen Demand (BOD_5), Chemical Oxygen Demand (COD), Suspended Solids (SS), Ammonia nitrogen ($NH_3 - N$), Total Phosphorus (TP), and Coliform bacteria at various depths. They noted that there was a variation in the above-mentioned parameters with depth.

The following were the findings of the tests performed in the vertical profiles of both the Laboratory Scale WSP (LSWSP) and Field WSP (FWSP). These tests were performed on a weekly basis (Ukpong *et al.*, 2006):

- **Dissolved Oxygen (DO)** – It was found that when the temperature increases, the DO at the surface layer decreases. The minimum and maximum DO values were found to be at the bottom and surface layers of the WSP, respectively.
- **Hydrogen ion concentration (pH)** – It was found that the pH values are controlled by photosynthesis at the surface layer of the WSP, whereas they are controlled by the anaerobic degradation at the bottom layer of the WSP. It was also noted that there was an increase in pH from the bottom to the surface layer and, hence, high pH values are obtained at the surface layer of the pond.
- **Biochemical Oxygen Demand (BOD) –** It was found that there were variations in the BOD values in the water column, which indicated that the maximum and

minimum BOD values were found at the bottom and surface layers of the WSP, respectively.

- **Chemical Oxygen Demand (COD)** – It was found that the COD values were highest and lowest at the bottom and surface layers of the WSP, respectively. The former is found to be due to the anaerobic re-dissolution and digestion of the matter previously settled and sedimented. The latter is found to be associated with maximum DO, pH, and temperature.
- **Coliform bacteria** – It was found that the bacteria die off rate was higher at the surface layer, compared with the bottom layer of the WSP. This was due to bicarbonates, which are utilized in the metabolic activity of algae. This is also in agreement with observations made by Almasi and Pescod (1996).
- **Ammonia nitrogen, Phosphorus, and Suspended Solids (SS) –** It was found that the concentrations of these parameters were higher at the bottom layer, compared with the surface layer of the WSP.

Ukpong, Agunwamba and Egbuniwe (2006) indicated the requirement to undertake and investigate the occurrence of stratification when data are collected at an hourly interval, over 24 hours, during both the winter and summer season. By doing so, the outcomes of the investigation will enhance and improve the understanding of the interaction and trade-off between the stratification and pond performance.

2.8.3 Effect of wind in WSPs

Ukpong (2012) stated that wind has an important effect on the mechanism of operation of FPs, as it induces the vertical mixing of the pond liquid. The author concluded that the wind sweeping over the WSP surfaces has an effect on the performance of the WSP. For example, the wind reduces the coliform number, which has an impact on the efficiency of the pond, and the odour problems are minimised. Previous studies on modelling reveal that the addition of baffles or wind-induced mixing improves the hydraulic conditions in the pond and the effluent quality of the pond (Sah *et al*., 2011).

Nameche and Vasel (1998) stated that baffles should be carefully orientated with respect to the prevailing wind direction, as they can improve the efficiency of the biological reactors. They also mentioned that macro-mixing conditions, caused by wind and aerators, are represented by the axial dispersion coefficient, which can also depend on the pond dimensions. Frederick-van Genderen (1995), Metcalf and Eddy (1972), and Spellman and Drinan (2014) indicated that the WSPs have a dispersion number (D) value which varies between 0.1–2.0, and the axial dispersion coefficient can be calculated by using Equation 2.20 in Section 2.9.2.1.

Ukpong (2012) concluded, after verification of the wind effect model, that there is a requirement to investigate its application in WSPs. This is also in agreement with Sah *et al*. (2011) whose conclusions are based on the simulation results from the model (Delf3D software). However, Sah *et al*. (2011) indicated that the model provided in the Delf3D software required a further calibration and validation base.

2.8.4 Mixing conditions in WSPs

The study carried out by Marais (1966) in oxidation ponds in Lusaka reveals three states of mixing as follows:

- A complete mixing occurring in the morning (from 6 am to 12 pm) whereby the temperature is uniform throughout the pond;
- A stratification and thermocline developing during the course of the day (from 12 pm to 9 pm) whereby the temperature above the thermocline increases to a maximum, at approximately 4 pm, and subsequently decreases. On the contrary, the temperature below the thermocline decreases to room temperature and remains constant thereafter.
- A third period of mixing occurring in the afternoon and evening:
    - During the calm conditions, the top layers lose heat more quickly, compared with the bottom layers.
    - During the windy conditions, the thermocline overcomes the stratification forces and leads to the mixing of the warmer and colder layers adjacent to it; this is caused by the wind induced water above the thermocline.

It was also observed that during the mild and hot conditions (summer season), the E. coli concentrations in the surface overflow of the pond, during the mixing period (morning), were higher than those during the stratification period (during the day). Also, the thermocline is found to be near the pond surface in the summer season, and the E. coli concentrations are found to be higher in the summer season, compared with the winter season.

The conclusions by Nameche and Vasel (1998) highlighted the ponds with a low length/width ratio (i.e. below 8) correspond well to completely mixed reactors.

2.8.5 Effect of algae in WSPs and interaction between algae and bacterial

Von Sperling (2007b) indicated that algae play a fundamental role in FPs, and their concentration is higher than the concentration of bacteria. Marais (1966) carried a study on the behaviour and concentration of algae in a 5-foot (1.5 m) deep primary oxidation pond in Lusaka. The observations were done over a year, and the findings were as follows:

- In the winter season, the concentration of algae increased, and *Micractinium* was dominant during this season.
- In the summer season:
    - The concentration of algae decreased, and *Euglena* and *Chlorella* were dominant.
    - The upper thermal limit of algae is exceeded, and this results in a decrease in their concentration, in the tropics.
- In both the winter and summer seasons, the mixing affects the algal concentration.

Ashworth *et al.* (2011) noted the relationship between algae and bacteria as illustrated in Figure 2.6. The relationship is summarised as follows:

- Aerobic bacteria break down (catabolism) the organic waste to give off carbon dioxide.
- Algae use the carbon dioxide for cell growth (anabolism) and give off oxygen during daytime.
- Aerobic bacteria use oxygen given off by algae and diffused from the atmosphere for their own metabolism and new cell production.

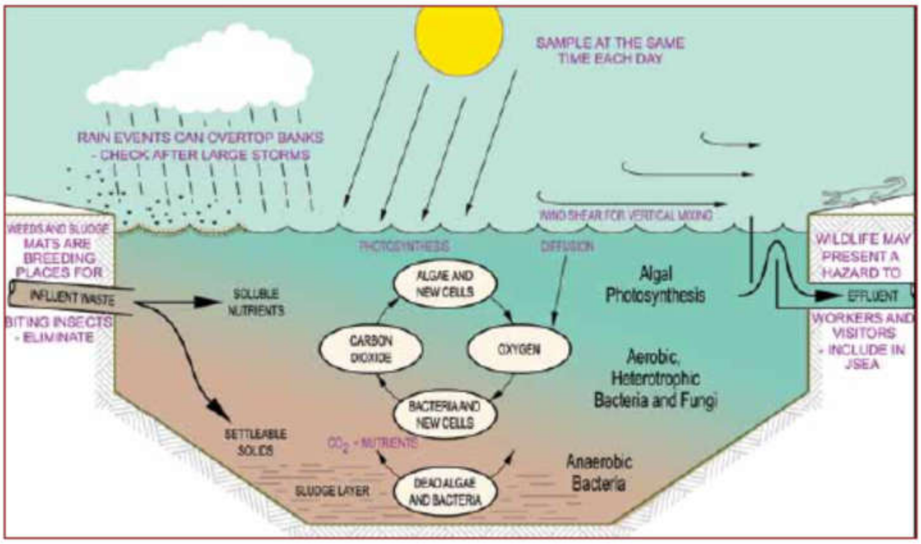


Figure 2.6: Algae and bacteria relationship during the daytime, extract from Ashworth and Skinner (2011).

Marais (1966) noted that, in the tropics, the algae concentration in a primary pond increases during winter and decreases during summer. The decrease in algae concentration seems to be due to windless days and long periods of stratification. It was noted that the mixing affects the algal concentration, and the primary pond stands a better chance of being anaerobic in summer than in winter. The process of photosynthesis plays a role during the seasons and depends on if the process is either during the day time or night time as briefly discussed below.

USEPA (2011) defined photosynthesis as a process whereby organisms use solar energy to fix CO_2 and obtain the reducing power to convert it to organic compounds.

From the literature, it was observed that authors classified the photosynthesis in a WSP as oxygenic or anoxygenic, and both depend on the source of reducing power used by a particular organism. Both photosynthesis processes are illustrated in Figure 2.7.

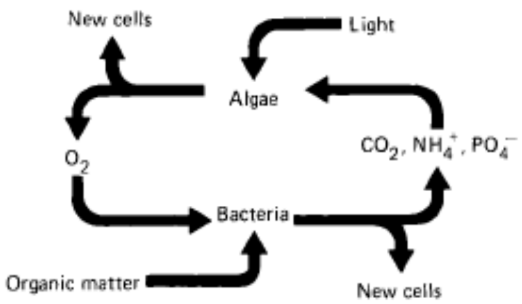


Figure 2.7: The relationship between algae and bacteria in WSPs, extract from Pescod and Mara (1988).

Water serves as the reducing source (electron donor) in oxygenic photosynthesis, with oxygen (O_2) as a by-product, and is represented by Equation 2.3 (USEPA, 2011). The oxygenic photosynthesis algae and cyanobacteria convert CO_2 to organic compounds, which serve as the major sources of chemical energy for the aerobic organisms. Aerobic bacteria require the O_2 produced to function in their role as consumers that degrade complex organic matter.

$$H_2O + \text{sunlight} \rightarrow 1/2O_2 + 2H^+ + 2e^- \qquad \text{Equation 2.3}$$

USEPA (2011) stated that the anoxygenic photosynthesis does not produce O_2 and occurs in the absence of O_2. The anaerobic bacteria obtain energy by reducing the inorganic compounds. According to USEPA (2011), the photosynthetic bacteria utilize the reduced Sulphur (S) compounds or elements in anoxygenic photosynthesis as represented in Equation 2.4.

$$H_2S \rightarrow S^0 + 2H^+ + 2e^- \qquad \text{Equation 2.4}$$

Ashworth *et al.* (2011) also indicated that the relationship between algae and bacteria during night time is as illustrated in Figure 2.8. Algae use oxygen and give off carbon dioxide through the respiration process. The carbon dioxide is stored in water and provides food for algae during the daytime; it will be processed through photosynthesis. Ashworth *et al.* (2011) indicated that the storing of carbon dioxide (CO_2) lowers the pH during the night time. To the contrary, the use of carbon dioxide during the daytime can increase the pH to above 10, and this depends on the alkalinity.

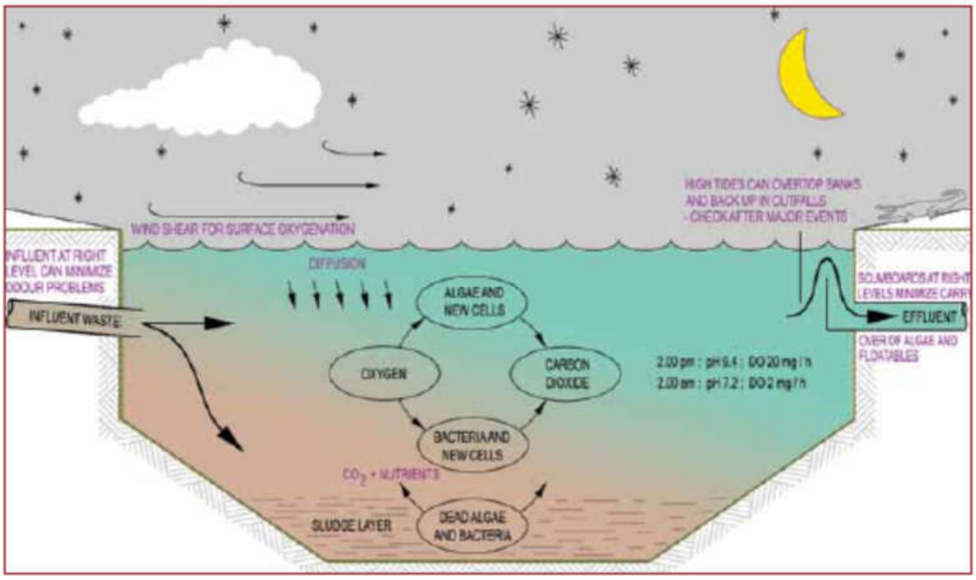


Figure 2.8: The relationship between algae and bacteria during the night time, extract from Ashworth and Skinner (2011).

USEPA (2011) defined respiration as a physiological process by which organic compounds are oxidized into carbon dioxide and water. USEPA (2011) represented the aerobic respiration by the Equation 2.5.

$$C_2H_{12}O_6 + 6O_2 + \text{enzymes} \rightarrow 6CO_2 + 6H_2O + \text{new cells} \qquad \text{Equation 2.5}$$

USEPA (2011) indicated that when there is light, both respiration and photosynthesis can occur simultaneously in algae. However, the respiration rate is low, compared with the photosynthesis rate, and hence, the photosynthesis process prevails.

Ashworth *et al.* (2011) also noted that the dominance of the individual algae species give an indication of the effectiveness of the treatment process. They went on describing five distinct WSP operations as follows:

1) Under loaded FPs – *Euglena polymorphia* are dominant.
2) Light loaded FPs or still wind conditions – *Cyanobacteria* (blue green algae), which form mats and create odours, and *Scenedesmus* are dominant.
3) Normally loaded FPs – *Euglena, Phacus*, and *Chlorella* are dominant.
4) Highly loaded FPs – *Chlamydomonas* and *Pandorina* are dominant.
5) Failed FPs – Algae have died from high organic and sulphate levels. Dominance of purple and green anaerobic photosynthetic bacteria.

2.8.6 Effect of pond configuration

The configuration or arrangement of the ponds plays an important role regarding the performance of the WSP system.

Butler *et al.* (2015) highlighted the two different arrangements for a multiple pond system. They also added that the choice of the multiple pond has advantages compared with the single pond configuration. The idea regarding the reduction of bacteria is also indicated by Marais and Shaw (1961), as discussed in Section 2.9.2.1, and shown in Figure 2.13.

Butler *et al.* (2015) indicated two types of pond arrangements, which are:

- Serial arrangement: the wastewater is treated in the initial and subsequent ponds and then polished in the final pond.
- Parallel arrangement: the wastewater is evenly distributed in the parallel pond arrangement.

2.8.6.1 **Serial pond arrangement**

Ponds in series are suitable during the summer season and periods of low biological loading, as advised by Butler *et al.* (2015). Mara (2003) indicated the first pond to be an AP, followed by a FP, and finally MPs, as shown in Figure 2.9 below. The MPs can be omitted if the effluent quality meets the required discharge guidelines, as indicated in Figure 2.10.

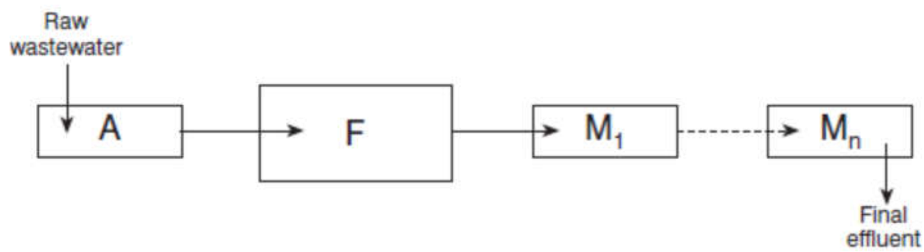


Figure 2.9: The typical WSP layout indicating the serial configuration: A for AP, F for FP, and M_1 to M_n for MPs, extract from Mara (2003).

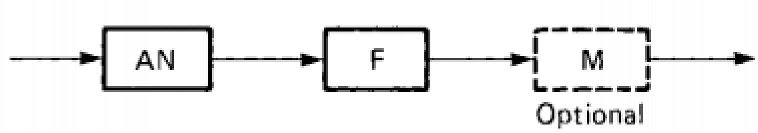


Figure 2.10: The typical WSP layout indicating the serial configuration: AN for AP, F for FP, and M for MPs, extract from Pescod and Mara (1998).

2.8.6.2 **Parallel pond arrangement**

Butler *et al.* (2015) and (USEPA, 2011) indicated that ponds operating in parallel have the following benefits:

- Prevention of treatment interruption in cooler seasons when the pond can be experiencing low biological activity.
- Reduction of issues related to the periodic low dissolved oxygen concentrations, which happen in the morning hours.
- Effective Reduction of pond loadings, particularly when compared with the serial configuration.

In addition to the above benefits, the parallel arrangement is useful during the desludging of the FP or AP. This arrangement is shown in Figure 2.11 and Figure 2.12. It is recommended to be used when the population reaches approximately 10 000 (Butler *et al.,* 2015).

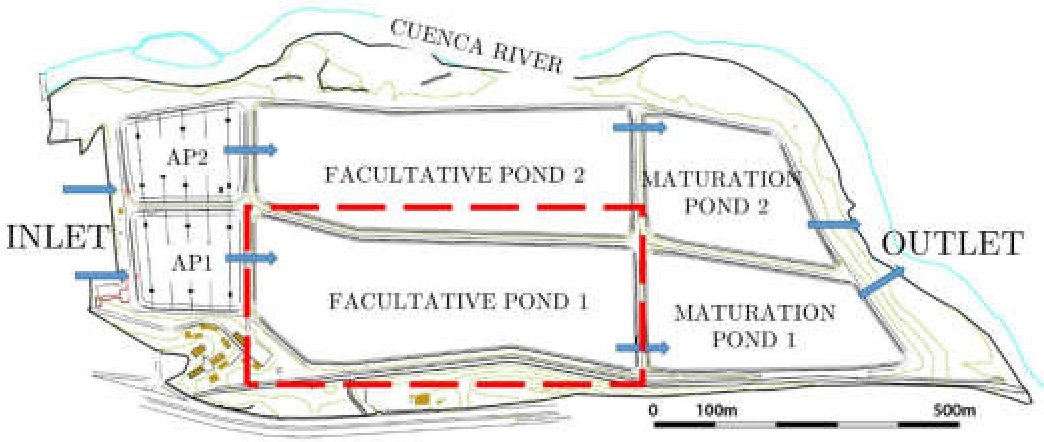


Figure 2.11: The typical WSP layout indicating the parallel configuration, extract from Ho *et al.* (2018).

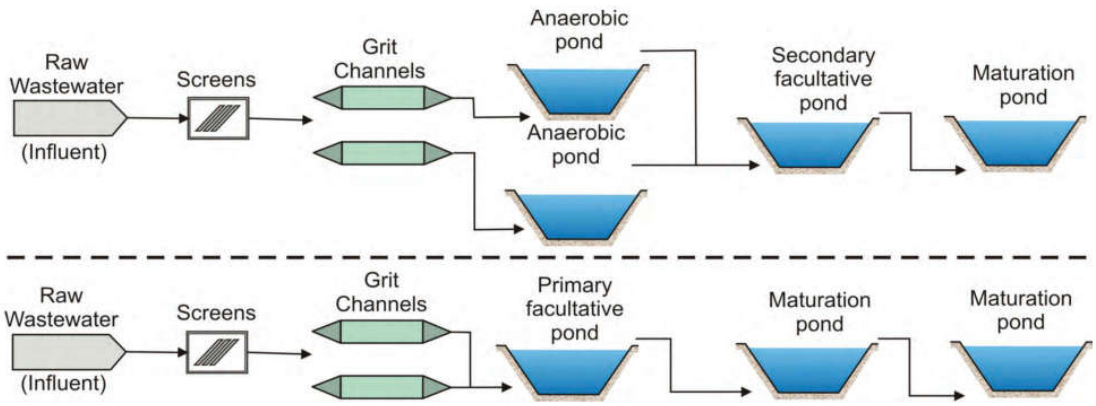


Figure 2.12: The A-F-M system and FM system, extract from de Souza and Jack (2010).

2.9 DESIGN OF WSPs

There are different methods that can be used for the design of WSPs. These design methods depend on the desired effluent quality, assumptions and limitations, and pond type (Butler *et al.,* 2015). Pescod and Mara (1988) stated that the WSPs are designed to achieve three different forms of treatment (anaerobic, facultative, and aerobic treatments), which all depend on the organic strengths of the influent and effluent. Mara and Pearson (1987) indicated the four most important design parameters of WSPs to be temperature, BOD_5 and faecal coliform concentrations, and flow rate of the untreated wastewater.

Von Sperling (2007b), amongst others, presented a summary of the design criteria/parameters for WSP systems, as shown in Table 2.9 below. These design parameters are distinct from those of the baffled ponds and mechanical aerated ponds. The recommended retention time and depth of the pond units are discussed in Section 2.6.

Table 2.9: The typical design parameters for a WSP system

Pond Design Parameters and references	AP			FP					AeP / MP	
Volumetric Loading rate (kg BOD_5 / m^3.d)	0.10 – 0.35	0.04 – 0.3		Not Applicable					Not Applicable	
Length / Breadth (L/B) ratio	1 to 3 (1)		2 to 1	2 to 4		3 to 1		2 to 3 2 to 1	1 – 5 for each pond in series of more than 3 ponds	
BOD removal coefficient K at 20 °C (complete mix) d^{-1}	Not Applicable			0.25 – 0.40	0.1 – 0.3		0.276	0.1 -0.3	Not Applicable	
BOD removal - Temperature coefficient Ө (complete mix)	Not Applicable			1.05 – 1.085	1.05		1.036	1.05	Not Applicable	
Coliform die-off coefficient K_b at 20 °C (complete mix) d^{-1}	Not Applicable			0.4 – 5.0			0.06 - 0.12	2.6	0.6 – 1.2	2.6
Coliform die-off - Temperature coefficient Ө (complete mix)	Not Applicable			1.07			1.03 – 1.04	1.19	1.07	1.19
Reference	von Sperling (2007b), de Souza and Jack (2010)	USEPA (2011)	De Souza and Jack (2010)	von Sperling (2007b)	Ashworth and Skinner (2011)	De Souza and Jack (2010), Ashworth and Skinner (2011) and van Niekerk *et al.* (2009)	USEPA (2011)	Nozaic and Freese (2009)	von Sperling (2007b)	Frederick-van Genderen (1995) and Nozaic and Freese (2009)

(1) Ashworth and Skinner (2011)

2.9.1 Anaerobic treatment

Nozaic and Freese (2009) indicated that the APs do not commonly produce an effluent complying with the regulatory discharge guidelines. The anaerobic treatment units that are normally used in South Africa are the septic tanks, APs or lagoons and aqua privies. The aqua privies are not discussed in the present study.

2.9.1.1 Septic tank

Nozaic and Freese (2009) indicated that the anaerobic treatment process takes place in a septic tank, and the treatment is only partial. The common design parameters of the septic tank are as follows:

- The retention time available for the settlement of solid matter;
- The capacity provided for storage; and
- The partial degradation of the sludge.

The sizing of the septic tanks is determined and based on the frequency of desludging and daily per capita contribution.

Nozaic and Freese (2009) recommended two or three additional compartments, which are smaller than the first compartment to be used for larger tanks, particularly for a community of over 200 people. They also indicated the frequency of desludging of septic tanks as shown in Table 2.10 below.

Table 2.10: The desludging frequency for septic tanks (Nozaic and Freese, 2009)

Population served	Desludging Interval
Single Household	5 to 8 years
10 to 30 persons	2 years
50 to 200 persons	1 year
200 to 500 persons	6 months

The volume of the septic is determined as shown in Equation 2.6 (Nozaic and Freese, 2009 and van Niekerk *et al.*, 2009).

$$V_{ST} = P\,(Q + 0.1\sqrt{S}) \qquad \text{Equation 2.6}$$

Where:

V_{ST} volume of the septic tank, m^3

P contribution population, No.

Q flow per capita per day, $m^3/c/d$

S years between desludging (maximum of 10 years allowed), years

Nozaic and Freese (2009) recommend a length to breadth ratio of 3:1 to 4:1, with a minimum dimension of 0.7 m in any direction, and minimum water depths of 1 m and 2 m for single house units and large populations, respectively.

2.9.1.2 **APs**

Pescod and Mara (1988) noted that APs are cost-effective for the removal of high BOD concentrations and recommended the installation of serial APs when treating high-strength industrial wastewaters. The AP acts like an uncovered septic tank.

The BOD volumetric loading rate, depth, retention time, and geometry, are the main parameters for the design of APs (von Sperling, 2007b). The volume is calculated by using Equation 2.7 below, as advised by (Pescod and Mara, 1988). Frederick-van Genderen (1995), Machibya and Mwanuzi, (2006), Nozaic and Freese (2009), and Ashworth and Skinner (2011) supported the design concept indicated by Equation 2.7 below.

$$V_{ap} = L_i\,Q / \lambda_v \qquad \text{Equation 2.7}$$

Where:

V_{ap} AP volume, m^3

L_i untreated wastewater strength, mg BOD/l

Q average flow, m^3/d

λ_v (B_v) volumetric loading, g/ (m^3 day)

Pescod and Mara (1988) and Nozaic and Freese (2009) both indicated the expected BOD removal for a typical loading, for various retention times, as shown in Table 2.11.

Table 2.11: The BOD removals in APs loaded with 250g BOD/m^3.d (Mara, 1976)

Retention Time (Days)	BOD_5 removals (%)
1	50
2.5	60
5	70

Mara (2003) indicated that the performance of APs increases with temperature and suggested that Table 2.12 be used to provide the design assumptions for BOD removal. De Souza and Jack (2010) cited Mara (2005) in support of the above observation, which agrees with the content indicated in Table 2.12.

Table 2.12: The design values of the volumetric BOD loadings and percentage BOD removals in APs at various temperature (Mara, 2003 and Arthur, 1983)

Temperature (^{0}C)	Volumetric loading (g/m^3day)	BOD removals (%)
<10	100	40
10–20	20 T–100	2T + 20
20–25	10T + 100	2T + 20
>25	350	70

T Mean air temperature in the design month, ^{0}C

The design temperature of APs, including FPs and MPs, is the mean air temperature of the coldest month, as advised by both Mara (2003) and Arthur (1983).

von Sperling (2007b) indicated a sludge accumulation rate between 0.02–0.10 m^3 per capita served per year for APs.

2.9.2 FPs

The surface organic loading rate, depth, retention time, and geometry, are the main parameters for the design of FPs (von Sperling, 2007b; Marais and Shaw, 1961). The types of FPs (primary and secondary) are discussed in detail in the following Sections.

Arthur (1983) advised providing for pond desludging in case where a FP is the primary unit in a series. This can be done by including a pond bypass or constructing parallel units. The author also indicated that a sludge accumulation of approximately 0.04 m^3 per capita served per year, in the primary unit, can be expected. This rate of sludge accumulation is in agreement with that of Mara (2003) who indicating that it is applicable to APs located in warm climates. On the contrary, von Sperling (2007b) indicated a sludge accumulation rate between 0.03–0.09 and 0.03–0.05 m^3 per capita served per year for primary and secondary FPs, respectively.

Pescod and Mara (1988) recommended a depth of 1.5 m, of the water column, to be aerobic at the time of peak radiation and the FPs to be oriented with the longest dimension in the direction of the prevailing wind. The orientation helps to prevent the short circuiting, and in doing so, a maximum mixing of wastewater in the FP can be achieved.

2.9.2.1 **Design methods for the FPs**

Butler *et al*. (2015) and USEPA (2011) indicated four design methods for the FPs, namely loading rate, Gloyna equation, plug flow model, and Marais and Shaw (complete mix kinetics). Adhikar and Fedler (2019) added the Wehner-Wilhelm equation and Thirumurthi application as additional design methods for the FPs. Frederick-van Genderen (1995) highlighted that some of the design formulae used for the FP designs focus on principles that involve the fundamental interactions and relationships between factors, particularly the areal loading, pond depth (geometry), and quantity and quality of the influent domestic wastewater, while others consider the retention time and temperature. These design models tend to fall into one of the three categories, namely empirical, kinetic, and dispersive (Frederick-van Genderen, 1995).

The FP design methods are briefly discussed below.

a) **Areal loading rate**

The areal loading rate is mostly used in the United Stated where a series of detailed evaluations of FP systems were conducted by USEPA to set the design criteria for each location when designing the FP systems (Spellman & Drinan, 2014). The typical surface organic loading rates range from 11 to 90 kg/ (ha.day).

(Butler et al. (2015) highlighted that this method optimizes the organic loading rate to a WSP by examining various factors, namely the volumetric loading, organic constituents in wastewater, ability of algae to use sunlight to grow and supply oxygen, and BOD loading per unit area. Adhikar and Fedler (2019) and Butler et al. (2015) agreed that the use of this method depends on the climate. Adhikar and Fedler (2019) indicated that the design criteria for the areal loading design approach is based on the organic loading or hydraulic retention time for FPs. They also added that the pond dimensions and configurations are not considered in the design of the ponds.

b) **Gloyna equation**

Adhikar and Fedler (2019) indicated that the Gloyna equation or method is a regression equation, which is developed for the design of FPs. Adhikar and Fedler (2019) cited Gloyna (1996) about the proposed empirical equation for pond designs as shown in Equation 2.8.

$$V = (3.5 * 10^{-5}) Q * L_a * \theta^{35-T} * f * f' \qquad \text{Equation 2.8}$$

Where:

V Pond volume, m^3.

Q Influent flow rate, l/d.

L_a Ultimate influent BOD or COD, mg/l.

θ Temperature correction coefficient =1.085.

T pond design temperature, ^{0}C.

f Algal toxicity factor.

f ' Sulfide oxygen demand.

Butler et al. (2015) indicated that this design method is used to determine the volume of a WSP that can maintain a high BOD removal despite the change in temperature. This method has been questioned by the authors because it does not always predict the correct pond depths and BOD removal efficiencies. This agrees with Adhikar and Fedler (2019) who indicated that the Glogna design is based on a depth of 1 m, and that greater depths are recommended for anaerobic conditions. In addition, Marais (1996) noted that the Gloyna equation results in smaller ponds in series, particularly when it is used for the design of pond sizes.

c) **Plug flow model**

This model is derived from first-order kinetics and considers the BOD_5 concentration and reaction rate. The reaction rate is chosen based on the BOD_5 loading rate and pond temperature. The plug flow model is shown in Equation 2.9 below (Adhikar and Fedler, 2019):

$$C_e / C_0 = e^{-k_p t} \qquad \text{Equation 2.9}$$

Where:

C_e Effluent BOD_5 concentration, mg/l.

C_0 Influent BOD_5 concentration, mg/l.

k_p Plug flow first-order reaction rate, $days^{-1}$.

t Hydraulic retention time, days.

e Base of natural logarithms = 2.718282.

Adhikar and Fedler (2019) cited USEPA (1983) on the observation that the selection of the reaction rate is challenging, and it can have a significant effect on pond retention time.

d) **Marais and Shaw model – complete-mix model**

The design method is developed to combine both the first-order kinetics and completely mixed conditions. The design equation is based on BOD, which does not settle as or with sludge and based on the assumptions set out below.

Marais and Shaw (1961) derived the fundamental equations, which are used to determine the concentrations of BOD and faecal bacteria in the ponds. They further looked at the limitation of the theory by performing experimental evidence in ponds in Lusaka (Zambia) and other areas in Southern Africa. In addition, their theory was developed further to investigate the maximum loading on an aerobic pond before the anaerobic condition can be developed.

i. <u>Determination of the 5-day BOD and faecal bacteria concentrations in WSPs</u>

Two assumptions were made by Marais and Shaw (1961) when developing the design theory for the WSP. These assumptions are confirmed in a paper by Marais (1974) and are listed below:

1. The reduction in concentration takes place according to the monomolecular law, which states: "*the rate of change of concentration at any time is directly proportional to that concentration at any time*." Marais (1974) indicated that the reduction in bacteria takes place according to Chick's Law as indicated in Equation 2.10 below.

$$dN / dt = -KN \qquad \text{Equation 2.10}$$

Where:

- N Concentration of faecal organisms per unit volume.
- t Time, days.
- K Decay constant dependent on temperature, day^{-1}.

Marais (1974) suggested that the relationship between K and temperature is to be expressed as indicated by Equation 2.11.

$$K_T = K_{20}\,\theta^{T-20} \qquad \text{Equation 2.11}$$

Where:

K_T and K_{20} decay constants at T ^{0}C and 20 ^{0}C, respectively.

θ Temperature correction coefficient.

2. The mixing of the influent with the pond contents is instantaneous and complete. This means that the concentrations in the pond and pond effluent are identical.

A general differential Equation 2.12, which governs the concentration in the pond was developed based on the increase in the influent concentration, decrease caused by the effluent concentration, and bacterial concentration. The details are mentioned in a paper by Marais and Shaw (1961).

$$\frac{dS}{dt} + \left(K + \frac{Q_2}{V}\right)S = \left(\frac{Q_1}{V}\right)S_0 \qquad \text{Equation 2.12}$$

Where:

S Concentration in the pond and in the effluent from the pond at time, t.

t Time in days measure from any arbitrary instant, d.

K Monomolecular or velocity constant measured in log S-day units.

Q_1 Influent flow to the pond at any time, t, m^3/d.

V Volume of the pond, m^3.

Q_2 Effluent flow from the pond at any time, t, m^3/d.

S_0 Concentration in the influent to the pond, t.

Marais and Shaw (1961) also developed the theorem, which states that the mean value of the concentration (S) in the pond can be determined from the mean value of the influent concentration (S_0), influent flow (Q_1), and effluent flow (Q_2), provided that the values of S_0, Q_1, and Q_2 are daily cyclic.

Under steady state conditions in the WSP, the mean daily values of Q_1, Q_2 , and S_0 must remain constant with respect to time. Also, if sufficient time is allowed after the pond is put in operation, or after the loading is changed, S will become constant. Based on the above theorem, Equation 2.13 was developed for single ponds, for the condition where the retention time is short, and the losses due to evaporation are minor. In this case, the inflow retention (R_1) is almost equal to the outflow retention (R_2). Marais and Shaw (1961) indicated that Equation 2.13 can be used to determine the value of K.

$$S = S_0 / (K R + 1) \qquad \text{Equation 2.13}$$

Where:

R Retention time (V/Q), days.

Marais and Shaw (1961) developed and recommended that Equation 2.14 be used when the retention time of ponds in series is kept constant. Equation 2.14 is also referred to by Adhikar and Fedler (2019) and is based on the completely mixed model and first order reaction rate.

$$S_n = S_0 / (K R + 1)^n \qquad \text{Equation 2.14}$$

Adhikar and Fedler (2019) also indicated that the model should have an upper limit of 55 mg/l as BOD concentration to avoid anaerobic conditions and odours when designing the FPs.

The theoretical relationship was derived by using the K value of 2 to calculate the concentration of faecal bacterial in ponds (Marais and Shaw, 1961). The relationship between the log faecal bacterial reduction and retention time is shown in Figure 2.13 below. Equation 2.15 was used to develop the relationship between the percentage concentration of bacteria in a single pond and the influent concentration, for various retention times.

$$(N / N_0)\% = 100 / (K R + 1) \qquad \text{Equation 2.15}$$

Where:

N Number of bacteria per millilitre in the pond.

N_0 Number of bacteria per millilitre in the pond influent.

Equation 2.16 is used to determine the concentration of bacteria in a series of ponds, where each pond in series has a different bacteria concentration but constant retention time (Marais and Shaw,1961).

$$(N / N_0)\% = 100 / (K R + 1)^n \qquad \text{Equation 2.16}$$

Where:

R Retention time is R_1, R_2, R_3, R_4 to R_n , days.

n n_1, n_2, n_3, n_4 to n_n , No.

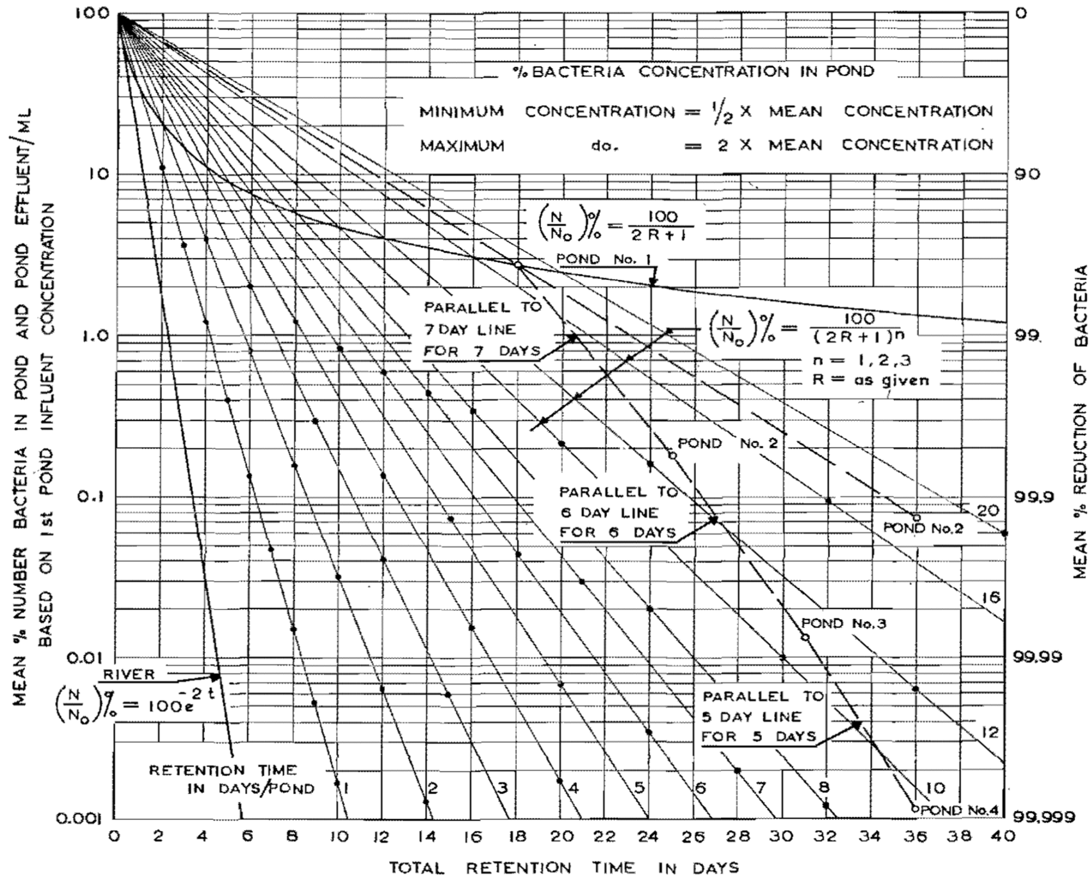


Figure 2.13: The design chart of the reduction in faecal bacteria in a serial configuration of the oxidation and maturation ponds, (K = 2), extract from Marais and Shaw (1961).

Figure 2.13 indicates that a river (plug flow) is the most efficient method for treating wastewater. This is given that it has a higher bacterial reduction for any given retention time, compared with the single pond or ponds connected in series. Marais and Shaw (1961) showed the river to be an ideal situation. In other words, for a retention time of 5 days, the river, series of four ponds, and single pond have mean bacterial reductions of 99.995%, 99.993%, and 90.9%, respectively.

Marais (1974) noted that the most efficient pond configuration is a plug flow condition, followed by the serial configuration, for the total retention time. The total retention time is the sum of the retention time of each pond interconnected in series. The single pond configuration is the most inefficient pond system. Marais (1974) concluded that the plug flow conditions are difficult to be achieved, as the ideal situation hence the number of ponds become very large which is the finite number of ponds in series and R tends to zero the same condition of piston flow, for the design of WSP. The plug flow conditions is not considered in the present study.

ii. Verification of the theory - faecal bacteria concentrations in WSPs and determination of the K value

The measurement of the bacterial concentration was done by Marais and Shaw (1961) for three different groups of faecal indicator organisms, which are total coliform, Escherichia coli (E. coli), and faecal streptococci (F. strep). The degradation constant K values of the different groups were determined by statistics to be 2.13, 2.14, and 2.82, for total coliform, E. coli, and F. strep, respectively.

The validation of the theory was done by testing the results against Equation 2.15 and Equation 2.16. The discussion by Marais and Shaw (1961) was based on the monomolecular constant (K), instantaneous and complete mixing, evaporation losses, daily variations in bacteria, and dispersion. The authors concluded that the experimental data validated the theory when applied to bacterial reduction in a series of ponds.

The theory in Equation 2.19 was tested against the experimental results of the faecal bacterial concentration and BOD, where the degradation constant K was equal to 2. The results indicated a good correlation with the theory, for bacterial reductions in ponds in South Africa and Zambia. Also, there was no decrease in the K value when the retention time was increased (Marais, 1966). Marais (1974) indicated that the long retention times are accompanied by high bacterial reductions. Marais and Shaw (1961) concluded that the experimental data validated the theory.

Marais (1974) investigated the effect of temperature on the K value. This was done by obtaining the reductions of coliforms, FC, and F. strep data over a period of 2 years in ponds in Windhoek in Namibia. The author did several simulations by using Equation 2.12, including the effect of temperature in accordance with Equation 2.11, and the temperature variation in K was found to be in line with

Equation 2.17. The equation indicated a good correlation for coliforms, faecal coliforms, and F. strep data at temperatures between 2–21 ^{0}C.

$$K_T = 2.6\,(1.19)^{T-20} \qquad \text{Equation 2.17}$$

It was noted that the K value is sensitive to temperature. Marais (1974) indicated that the death rate shows a sharp decrease, which indicates a reduction in the K value when the temperature is above 21 ^{0}C.

iii. Verification of the theory - BOD concentrations in WSPs and determination of the K value

Marais and Shaw (1961) verified the theory for the total BOD concentration in the primary pond, which cannot be used for the second pond or for subsequent ponds in series. Hence, the total BOD is assumed to comprise the 5-day BOD concentration, including algae, and some of the BOD is assumed to be consumed by algae.

The evaluation of the experimental data indicated that the mean value of K is equal to 0.23 for ponds in Southern Africa. Marais and Shaw (1961) argued that this K value was high for the primary pond. Then, the recommended design value of K is 0.17, for the BOD, and this value is to be used for retention times of up to 40 days. The authors further argued that the concentrations in the first pond, when the K value of 0.17 and Equation 2.15 are used, are expected to be marginally higher than the experimentally determined concentrations. Marais (1966) noted that the K value decreases as the retention time increases, in the primary pond.

It was also noted that the effluent from the ponds in series was nearly clear of algae, compared with the single pond; the effluent from this latter contained high concentration of algae.

iv. <u>Criterion for determining the aerobic and anaerobic conditions in WSPs</u>

Marais and Shaw (1961) indicated that anaerobic conditions normally arise from overloading, and they also showed that the effective way to stabilize the wastewater is by having anaerobic conditions in a first pond.

The above authors also viewed the re-oxygenation process in a river and in ponds, where the re-oxygenation process in a river takes place by diffusion of atmospheric oxygen into river water. However, in a series of ponds, particularly in the first pond, re-oxygenation takes place through the photosynthesis process of algae (i.e. the oxygen is generated as a bi-product of the photosynthesis). The criterion for aerobic and anaerobic processes in ponds was developed by Marais and Shaw (1961) and is shown in Figure 2.14.

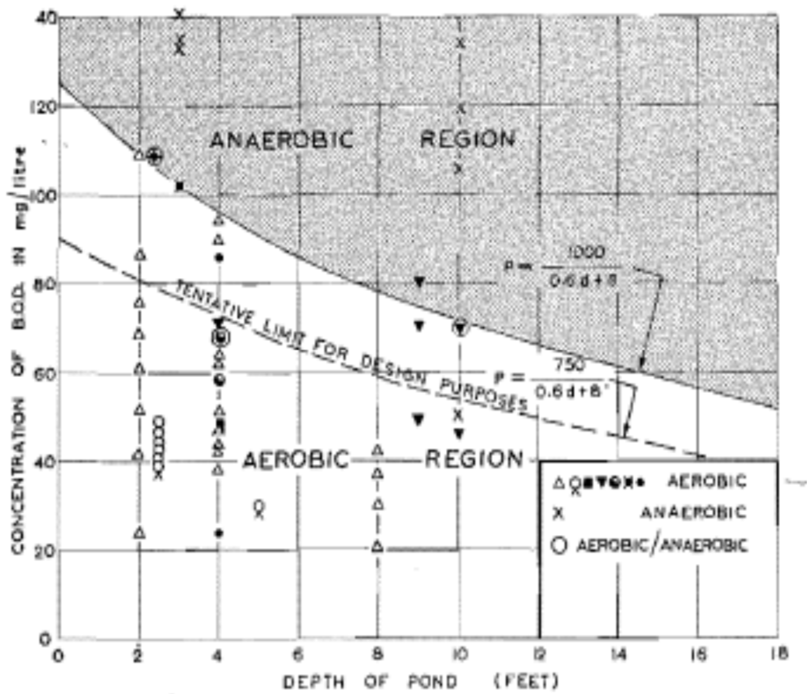


Figure 2.14: The BOD concentration in aerobic ponds for various pond depths. (Source: Marais and Shaw, 1961).

Figure 2.14 shows that there is a boundary line between the aerobic and anaerobic conditions, which depends on the BOD concentration and pond depth. Marais and Shaw (1961) derived the boundary line as shown in Equation 2.18 below:

$$P = P_o / (0.17 R + 1) = 1000 / (0.6d + 8) \qquad \text{Equation 2.18}$$

Where:

d Total depth, feet.

P Total BOD concentration in the pond, mg BOD/l.

P_o. Influent BOD concentration, mg BOD/l.

Marais and Shaw (1961) recommended that Equation 2.18 be used for ponds, which have a depth between 2–10 feet (0.61–3.05 m) and not for pond depths outside this range. It was also noted that for a specific pond depth, there is a BOD concentration that must not be exceeded in order for the pond to remain aerobic. This was also suggested by Marais (1966), indicating that the oxygen production throughout the water body in a pond is limited to the photic zone, which is between 2–6 feet (0.6–1.8 m) water depth.

The BOD concentrations in the pond can vary seasonally, and they can depend on the temperature, which is essentially the solar radiation. Marais (1966) recommended that the mean temperature of the coldest month of the year be used for design purposes to take into account the temperature variations.

Marais and Shaw (1961) advised the use of a safety factor for design purposes, and therefore, Equation 2.18 was adjusted by a factor of 75%, as shown in Equation 2.19 below.

$$P = P_0 / (0.17 R + 1) = 750 / (0.6d + 8) \qquad \text{Equation 2.19}$$

Where:

R Retention time (V/Q), days.

For design purposes, a lower limit for the area over depth ratio was set at no less than 1 000. This was concluded by Marais *et.al*. (1961), given that the area over depth ratio does not have a significant effect on the concentration in the pond. The authors added that the theory should not be applied to pond depths less than 2 feet (0.61 m).

e) **Thirumurthi application – Non -ideal or dispersed flow**

Thirumurthi (1974) modified the work by Marais and Shaw (1974), given that the latter authors do not agree with the assumption of completely mixed ponds, and suggested the use of the chemical reaction. The design method comprises the first-order BOD removal coefficient, correction factor for temperature, correction factor for organic load, and correction factor for toxic chemicals (Butler *et al.* 2015 and Adhikar and Fedler, 2019).

Adhikar and Fedler (2019) cited Thirumurthi (1974) in stating that the Wehner-Wilhelm Equation, shown as Equation 2.20 below, is recommended for the design of FPs, and they developed various charts to be used for ideal plug flow and completely mixed ponds. USEPA (2011) indicated that the non-ideal flow model or dispersed flow model is accepted as being more representative of the flow characteristics of WSPs than either the plug flow model or completely mix model.

$$Ce/_{C_0} = \frac{4ae^{(1/2D)}}{(1+a)^2\left[e^{(a/2D)} - (1-a)^2\, e^{-(a/2D)}\right]} \qquad \text{Equation 2.20}$$

Where:

C_e effluent BOD_5 concentration, mg/l.

C_0 influent BOD_5 concentration, mg/l.

a is equal to $(1 + 4\ kt\ D)^2$.

k first-order reaction rate, $days^{-1}$.

t hydraulic retention time, days.

D dimensionless dispersion number = H / vL = Ht / L^2

H axial dispersion coefficient, area per unit time

v fluid velocity, length per unit time

L Length of the travel path of a typical particle

e base of natural logarithms = 2.718282.

USEPA (2011) and Adhikar and Fedler (2019) recognised that the dispersion numbers or factors (D) identified the type of flow to be either plug flow or completely mixed, where D is equal to zero for plug flow and equal to infinity for completely mixed. Frederick-van Genderen (1995) cited Metcalf and Eddy (1972) by indicating that the WSPs have a D value that varies between 0.1–2.0. Adhikar and Fedler (2019) and USEPA (2011) also referred to the same range of D values, but USEPA (2011) indicated that the measured D values in WSPs are commonly found to be less than 1.0.

2.9.2.2 **Primary FPs**

Primary FPs are ponds that receive the untreated wastewater. The literature indicated different design approaches and timelines for the development of the Equations. McGarry and Pescod introduced the design method for FPs as indicated in Equation 2.21 below.

$$\lambda = 11.2(1.054)^{T}$$ Equation 2.21

Where:

λ maximum BOD loading, kg/ ha/ day

T temperature, °C

The literature indicated that primary FPs are not designed to operate to the point of failure, hence, Nozaic and Freese (2009) advised the introduction of a safety factor for design purposes, and Equation 2.21 was modified to Equation 2.22 below.

$$\lambda_d = 7.5\ (1.054)^{T}$$ Equation 2.22

Where:

λ_d design loading, kg/ ha/ day

Pescod and Mara (1988) indicated that Mara modified Equation 2.22 in 1976 by suggesting a linear design equation (straight line relationship), as indicated by Equation 2.23 below.

$$\lambda_d = 20\,T - 120 \qquad \text{Equation 2.23}$$

Arthur (1983) recommended Equation 2.24 to provide a best fit with the available data.

$$\lambda_s = 20\,T - 60 \qquad \text{Equation 2.24}$$

Where:

λ_s areal loading rate, kg BOD_5 / (ha day)

Thus, the design equation for the primary FP area becomes Equation 2.25, which is obtained by substituting Equation 2.23 in Equation 2.28.

$$A_f = L_i\,Q / (2T - 12) \qquad \text{Equation 2.25}$$

Figure 2.15 indicates the design alternatives of the primary FP when using Equation 2.23 and Equation 2.24 in relation to the changes in temperature.

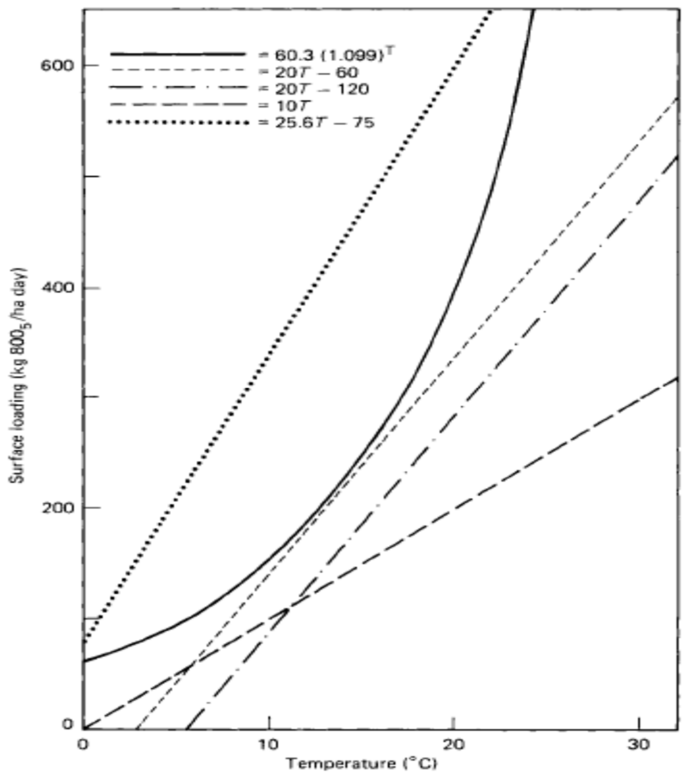


Figure 2.15: The comparison between the alternative design equations for FPs, extract from Mara and Pearson (1987), and Pescod and Mara (1988).

Mara developed an empirical equation in his 1987 publication as shown in Equation 2.26 below (Nozaic and Freese, 2009).

$$\lambda_s = 350\,(1.107 - 0.002T)^{T-25} \qquad \text{Equation 2.26}$$

Mara (2003) indicated that Equation 2.26 is based on a loading of 80 kg/ ha/ day at less than 8°C in European winters, a loading of 350 kg/ ha/ day at 25°C in northeast Brazil, and an arbitrary loading of 500 kg/ ha/ day at 35°C.

Nozaic and Freese (2009) recommended the Marais design sequence given that it is a more conservative design for primary FPs. The Marais design sequence is shown in Equation 2.27 below and discussed above in detail under sub-section about the Marais and Shaw model which is the complete-mix model.

$$R = (P_o / P_e - 1) / K_e \qquad \text{Equation 2.27}$$

Where:

R Pond Retention, days.

P_e Effluent BOD concentration, mg BOD/l.

P_o Influent BOD concentration, mg BOD/l.

K_e Equivalence rate constant = 0.11 (for variation in the range 10–32 ^{0}C).

Nozaic and Freese (2009) concluded that the Marais sequence, McGarry and Pescod curve, and Mara global design empirical equation are all proving very similar results and probably give the best pond design (sizing). These models are discussed in` Chapter 3 (Materials and Methods) and compared quantitively in Chapter 4 (Results and Discussion).

2.9.2.3 **Secondary FPs**

Secondary FPs are ponds that receive the treated wastewater from APs. FPs are designed based on the surface BOD loading parameters to correlate with the photosynthesis area of algae. The pond area is calculated as per Equation 2.28 below (Ashworth and Skinner, 2011).

$$A_f = 10\, L_i\, Q / \lambda_s \qquad \text{Equation 2.28}$$

Where:

A_f FP mid depth area, m^2

L_i untreated wastewater strength, mg BOD/l

Q average flow, m^3/d

λ_s surface loading, kg/ ha/ day

Ashworth and Skinner (2011) calculated the BOD removal by using Equation 2.29. It is similar to Equation 2.18 and Equation 2.27 as proposed by Marais *et.al.* (1961) and Nozaic *et.al*. (2009), respectively. The only difference is the degradation constant *K*, which takes different values, as indicated in Equation 2.27 and Equation 2.29 in Section 2.9.2.1d).

$$L_e = L_i / (1 + k_{1T}\, \theta_f) \qquad \text{Equation 2.29}$$

Where:

L_e unfiltered effluent strength, mg BOD/l

L_i untreated wastewater strength, mg BOD/l

k_{1T} first order rate constant, day^{-1}

θ_f Retention time, day

The first order rate constant is calculated as indicated by Equation 2.30 below.

$$k_{1T} = k_{1(20)}\, (1.05)^{(T-20)} \qquad \text{Equation 2.30}$$

Where:

$k_{1(20)}$ facultative pond first order constant, day^{-1}

T mean air temperature in the design month, ^0C

Ashworth and Skinner (2011) tabulated the surface loading and BOD removal rates of both the primary and secondary FPs as shown in Table 2.13. They also recommended the values of $k_{1(20)}$ to be 0.3 and 0.1 for the primary and secondary FPs, respectively. The values of the FP first order constant were also in agreement with those of Mara (2003).

Table 2.13: The FP surface loading and BOD removal (Ashworth and Skinner, 2011)

FP	Surface loading kg BOD/ (ha day)	BOD removal %
Primary FP	$350\ (1.107 - 0.002T)^{T-25}$	$100\ (0.3\theta_f) / (1 + 0.3\theta_f)$
Secondary FP	$350\ (1.107 - 0.002T)^{T-25}$	$100\ (0.1\theta_f) / (1 + 0.1\theta_f)$

Frederick-van Genderen (1995) and von Sperling (2007b) agreed with the surface loading rate equations indicated in Table 2.13 above and also recommended that the maximum value of 350 kg BOD / (ha day) be used as the limit for design purposes. The loading of 350 kg/ (ha/ day) is also in agreement with that reported by Mara (2003).

2.9.3 MPs

The effluent from the FP is treated in the MP. MPs are used as the second stages of FPs, and secondary ponds, for COD reduction (Nozaic and Freese, 2009). Nozaic and Freese (2009) also indicated that placing two MPs in series, each with a retention time of 7 days, can be used to treat an influent BOD of not more than 75 mg/l to produce an effluent BOD of less than 25 mg/l. Furthermore, the authors highlighted the fact that the COD is not a good indicator of the pond effluent quality as the prevalence of algae tend to give rise to high COD readings. The authors recommended measuring the COD on a filtered sample when testing the pond effluent.

Nozaic and Freese (2009) and Frederick-van Genderen (1995) found that the reduction in the faecal bacteria of a pond (anaerobic, facultative or maturation) followed the first order kinetics and recommended Equation 2.31 to be used. Equation 2.31 is similar to Equation 2.15 developed by Marais (1974) in Section 2.9.2.1d).

$$N_e = N_i / (1 + K_b\, t) \qquad \text{Equation 2.31}$$

Where:

N_e	Number of effluent E. coli/100ml
N_i	Number of influent E. coli/100ml
K_b	First order rate constant for FC removal, day^{-1}
t (R)	Retention time, day

The K_b variable is found to be temperature sensitive (Nozaic and Freese, 2009), and this also agrees with the observation by Marais (1974), see Equation 2.32, which is similar to Equation 2.17 developed in Section 2.9.2.1d).

$$K_b = 2.6\,(1.19)^{T-20} \qquad \text{Equation 2.32}$$

Where:

T	Mean air temperature in the design month, ^0C

Nozaic and Freese (2009) suggested the design value of 4 x 10^7 FC/100 ml, for N_i, to determine N_e in Equation 2.31. A trial-and-error method is used to determine the retention time in order to achieve the acceptable N_e of the regulatory discharge limits.

Frederick-van Genderen (1995), Machibya and Mwanuzi, (2006) and Nozaic and Freese (2009) indicated that the reduction in faecal bacteria at the effluent of ponds, installed in series, can be calculated as per Equation 2.33.

$$N_e = N_i / \{(1 + K_b\, t_1)\,(1 + K_b\, t_2)\,(1 + K_b\, t_n)\} \qquad \text{Equation 2.33}$$

Where:

t*	Retention time in the nth pond, day

2.9.4 Operation and Maintenance

Pescod and Mara (1988) recommended at least two sequences or arrangements of SWPs in parallel to be incorporated in the design for easy maintenance and flexibility of operation of WSPs. They added that the designer should ensure that there is an access around the ponds. Also, all ponds should have access, at least on one side, by cranes and heavy machines. They recommended at least 10 m of access on one side and 5 m on other sides.

Arthur (1983) mentioned that the omission of the maintenance can cause mosquitoes, flies, and odour nuisances. The author recommended the regular maintenance to assist in maintaining the high effluent guideline and to avoid the above-mentioned nuisances. The author summarised the maintenance duties as follows:

- Regular cleaning and clearing of both the screens and grit removers at the manual bar screens and grit channels, respectively.
- Cutting or mowing of grass and vegetation at the pond embankments.
- Removal of scum from FPs.
- Solid accumulations at the inlets and outlets of the ponds to be avoided.
- Sludge removal.

The routine maintance of the ponds is in agreement with Sexauer and Karn (2013) who also added methods for controlling erosion and rodents (burrowing animals).

Arthur (1983) and Sexauer and Karn (2013) also advised on the treatment monitoring, which includes the regular monitoring of flow rates and influent and effluent qualities. This will assist in the monitoring of the effluent qualities, measurements of water losses, and calculations of both the hydraulic and organic loading rates.

2.10 MODELLING OF WSPs

Ho *et al*. (2017) stated that a better design of WSPs is required because of the growing application of wastewater purification, strict environmental regulations, and the fact that most of the previous design manuals are outdated. The authors indicated that most of the design guidelines for WSPs focused on data-based approaches, namely the rule of thumb or regression equations, to determine the dimensions of the pond treatment system.

The model-based approaches for WSP designs, indicated by Ho *et al*. (2017), are as follows:

1. The rule of thumb: this design approach is also called black box approach. The loading rate and hydraulic retention time are used as design criteria for APs, FPs, and MPs. The authors summarized the commonly used loading rates and hydraulic retention times as found in the literature.

2. The regression equations: these are based on an inductive methodology whereby the observed data are used to build empirical relationships between a dependant variable and covariates. The authors also indicated the four main parameters used to assess the performance of APs, namely organic removal efficiency, sludge accumulation, pathogen removal efficiency, and biogas production.

Ho *et al*. (2017) further indicated three well known regression equations that were used to determine the surface loading rate of FPs as follows:

i. The Arceivala equation, which focuses on the role of local climatic conditions, particularly sunlight and temperature, in the organic removal efficiency;
ii. The McGarry and Pescod equation, which associates higher ambient temperatures with higher surface loading. This equation is discussed in Section 2.9.2.2, where the maximum BOD loading is expressed by using in in Equation 2.21; and
iii. The Gloyna method, which includes factors, namely algal toxicity, solar radiation, and sulphate ion concentration, to evaluate both the organic matter removal and odour problems. This method is discussed in Section 2.9.2.2, where the pond volume is calculated by using Equation 2.8.

3. The first order models: these are process-based models that simulate and quantify the effect of the removal processes in WSPs based on their kinetics. The reaction rate of chemical and biological processes is assumed to follow the first-order kinetics. The reaction rate coefficient (k) and dispersion number (d) are regarded as the important factors in the design and performance evaluation of WSPs in these models. Ho *et al*. (2017) also highlighted the three options of flow patterns that are applicable to WSPs. These options are the dispersion model, modified dispersion model, and plug flow model. The authors also stated that the dispersion models accurately predicted the total and faecal coliform die-off.

4. The mechanistic models: these models depend on the conceptualisation and determination of the biochemical and physical mechanisms in the WSPs. Ho *et al*. (2017) and Adhikar and Fedler (2019) indicated the three types of mechanistic models as follows:

    i. The Computational Fluid Dynamics (CFD) models: these models were used for over 2 decades, and they started with 2D CFD, which further developed into 3D models. The 3D models incorporated external factors, namely wind, thermal stratification, and baffling, which impact the performance of WSPs. The chronological evolution of the CFD applications on WSPs are shown in Figure 2.16.
    ii. The biokinetic models: the kinetic rate for these models is based on the Monod equation for which the reaction rate depends on the substrate concentrations, organism concentrations, and interactions between the organisms and the environment. The Activated Sludge Models (ASMs: Henze *et.al*., 2000) and Anaerobic Digestion Model 1 (ADM1; Batstone *et al*., 2002) were initially used to perform the biokinetic modelling. These two methods were followed by the development of compatible biokinetic models, namely the river water quality model (RWQM) and constructed wetland model (CWM1), to simulate natural systems. The ASM3 model was also developed to simulate the integration of algae biomass, influences of sun and wind, and strippings of ammonia and carbon dioxide in the FPs and MPs.

iii. The integrated models: these are 3D semi-dynamic models based on the concepts of ASMs and ADM1. The effects of wind directions and baffles on the flow characteristics, including the effluent qualities of WSPs, were evaluated using these models.

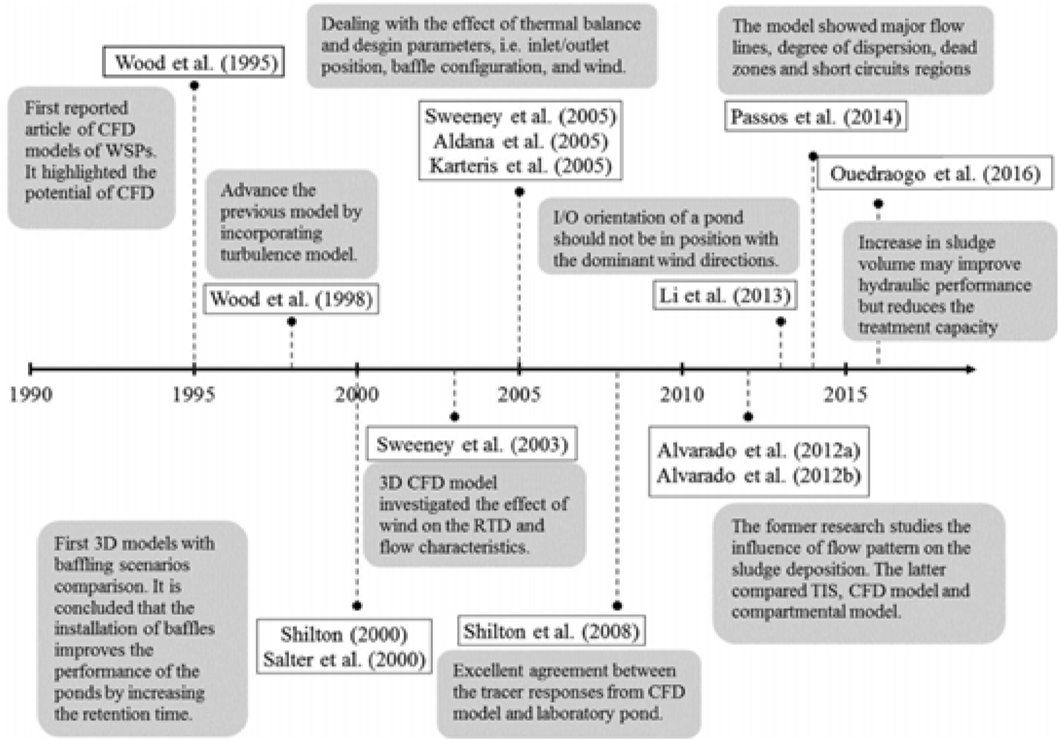


Figure 2.16: The chronological evolution of the main CFD models used in the simulations of WSPs, extract from Ho *et al.* (2017).

Ho *et al*. (2017) indicated the required number of data, i.e. the inputs for each model type, and corresponding degree of design specification, for each model type, as shown in Figure 2.17. The authors added that the mechanistic models, especially the integrated models, appeared to be the comprehensive and multi-disciplinary tools that can deliver useful informations for detailed design or process optimization. The data-based models (mechanistic models) should imitate the aquatic ecosystems to purify the contaminated water as suggested by Ho *et al*., 2017 and should be based on the complex interactions between the bio-geochemical cycles and an interconnected web of biochemical processes and reactions that occur in WSPs. Furthermore, the authors stated that there were few mechanistic models that were calibrated due to the lack of data availability and collection.

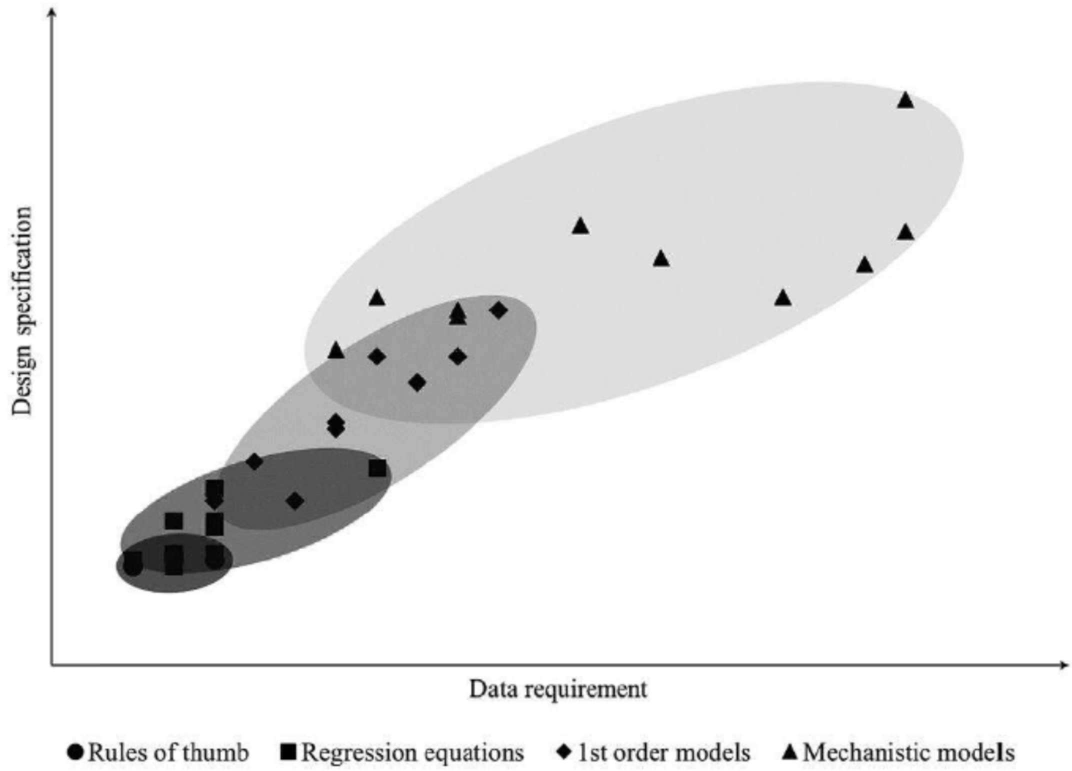


Figure 2.17: The data required for each model type and the level of design specification, extract from Ho *et al.* (2017).

Venter (1996) stated that the MINLAKE model can simulate both the hydrodynamic and water quality behaviours. The modified MINLAKE model was used by Venter (1996) to simulate the Roodeplat Dam, the model has variables, which are simulated from the surface to the bottom of the reservoir. These variables include the following:

- Water temperature
- Mixing depth
- Chlorophyll-a (up to three algal classes)
- Dissolved phosphate
- Dissolved oxygen
- Total inorganic suspended sediments (TSS)
- Nitrate and ammonia
- Total dissolved salts (TDS)

- Water level (stage)
- Soluble detritus
- Particulate detritus
- Zooplankton

The first seven parameters, namely water temperature, mixing depth, chlorophyll-a, dissolved phosphate and oxygen, TSS, and nitrate and ammonia, are used to mimic the condition of the WSP, which implies that the modified MINLAKE model can be used to model the WSP. In addition, Venter (1996) stated that the modified MINLAKE is the only dynamic water quality model calibrated for a South African reservoir that can simulate more than one algal class.
Von Sperling (2007a) indicated that the river quality models have been used since 1925 and have represented a milestone in water and environmental engineering. These models were developed by increasing the level of complexity and number of state and input variables. von Sperling (2007a) also presented ten models and compared them in terms of the required quality parameters. The comparison of the models was done against the nine quality parameters, namely the temperature, bacteria, DO-BOD, Nitrogen, phosphorus, silica, phytoplankthon, zooplankton, and benthic algae.

In the Materials and Methods and Results Sections of the present study, a comparison of the four models is presented. The selected models are the Marais sequence (first order kinetics), McGarry and Pescod curves, and Mara global design empirical equation, which have been designated as the design methods for the WSPs. The comparison between the design models for ponds will assist the process designers in the early stages of projects, particularly during the feasibility study stages. Likewise, in the operation and maintenance stages, the model must be easy to use to check the performance of the pond. This will also assist the operators in determining the pond overload time and subsequent desludging required.

The validation of the design models and the water quality data for ponds at selected periods, or during the lifetime of the WSP, must be done, including checking the performance of the existing ponds. For the present study, this will be achieved by using the same input data, parameters, including but not limited to, temperature, Chemical

Oxygen Demand (COD), Biochemical Oxygen Demand (BOD), and Number of E. coli. and FC. Subsequently, the effluent results and pond sizes will be compared.

2.11 EFFLUENT DISPOSAL AND DISCHARGE GUIDELINES

Nozaic *et al*. (2009) indicated that the discharge or irrigation of wastewater to a water course is controlled and governed by government legislation. WSPs in South Africa are required to treat the wastewater to comply with the general limit as prescribed by the DWS (see Table 2.14). Other countries are using the World Health Organisation (WHO) guidelines.

Table 2.14: The effluent limit guidelines of treated wastewater

Parameters	**DWS guidelines**		**WHO guidelines**
	Irrigation	**Discharge to water source**	
Faecal coliforms	<1 000/100 ml	<1 000/100 ml	<1 000/100 ml
COD		<75 mg/l	<100 mg/l
pH	6 – 9	5,5 – 9,5	5,5 – 9,0
Ammonia (NH_4 – N)		<6 mg/l	
Nitrate/nitrite (as N)		<15 mg/l	
Suspended Solids		<25 mg/l	<100 mg/l
Ortho-phosphates (as P)		<10 mg/l	
Free Chlorine		<0,25 mg/l	
Conductivity increase	<200 mS/m	70 mS/m above intake to a maximum of 150 mS/m	
BOD			<30 mg/l
TSS		<25 mg/l	<30 mg/l
Fluoride		<1 mg/l	
Soap, oil or grease		<2,5 mg/l	

The DWS also stated the monitoring requirements for domestic wastewater discharges where selected parameters must be measured depending to the plant size or discharge volume.

2.12 POPULATION FORECAST – QUANTITY PARAMETERS

Population forecasting is referred to as a method used to predict or forecast the future population of the study area. The size of the current population is used together with the population change, which are the numbers of births and deaths occurring between the date of the population census and the date of the forecasting (Aryal, 2020).

Von Sperling (2007a) presented eight methods for calculating the population forecast. These methods are based on the mathematical formulae and indirect quantification as shown in Table 2.15 and Table 2.16, respectively. On the other hand, Aryal (2020) and Department of Economic and Social Affairs (DESA, 1956) presented the mathematical method, economic method, and Cohort or Growth component method as the three methods that are used for population projections. Aryal (2020) indicated eight mathematical methods viz. linear growth model, geometric growth model, exponential growth model, Gompertz Curve, modified exponential function, Makeham's Curve, Polynomial of Degree and Logistic Growth model.

Table 2.15: The method based on the mathematical formulae when forecasting the population (von Sperling, 2007a and Aryal, 2020)

Item No.	Method	Description	Function/ Use
1	Linear growth	The population growth follows a constant rate	It is used for a short-term forecast.
2	Geometric growth	The population growth is a function of the existing population at any time.	It is used for a short-term forecast.
3	Multiplicative regression	The population growth is fitted by linear regression (logarithmic transformation equation) or non-linear regression.	
4	Decreasing growth rate	The population growth is based on the assumption that when the town grows or expand, the growth rate becomes lower. The coefficients can be estimated by non-linear regression.	
5	Logistic growth	The population growth follows an S-shaped curve. The coefficients can be estimated by non-linear regression.	

Table 2.16: The method based on indirect quantification when forecasting the population (von Sperling, 2007a)

Item No.	Method	Description and Use
1	Graphical comparison	This method uses the graphical fitting of the previous population under study, where the population data of the other similar town are plotted so that the curves coincide or match at the current value of the population of the town under study.
2	Ratio and correlation	This method assumes that the town population follows the same trend as the region in which it is inserted. This is based on the census records where the ratio of town population over the region population is used to project the future population.
3	Forecast of employment and utility services	This method is based on the past population data and people employed, the ratio of job over population is calculated and projected for future years.

The geometric growth method for forecasting the population is recommended by von Sperling (2007a) and Umara, *et al.* (2010). This growth method is used in Section 3.3.2, and hence, the existing population is easier to be obtained, and the population is assumed to constantly increase by numbers proportionate to its varying size.

2.13 CONCLUSIVE SUMMARY

WSPs in South Africa are required to treat wastewater to comply with the discharge limit guidelines prescribed by the DWS. The process designers of WSPs require to consider, among others, the influent loading and climate (i.e. temperature, wind, and others), and geometry of the WSP when designing the capacity of or modelling WSPs. The degradation constant K value varies with temperature, it decreases when the temperature is above 21 °C, and it is also found to decrease as the retention time of the primary pond increases.

Nozaic and Freese (2009) and Butler *et al*. (2015) indicated that APs can remove approximately 60 % of the influent BOD, and this plays an important role in reducing the required land size or area of the WSP. It is vital to include the anaerobic unit in the WSP system, particularly when dealing with heavily polluted industrial wastes (Pescod & Mara, 1988). The present study will investigate the benefits of including the AP in the WSP system by modelling both the AFM and FM systems, where the results from the four models, namely the Marais sequence, First order kinetics, McGarry and Pescod curve, and Mara global design empirical equation, will be compared in terms of unit size and effluent water quality.

Table 2.17 provides the recommended design parameters and pond geometry to be used for the WSP, which will be used to perform the analyses during both the Scenario 1 and 2 stages, as detailed in Section 3.6 of the Materials and Methods Chapter.

Marais and Shaw (1961) indicated that the depth of the pond should be between 0.6–1.8 m, and particularly the minimum depth should be 1m, to maintain the aerobic bacteria and ensure bacteria reduction is achieved. Figure 2.14 indicated that the influent loading plays a significant role in determining the above-mentioned minimum depth to maintain the aerobic zone.

Table 2.17: The recommended design parameters for WSPs in EC

Parameter	Minimum Depth (m)		Maximum Depth (m)		Volumetric BOD loading (g BOD/ m^3 d)		Surface loading (kg / ha. d)		BOD removal (%)		Minimum Retention (days)		Maximum Retention (days)	
Pond Unit	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**
AP	2	sect. 2.6, Table 2.3	5	sect. 2.6, Table 2.3	100–400	sect. 2.6.1.3	N/A		60–85	sect. 2.6.1.4, Table 2.11, Table 2.12,	1	Table 2.3, Table 2.11	4	Table 2.3, Table 2.11
Septic Tank	1	sect. 2.9.1.1	2	sect. 2.9.1.1					30–35	sect. 2.6, Table 2.3				
Facultative Pond (FP)									75 - 85	sect. 2.7, Table 2.4				
Primary FP	2	sect. 2.6, Table 2.3	3	sect. 2.6, Table 2.3	N/A		100–400	sect. 2.6, Table 2.3	75 - 85	sect. 2.7, Table 2.4	30	sect. 2.6, Table 2.3	50	sect. 2.6, Table 2.3
Secondary FP	1	sect. 2.6, Table 2.3	2	sect. 2.6, Table 2.3	N/A				75 - 85	sect. 2.7, Table 2.4	10	sect. 2.6, Table 2.3	15	
MP	1	sect. 2.6, Table 2.3	1.3	sect. 2.6, Table 2.3	N/A				20 – 25	sect.2.6.4.2	3	sect. 2.6, Table 2.3	6	sect. 2.6, Table 2.3

Parameter	L/B Ratio (min)		L/B Ratio (max)		Minimum dimension (m)		Sludge accumulation rate (m^3/capita/year)		Internal embankment slope	External embankment slope
Pond Unit	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**	**Quantity**	**Refer to Lit Review**		
AP	1 :1	sect. 2.9, Table 2.9	3:1	sect. 2.9, Table 2.9			0.02 -0.1	sect. 2.9.1.22.9.1	1:3	1:1.5 to 1:2
Septic Tank	3:1	sect. 2.9.1.1	4:1	sect. 2.9.1.1	0.7	sect. 2.9.1.1	0.015 – 0.035	sect. 2.6, Table 2.5		
Facultative Pond (FP)	2:1	sect. 2.6.2.1	3:1	sect. 2.6.2.1			0.015 – 0.04	sect. 2.6, Table 2.5, sect. 2.9.2		1:1.5 to 1:2
Primary FP	2:1	sect. 2.6.2.1	1:1	sect. 2.6.2.1			0.03 – 0.09	sect. 2.9.2		
Secondary FP	4:1		1:20				0.03 – 0.05	sect. 2.9.2	1:3	1:1.5 to 1:2
MP	1:1	sect. 2.9, Table 2.9	5:1	sect. 2.9, Table 2.9					1:3	1:1.5 to 1:2

Section = sect.

Literature = Lit

Marais (1966 and 1974), Ukpong, Agunwamba and Egbuniwe (2006), Nameche and Vasel (1998) indicated that the effect of stratification, mixing, wind, and temperature should not be ignored during the process of designing the WSP. Thermal stratification influences the distribution of parameters, namely DO, pHs, BOD, COD, NH_4, N, TP, and coliform bacteria, in the WSP system. This should be considered for Scenario 1 as it can impact the interpretation of the laboratory results and lead to erroneous WSP sizing or design modelling. In addition to the above, the laboratory company responsible for water sampling must be accredited with the South African National Accreditation System (SANAS), in terms of the SABS Code 0259, and the samples taken by the registered professional personnel or practitioner (Nozaic and Freese, 2009 and National Water Act No.36 of 1998, 1998).

There are different methods and models that can be used for the design of a WSP. The designer and/or operator must choose the appropriate design method or technique by taking into consideration the assumptions and limitations of the design method, pond type, and influent wastewater quality, including the required effluent guidelines as discussed in Sections 2.6, 2.9, and 2.10. In addition, there should be consideration of the mechanistic models, especially the integrated models, which are found to be the comprehensive tools for the detail design or process optimization of a WSP.

CHAPTER 3: MATERIALS AND METHODS

3.1 INTRODUCTION

Seven sites in the Eastern Cape (EC) Province were selected for the present study, and the location of the sites are shown in Figure 3.1 below. These sites are the 14 South African Infantry (SAI) Battalion, Bedford Hospital, Mthatha Prisons, St Patricks School, Marselle township, Fort Cox College (FCC), and Tsolo Agricultural and Rural Development Institute (TARDI). The water quality data was collected by three distinct organisations/groups (i.e. Local Authorities, Department of Water and Sanitation (DWS), and independent or private laboratories) during the years 2016 up to 2021.

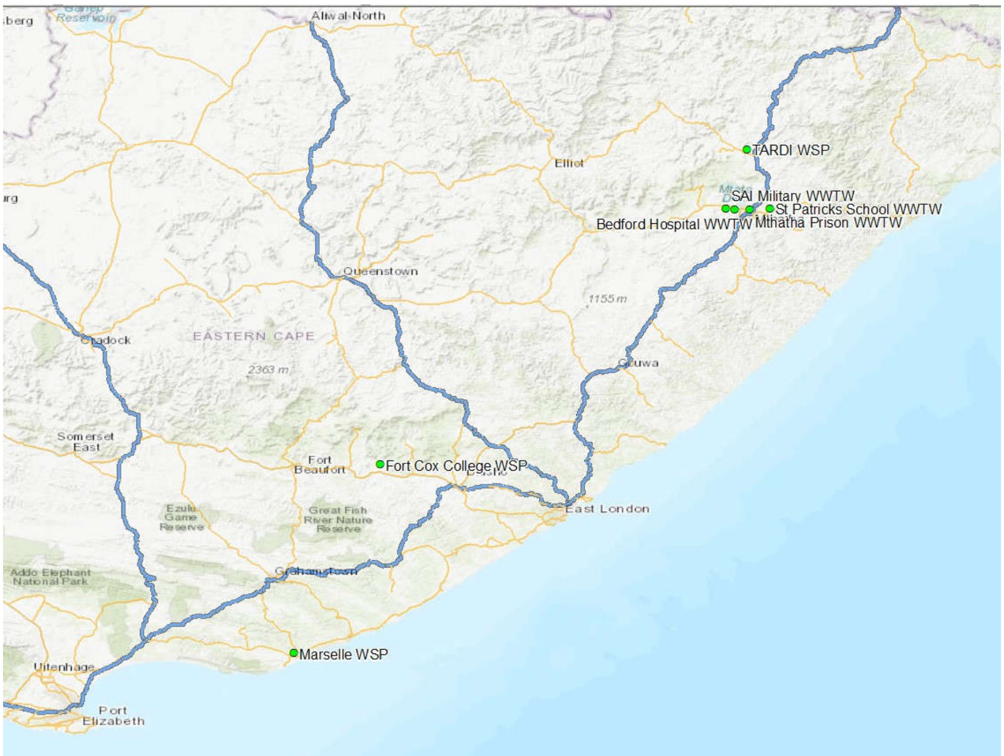


Figure 3.1: The map indicating the location of the seven sites.

The study area was narrowed down from seven sites to three sites viz. Marselle township, FCC, and TARDI. Since, among other things, the geometry of the ponds, and other parameters as indicated in Section 3.2, were unknown, the selection of the study area was further refined to the TARDI.

In April of the year 2017, the DWS of the EC Province provided the effluent quality data taken at the four Waste Stabilisation Ponds (WSPs) around the area of Mthatha, in the Eastern Cape Province, as shown in Figure A.2 of Appendix A. The samples were only taken at the effluent of the last pond to monitor the water quality being discharged to the existing rivers. The sample results data were then compared with the DWS Guidelines, as shown in Table A.1 of Appendix A.

Additional typical wastewater data, from three sites in the Eastern Cape Province, namely Marselle township, FCC, and TARDI, were also collected from one municipality (Ndlambe Local Municipality) and two institutions (FCC and TARDI). The results of the water in-situ tests at influent of the WSP systems, from the seven sites, were used to compare with the concentration of the typical domestic water quality indicated in Table 2.2 under Section 2.4. The detailed comparison was done and presented in Table 3.3 for TARDI only due to the financial and time constraints of the present study area. Two sites viz. FCC and TARDI out of the seven sites was found to have the record of both the influent and effluent water quality tests record. The effluent water quality of the WSP was used solely to check if the effluent met the requirements of the DWS. TARDI was selected, out of the seven WSP sites, given that its WSP had the measurable parameters and characteristics of the WSP as discussed in detail in Section 3.2.

The subsequent sections will discuss the required parameters, measured parameters, sampling points, sample dates, adopted data analysis method, and problems experienced during the data collection. The list of assumptions, including the design criteria and approach used for the purposes of sizing the WSP for the case study at TARDI, are also discussed.

3.2 PARAMETERS REQUIRED TO CARRY OUT THE STUDY

The following tools/parameters were required to carry out the present study:

- Wastewater flow data at the inlets and outlets of the pond units (influent and effluent flow data).
- Wastewater concentrations at the inlets and outlets of the pond units.
- Existing pond geometry, i.e. depth, bottom and top widths, bottom and top lengths, side slopes, water depths, and others.
- Design capacity of the existing WSP.
- Environmental data, including temperature, wind speed, and others, obtainable from a weather station.

3.3 CASE STUDY (STUDY AREA)

TARDI has a wind measuring equipment installed approximately 500 m away from the location of the WSP, which can be used to determine the effect of wind on the WSP. Due to the unavailability of data, assumptions were made for Scenarios 1 and 2 stages (i.e. feasibility and conceptual design studies), which are discussed in Section 3.8 below.

The subsequent Section provides details of the location, population, water and sanitation infrastructures, and design flow for TARDI.

3.3.1 Site description

TARDI is located approximately 45 km from the Mthatha Central Business Centre (CBD). The access to the site is from road R396, which is the main road between the N2 and R56 surfaced roads. The geographic location of the study area is approximately 31° 17' 37" latitude and 28° 45' 49" longitude. A locality plan for the study area is indicated in Figure 3.2.

Figure 3.2: The locality plan of the TARDI WSP.

3.3.2 Population

The TARDI provided the information on the land use and building occupancy, including the number of livestock, students, and others. The information is shown in Figure 3.3 below. Equation 3.1 below, as suggested by von Sperling (2007a) and Umara, *et al.* (2010), was used to forecast the population, and the growth rate of 0.5% over 20 years was assumed, hence, the information regarding the future upgrades was also provided by the TARDI and the expansion of the institution was anticipated to be minimal or negligible. The forecast method used for the study area was based on the geometric growth method, which is the mathematical method discussed briefly in Section 2.12.

$$P_N = P_0 (1+r)^N \qquad \text{Equation 3.1}$$

Where

P_N	population of Nth year, c.
P_0	present population (base population) , c.
r	rate of population growth, %.
N	period, years.

3.3.3 Water Resources and Associated Infrastructure

TARDI has two water sources, namely the Xokonxa River, which is a surface water source, and 2 boreholes (BHs), which are ground water sources. The study area is served by two BHs located near the Xokonxa river. The existing boreholes are approximately 300 m apart, as shown in Figure 3.3. TARDI is currently using the borehole water for domestic and livestock purposes.

To verify the aquifer pump testing data, the yield tests of the two existing boreholes, including the water quality tests, were done at TARDI. The water is pumped from the boreholes to the existing reservoirs, from where it is conveyed, by gravity, to the internal reticulation. The reticulated water is delivered to the study area via the existing standpipes, yard taps, and in-house connections.

DESCRIPTION OF ALPHABETS	
A	STAFF RESIDENCE
B	STAFF RESIDENCE
C	STAFF RESIDENCE
D	STAFF RESIDENCE
E	STAFF RESIDENCE
F	STAFF RESIDENCE
G	STAFF RESIDENCE
H	STAFF RESIDENCE
K	STAFF RESIDENCE
L	STAFF RESIDENCE
M	STAFF RESIDENCE
N1	STAFF RESIDENCE
N2	STAFF RESIDENCE
O1	STAFF RESIDENCE
O2	STAFF RESIDENCE
O3	STAFF RESIDENCE
P1	STAFF RESIDENCE
P2	STAFF RESIDENCE
P3	STAFF RESIDENCE
P4	STAFF RESIDENCE
Q1	STAFF RESIDENCE
Q1	STAFF RESIDENCE
Q3	STAFF RESIDENCE
Q4	STAFF RESIDENCE
R1	STAFF RESIDENCE
R2	STAFF RESIDENCE
R3	STAFF RESIDENCE
R4	STAFF RESIDENCE
S1	STAFF RESIDENCE
S2	STAFF RESIDENCE
T1	STAFF RESIDENCE
T2	STAFF RESIDENCE
T3	STAFF RESIDENCE
U1	STAFF RESIDENCE
U2	STAFF RESIDENCE
V1	STAFF RESIDENCE
V2	STAFF RESIDENCE
W	ELEVATED STEEL TANK
X	RESERVOIR - D
Y	IRRIGATION RESERVOIR (RES E)
Z	WATER TREATMENT PACKAGE PLANT

Figure 3.3: The TARDI site layout indicating the land occupancy.

Figure 3.4: The TARDI water layout.

3.3.4 Sanitation Infrastructure

TARDI is mostly provided with full waterborne sanitation and flushing toilets. The wastewater at TARDI is collected by the sewer mains, which discharge the liquid waste into the WSP located north of TARDI. The location of the existing WSP is shown in Figure 3.3 and Figure 3.4. The units of the WSP at TARDI comprises the septic tank, Facultative Pond (FP) and six Maturation Ponds (MPs). The actual dimensions, including the capacity of the ponds, are indicated in Figure 3.5 below.

The preliminary field investigation of the sanitation system at TARDI revealed the following typical issues, amongst others:

- The wastewater at the oxidation ponds does not follow the design sequence given that 1) the flow of wastewater from the FP only goes into the second last pond (5th MP). There was an overflow of wastewater that resulted in short circuiting, i.e. a shortened retention time, thereby leading to the washout of microorganisms mediating the treatment process. (The design values of the existing ponds were not known at the time of writing.); and 2) the wastewater at the manholes, next to staff residence, is discharged into the Primary MP (PMP).
- There was poor maintenance of the oxidation ponds and sewer network.
- The WSPs were not concrete lined to prevent the contaminated water to seep through the soil, thus degrading the water quality of both the underground and surface water.
- The capacity of these ponds was reduced by the settled sediments. The overloading happened before the upgrading of the ponds whereby the sludge was accumulated at the bottom of the FP because the ponds were not de-sludged ever since they were built
- The effluent from the MPs did not meet the minimum requirements of the DWS. The samples contained excessive Ammonia (7.7 mg N/l) as indicated in Table 2.14 and Table 3.4.

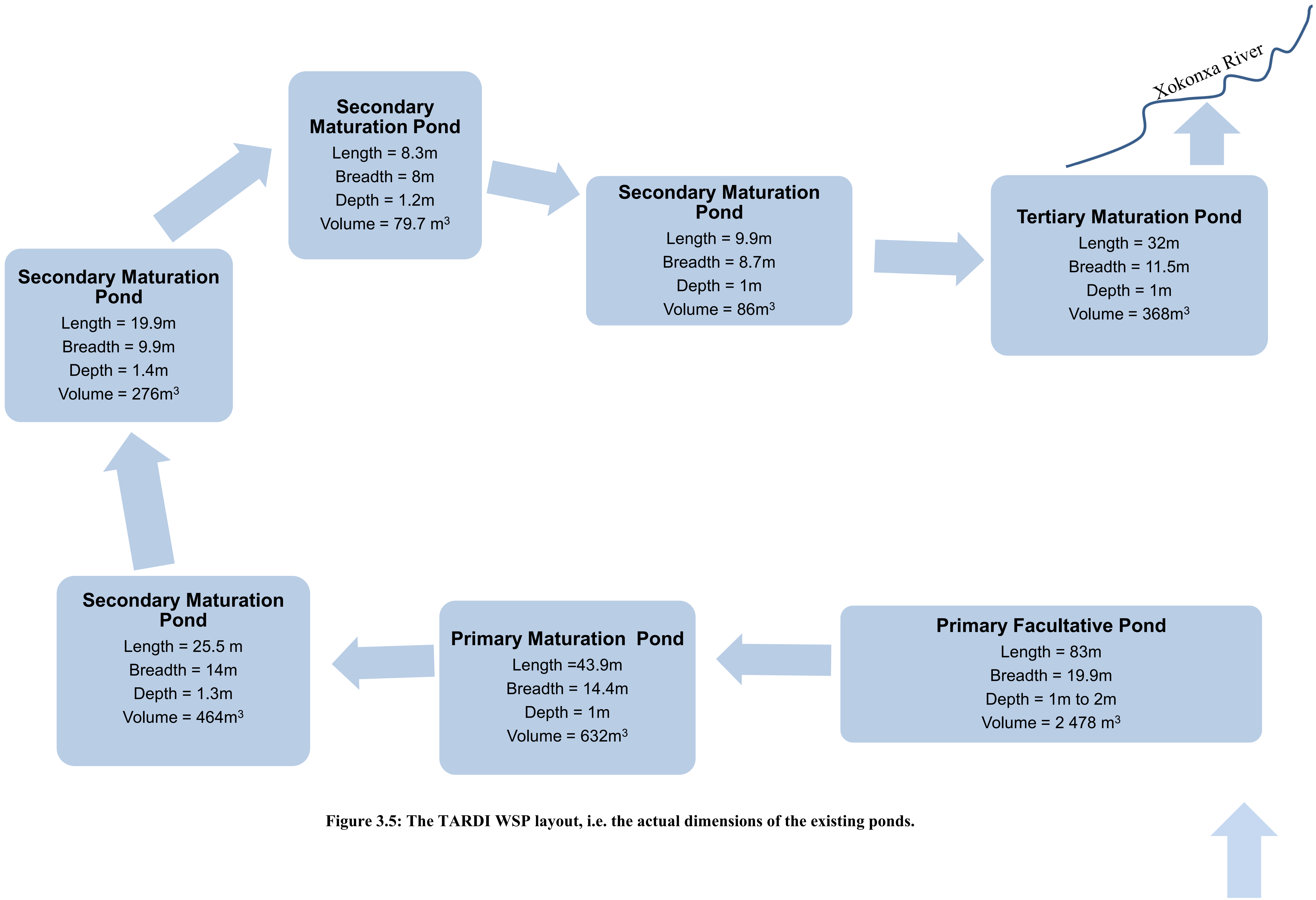


Figure 3.5: The TARDI WSP layout, i.e. the actual dimensions of the existing ponds.

3.3.5 Design Flow

The daily water demand of 100 l/person/day as suggested by (Department of Housing; DoH, 2000) is used to size the water infrastructure components for the in-house connection, of which the 85% of the water demand (600 l/d per dwelling, 150 l/occupant/day, 20 l/pupil/day, and others) are used to size the wastewater infrastructure components for TARDI. The average flow per dwelling is calculated to be 510 l/d/dwelling, which is close to the typical wastewater flow estimates for a full waterborne system of 500 l/dwelling, as per van Niekerk *et al.* (2009) and Nozaic and Freese (2009).

The average daily wastewater flow for TARDI is calculated to be 183 kl/d. The peak flow factor of 3.5, for populations less than 2 000, is used to determine the Peak Dry Weather Flow (PDWF) of 641 kl/d, as advised by Nozaic and Freese (2009) and DoH (2000). The Peak Wet Weather Flow (PWWF) of 737 kl/d is determined by using the long term PDWF (11–20 years) with an allowance of 15% for the stormwater infiltration (Nozaic and Freese, 2009).

The inflow to the TARDI WSP was not measured on site given that there was no flow meter in place.

3.4 COLLECTION, HANDLING, PRESERVATION, AND AND ANALYSIS OF WASTEWATER SAMPLES

The South African National Standards (SANS) were used to ascertain the sampling methods, and the preservation and handling of samples, including the testing procedures. The following SANS were referred to and used during the sample collection stage up to the data analysis and reporting stages:

- SANS 5667-2 Water quality- Sampling techniques.
- SANS 5667-3 Water quality- Preservation and handling of samples.
- SANS 5667-10 Water quality- Sampling.
- SANS 6047 Water - Dissolved oxygen content.
- SANS 6048 Water - Chemical oxygen demand.

- SANS 6049 Water - Suspended solids content.
- SANS 6051 Water - Oil and grease content.
- SANS 7888 Water - Determination of electrical conductivity.
- SANS 10523 Water - Determination of pH.

3.4.1 Sampling techniques and sample preservation and transportation

The wastewater (grab sample) was collected and sampled by independent laboratories (Talbot and Talbot, Monitor, and IDZ) at the inlet of the WSP and at the exit point of each pond unit as indicated in Figure 3.6 below. These samples were taken between 07/12/2020–26/03/2021, at monthly intervals for a short period of 3 months, and collected for testing purpose; the samples were collected from the sample points, shown in Figure 3.6, to the laboratories located offsite at approximately 280 km from the study area.

The samples for the microbiological analysis were collected by using a sterile bottle containing 0,1 g/l sodium thiosulphate. The samples were placed into 1 litre of polyethylene/glass bottles (depending on the type of parameters to be tested and analysed). The samples were preserved in an ice box during transportation before the analyses were done in the laboratory.

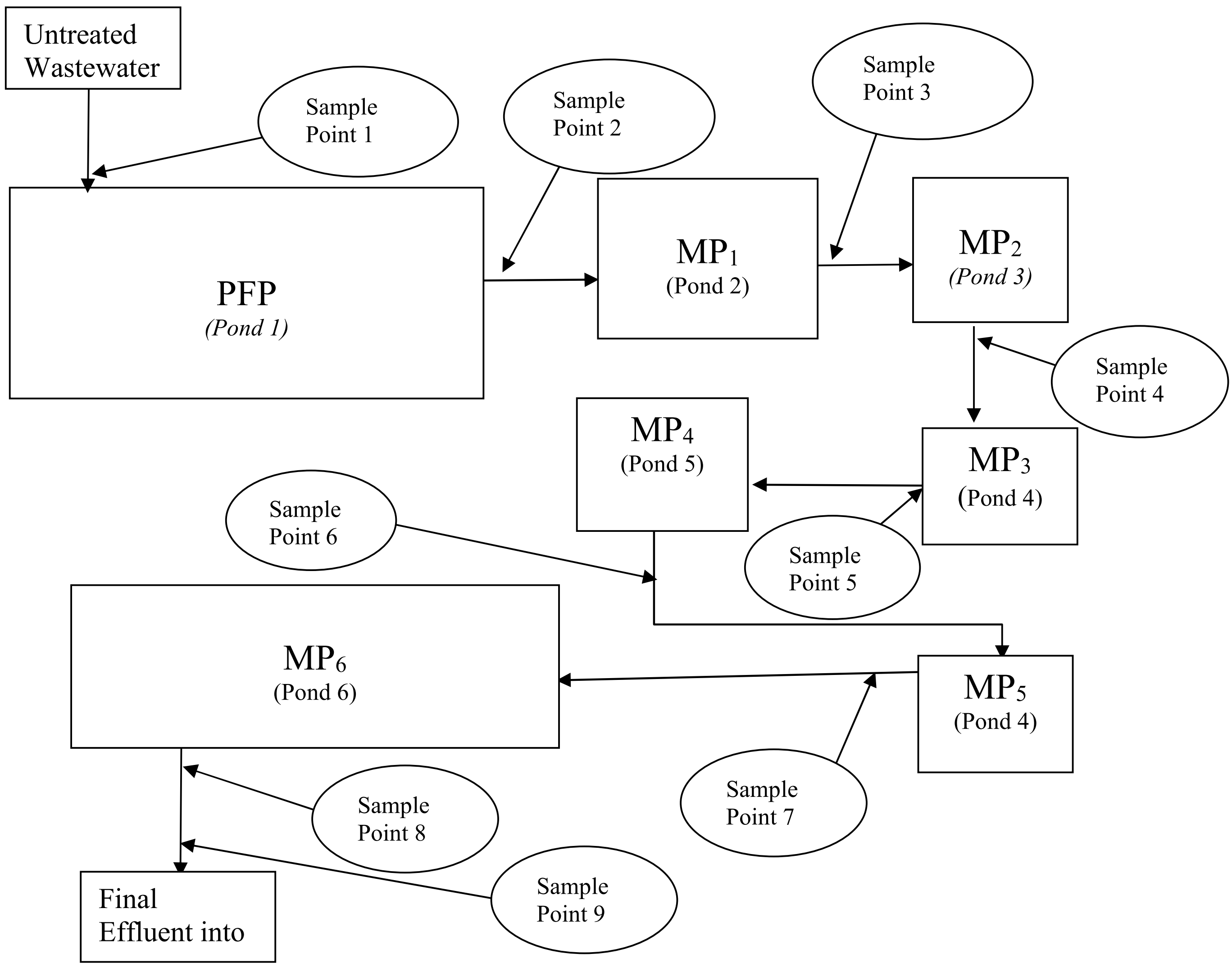


Figure 3.6: The sampling points at the TARDI WSP.

Sewer samples were taken at the inlet of the septic tank and outlet of the last MP at TARDI, and the results are presented in Table 3.2. The influent sewer characteristics at the entry point of the ponds were compared with the typical values of each parameter, as discussed in Section 2.4 of the literature review; this comparison is shown in Table 3.3. The comparison was also done to monitor the effluent from the TARDI WSP to assess its compliance with the DWS effluent guidelines, as indicated in Table 3.4.

3.4.2 **Experimental data analyses and methods**

This Section highlights the experimental data analyses and methods used in the present study. The methods used for the various parameters are shown in Table 3.**2**1. The analyses of the physical parameters are briefly discussed below.

Table 3.1: The methods used to analyse the parameters of the wastewater sample

Parameters	**Methods**
PHYSICAL PARAMETERS	
Electrical conductivity	Electrometry
COD	Closed vessel digestion and spectrophotometry, photometry
pH at 25 ^{0}C	Electrometry
BACTERIOLOGICAL/ MICROBIOLOGICAL PARAMETERS	
Faecal coliforms	Filtration
Total Coliforms	Colilert
E coli	Plate count

a) **Electrical conductivity**

The conductivity is a measure of the ability of an aqueous solution to carry an electric current. This depends on the presence of ions and their total concentration, mobility, and valence, including the temperature at which this measurement is made. For example, the warmer the water, the higher the conductivity. The solutions of most organic compounds that do not dissociate in aqueous solution conduct current poorly.

The electrical conductivity is done in accordance with SANS 7888 by using an apparatus called a conductivity meter. The apparatus consists of a conductivity cell and a meter that measures the conductivity. The conductivity cell consists of two electrodes (platinum plates) rigidly set at a constant distance from each other and connected to the meter by cables. The meter consists of a Wheatstone bridge circuit. The source of the electrical current in the meter applies a potential to the plates, and the meter measures the electrical resistance of the solution. To avoid changes in the apparent resistance with time, due to the chemical reactions (polarization effect at the electrodes), an alternating current is used. Some meters read resistance (ohm) while others read conductivity (milli-siemens per meter).

b) **Chemical oxygen demand**

Chemical Oxygen Demand (COD) analysis is a measurement of the oxygen-depletion capacity of a water sample contaminated with organic waste matter. Specifically, it measures the equivalent amount of oxygen required to chemically oxidize organic compounds in water. The COD test is used as a general indicator of water quality and is an integral part of water quality management programs.

The COD test is done in accordance with SANS 6048. This test involves a two-hour digestion at high heat under acidic conditions in which potassium dichromate acts as the oxidant for the organic matter present in a water sample and can oxidize nearly any organic matter to carbon dioxide and water. Given that potassium dichromate is a coloured material (yellow or orange, depending on the pH), it is reduced to chromic ion (Cr^{+3}), thereby producing a green solution. The spectroscopy is used to measure the extent of the reaction of the dichromate with the wastewater organics.

c) **pH**

The potential of hydrogen (pH) is a measure of the activity of the (solvated) hydrogen ion. p[H], which measures the hydrogen ion concentration is closely related to pH. Pure water has a pH very close to 7, at 25°C. Solutions with a pH less than 7 are acidic, and solutions with a pH greater than 7 are basic or alkaline.

The test related to pH is conducted in accordance with SANS 10523. The primary pH standard values are determined using a concentration cell with transference, by measuring the potential difference between a hydrogen electrode and a standard electrode, namely the silver chloride electrode. The measurement of the pH of aqueous solutions can be done with a glass electrode and a pH meter or using indicators.

Table 3.2: The influent and effluent test results of the wastewater samples taken from each pond unit

PARAMETER	UNIT	SAMPLE POINT (SP) No. 1							SP No. 2	SP No. 3	SP No. 4	SP No. 5	SP No. 6	SP No. 7	SP No. 8	SAMPLE POINT No. 9						
TEST DATE		**04/04/2017**	**09/10/2018**	**22/01/2019**	**07/12/2020**	**29/01/2021**	**24/02/2021**	**26/03/2021**	**07/12/2020**							**04/04/2017**	**09/10/2018**	**22/01/2019**	**07/12/2020**	**29/01/2021**	**24/02/2021**	**26/03/2021**
Faecal coliforms	No./100 ml	4100			>8000	>800	>8000	>8000	8600	120	23	7	4	4	<1	500			6	>800	7600	5200
COD (unfiltered)	mg/l	128	140	2438	26.2	27.2	906	32.6	28.7	24.6	29.3	33.6	42.6	60.4	23.3	56		67	24.6	46.9	63.9	54.1
pH at 25 ^{0}C	mg/l	8.4	6.78	6.86	7.61 (7.70)	7.31	7.78	(7.47)	8.12 (8.33)	8.17 (8.44)	8.25 (8.65)	8.58 (8.9)	8.71 (9.09)	8.86 (9.26)	9.44 (9.91)	7.9		9.02	9.39 (9.95)	8.92	8.78	(8.53)
Ammonia (NH_4 – N) dissolved	mg/l	5.8	8.4	38.4	10.3	2.58	8.13	12.1	8.33	5.34	3.94	2.2	1.29	0.487	<0.439	7.7		5.2	<0.439	0.439	<0.439	<0.439
Nitrate/nitrite (as N)	mg/l	<0.1	4.8		1.92	4.06	<0,195	0.872	<0,195	0.403	0.368	0.581	0.528	0.492	<0,195	<0.1			<0,195	0.698	<0,195	<0,195
Total Kjeldahl Nitrogen	mg/l				<0,15	0.20	23.3	24.8	<0,15	<0,15	<0,15	<0,15	<0,15	<0,15	<0,15				<0,15	0.80	2.8	0.28
Ortho-phosphates (as P)	mg/l	1.09			<1,00	<1,00	<1,00	1.26	1.09	1	1.05	<1,00	<1,00	<1,00	<1,00	1.61			<1,00	<1,00	<1,00	<1,00
Phosphates (as P)	mg/l		1.29	9.37														0.51				
Free Chlorine	mg/l	<0.1														<0.1						
Chlorophyll - a	µg				14	119	87.8	1.2	53	15	23	65	134	189	67				49	270	14.1	81.4
Oxygen Absorbed	mg/l			145														10				
Total Suspended Solids	mg/l	40	53	427	152	44	223	225	7	<1	6	9	17	30	15	11		60	11	45	15.0	23.0
Fluoride	mg/l	0.42														0.42						
Soap, oil or grease	mg/l	<3														<3						
Sodium Adsorption Ratio (SAR)					2.11	2.3	4.2	0.1	2.1	1.96	1.96	1.98	2	2	2.46				2.52	2.2	2.7	0.09
Sodium (dissolved)	mg/l				110	94.3	263	4.3	98.6	90.9	90.7	90	90.5	89.6	90.6				90.4	88.8	103	2.5
Calcium (Dissolved)	mg/l				108	61.2	97.7	77.5	90.3	86.1	86.2	82.4	82	79.5	43.2				38.1	49.8	33.6	13.1
Electrical Conductivity	mS/m	126	152.7	268	141	106	273	133	112	109	107	102	99.8	96.5	83.2	134		171	82.1	82.6	89.6	130
Alkalinity			414	582														352				
Magnesium (total)	mg/l				59.7	43.4	119	33.8	46.9	46.3	45.8	44.6	44.6	43.9	36				35.8	45.4	44.3	27.1
Temperature	°C				20.6	21.0	23.8		20.8	20.9	21	21.1	21.2	20.9	20.8				20.7	21.1	23.6	

Not measured parameters, data or result not available

(8.33) Additional test results

3.4.3 Comparisons between the typical domestic wastewater characteristics and influent data at the entry point of the TARDI WSP

Table 3.3 below shows that the COD, Ammonia, TKN, and TSS of the data samples taken from the TARDI WSP were all lower than those of the typical wastewater from the literature discussed in Section 2.4, except for the measurement of 906 mg COD /l taken on 24/02/2021, which was higher. There is no exact value on the water test results indicated for faecal Coliform.

The pH of the samples compared with the discharge guidelines were found to be between 6.7–8, which was deemed acceptable. Three of the four sample nitrates were found to be within the acceptable range of 0–2 mg N/l, except for the outlier sample, which was taken on 29/01/2021 and which had a nitrate concentration of 4.06 mg N/l.

The above scenarios indicate the precautions required when designing and sizing the WSPs as the on-site wastewater quality data differ from the literature data.

3.4.4 Comparisons between the effluent guidelines for treated wastewater and effluent test results of the TARDI WSP.

Most of the effluent data of the TARDI WSP meet the guidelines of both the DWS and WHO as indicated in Table 3.4. The pH values of 9.39 and 9.95, which were both measured on 07/12/2020, are found to be higher than the WHO guideline pH of 9.0. Only the pH value of 9.95 was found to be above the DWS guideline pH of 9.5.

There is no exact value indicated for faecal coliform tested on 29/01/ 2021. The test results of faecal coliforms of 7 600 and 5 200 FC /100ml, dating from 24/02/2021 and 26/03/2021, respectively, are found to be higher than both the DWS and WHO guidelines.

Table 3.3: The comparisons between the typical domestic wastewater characteristics and influent test results taken from untreated wastewater at the entry point of the TARDI WSP

PARAMETER	UNIT	TYPICAL DOMESTIC WASTEWATER QUALITY (minimum and maximum values)		SAMPLE POINT No. 1										
TEST DATE		Min	Max	04/04/2017	09/10/2018	22/01/2019	07/12/2020	29/01/2021	24/02/2021	26/03/2021	Max	Min	Average	Median
Faecal coliforms	No./100 ml	10^6	10^9	4 100			>8000	>800	>8000	>8000	8 000	800	5 780	8 000
COD (unfiltered)	mg/l	450	800	128	140	2438	26.2	27.2	906	32.6	2 438.0	26.2	528.3	128.0
pH at 25 ^{0}C		6.7	8.0	8.4	6.78	6.86	7.61 (7.70)	7.31	7.78	(7.47)	8.40	6.78	7.46	7.47
Ammonia (NH_4 – N) dissolved	mg/l	20	60	5.8	8.4	38.4	10.3	2.58	8.13	12.1	38.40	2.58	12.24	8.40
Nitrate/nitrite (as N)	mg/l	0	2	<0.1	4.8		1.92	4.06	<0,195	0.872	4.80	0.10	2.35	1.92
Total Kjeldahl Nitrogen	mg/l	35	85				<0,15	0.20	23.3	24.8	24.80	0.20	16.10	23.30
Ortho-phosphates (as P)	mg/l			1.09			<1,00	<1,00	<1,00	1.26	1.26	1.09	1.18	1.18
Phosphates (as P)	mg/l	4	15		1.29	9.37					9.37	1.29	5.33	5.33
Free Chlorine	mg/l			<0.1							0.10	0.10	0.10	0.10
Chlorophyll - a	µg						14	119	87.8	1.2	119.00	1.20	55.50	50.90
BOD	mg/l	250	350											
TSS	mg/l	700	1 350			427	152	44	223	225	427.00	44.00	214.20	223.00
Settleable solids	ml/l	8	20											
Suspended solids	mg/l	200	450	40	53						53.00	40.00	46.50	46.50
Fluoride	mg/l			420							420.00	420.00	420.00	420.00
Soap, oil or grease	mg/l			<3							3.00	3.00	3.00	3.00
Sodium Adsorption Ratio (SAR)							2.11	2.3	4.2	0.1	4.20	0.10	2.18	2.21
Sodium (dissolved)	mg/l						110	94.3	263	4.3	263.00	4.30	117.90	102.15
Calcium (dissolved)	mg/l						108	61.2	97.7	77.5	108.00	61.20	86.10	87.60
Electrical Conductivity	mg/l			126	152.7	268	141	106	273	133	273.00	106.00	171.39	141.00
Magnesium (total)	mg/l						59.7	43.4	119	33.8	119.00	33.80	63.98	51.55
Temperature °C	°C						20.6	21.0	23.8		23.8	20.6	21.8	21.0

Not measured parameters, data or result not available

(7.70) Additional test results

Table 3.4: The comparisons of the effluent discharge guidelines of the treated wastewater with the effluent test results of the TARDI WSP

PARAMETER	UNIT	DWS GUIDELINES	WHO GUIDELINES	SAMPLE POINT No. 9									
TEST DATE				**04/04/ 2017**	**22/01/ 2019**	**07/12/ 2020**	**29/01/ 2021**	**24/02/ 2021**	**26/03/ 2021**	**Max**	**Min**	**Average**	**Median**
Faecal coliforms	No./100 ml	<1 000/100 ml	<1 000/100 ml	500		6	>800	7600	5200	7 600.00	6.00	2 821.20	800.00
COD (unfiltered)	mg/l	<75 mg/l	<100 mg/l	56	67	24.6	46.9	63.9	54.1	67.00	24.60	52.08	55.05
pH at 25 ^{0}C		5,5 – 9,5	5,5 – 9,0	7.9	9.02	9.39 (9.95)	8.92	8.78	(8.53)	9.39	7.90	8.76	8.85
Ammonia (NH_4 – N) dissolved	mg/l	<6 mg/l		7.7	5.2	<0.439	0.439	<0.439	<0.439	7.70	0.44	2.44	0.44
Nitrate/nitrite (as N)	mg/l	<15 mg/l		<0.1		<0,195	0.698	<0,195	<0,195	0.70	0.10	0.40	0.40
Total Kjeldahl Nitrogen	mg/l	<25 mg/l	<100 mg/l			<0,15	0.80	2.8	0.28	2.80	0.28	1.29	0.80
Ortho-phosphates (as P)	mg/l	<10 mg/l		1.61		<1,00	<1,00	<1,00	<1,00	1.61	1.61	1.61	1.61
Phosphates (as P)	mg/l				0.51					0.51	0.51	0.51	0.51
Free Chlorine	mg/l	<0,25 mg/l		<0.1						0.10	0.10	0.10	0.10
Chlorophyll – a	µg					49	270	14.1	81.4	270.00	14.10	103.63	65.20
BOD	mg/l		<30 mg/l										
TSS	mg/l	<25 mg/l	<30 mg/l	11	60	11	45	15.0	23.0	60.00	11.00	27.50	19.00
Fluoride		<1 mg/l		0.42						0.42	0.42	0.42	0.42
Soap, oil or grease		<2,5 mg/l		<3						3.00	3.00	3.00	3.00
Sodium Adsorption Ratio (SAR)						2.52	2.2	2.7	0.09	2.70	0.09	1.88	2.36
Sodium (dissolved)	mg/l					90.4	88.8	103	2.5	103.00	2.50	71.18	89.60
Calcium (dissolved)	mg/l					38.1	49.8	33.6	13.1	49.80	13.10	33.65	35.85
Electrical Conductivity	mS/m	70 mS/m above intake to a maximum of 150 mS/m		134	171	82.1	82.6	89.6	130	183.40	-8.00	59.62	41.15
Magnesium (total)	mg/l					35.8	45.4	44.3	27.1	45.40	27.10	38.15	40.05
Temperature °C	°C					20.7	21.1	23.6		23.60	20.70	21.80	21.10

Not measured parameters, data or result not available

(9.95) Additional test results

3.4.5 **Evaluation of the performance efficiency of the existing ponds**

Selected physiochemical and microbiological parameters were analysed by private laboratories in accordance with SANS (SANS 5667 -10, 2022). The concentrations, at the inlet and outlet of each pond unit, were recorded as indicated in Section 3.3.

The percentage of the removal efficiency was calculated based on Equation 3.2 below.

$$\%\,removal = \left(\frac{C_i - C_e}{C_i}\right) 100\% \qquad \text{Equation 3.2}$$

Where:

C_i Concentration of the influent.

C_e Concentration of the effluent.

3.4.6 **Evaluation of the hydraulic performance of the existing ponds regarding to hydraulic retention time**

The retention time (expressed as the ratio of pond volume over flow rate) was calculated from the pond dimensions (length, width, and depth). The design flow was determined in detail in Section 3.3.5. Hence, there was no flow measuring device in place or installed.

The drogue method is commonly used to assess the actual hydraulic retention time of a pond unit (Gopolang & Letshwenyo, 2018). When using this method, five oranges are placed at the inlet of each pond and the time taken by each orange to travel from the inlet to the outlet position is recorded, and the average time of all five oranges is used as the actual hydraulic retention time. The actual hydraulic retention time was not measured at the study area, therefore no comparison was made between the actual and calculated hydraulic retention times.

3.4.7 BOD and COD

The influent BOD value of 350 mg BOD/ l, as recommended by von Sperling (2007a), van Niekerk *et al.* (2009), and Nozaic *et al*. (2009), is used to check the capacity of the existing WSP or design the WSP system. Nozaic *et al.* (2009) indicated that the range of COD/BOD ratio is approximately 1.8:1–2.5:1 in domestic wastewaters and the value of 2:1 is commonly assumed. The effluent COD value of 75 mg O/l is divided by two to obtain the effluent BOD, which is used for pond design calculation purposes.

The 24-hour weighted flow composite was not calculated due to the potential high costs of taking samples and measuring flows at 1-hour intervals over a 24-hour period.

3.4.8 Faecal coliforms (FC)

Nozaic *et al.* (2009) stated that the reasonable design value of N_i as 4×10^7 FC/100 ml can be used. Although this is higher than the average values that are commonly used in South African practice. The grab samples can be used to measure the faecal coliform concentrations.

3.4.9 Design Temperature

Nozaic *et al.* (2009), Mara (2003), and Arthur (1983) recommended the mean temperature for the coldest month to be used as the design temperature for APs and FPs. The coldest and hottest months of the year in TARDI are July and January with the average temperatures of 7 ºC and 26 ºC, respectively ("Umtata Climate Weather…", n.d.).

3.4.10 Data verification

The verification of the water samples and design flows are indicated below:

1. Water Samples: the comparison of the pond tested data and literature data was done, and it is tabulated in Table 3.3 and Table 3.4. Both Table 3.3 and Table 3.4 indicate

that test results should be monitored over a long time, given the outliers found, and it is difficult to make an informed judgement on few sample data.

2. Design Flows: the existing flow was estimated by measuring a pipe diameter and the depth of the water inside the sewer pipeline and subsequently calculating the cross-sectional area. A leaf was placed approximately a meter upstream of the marker, and the time taken by the leaf to reach the marker was recorded. The flow rate was calculated by using the volume of the water divided by the time taken by the leaf to move over a meter (Boyd & Mbelu, 2009).
3. Temperature: the design temperature used in Section 3.4.9 was compared with the weather data obtained from the South African Weather Station (SAWS).

3.5 PROBLEMS ENCOUNTERED DURING DATA COLLECTION, PREPARATION, AND ANALYSIS

The samples were initially collected by environmentalists for Scenario 1 and later collected by professionals from the SANAS registered company, as prescribed in the National Water Act (Act 36 of 1998) and SABS Code 0259.

The data received from the laboratories were checked for inaccuracies and errors prior to commencement of data analysis. The Microsoft Excel Software was used for the analysis of the data, including the creation of graphs and tables presented in Section 4, and comparisons of the data sets.

The following is the summary of the main problems and challenges encountered during the collection, preparation, and analysis of the data:

- The unavailability of the influent and effluent water quality test results, including flow test results.
- The unavailability of the flow measuring device and test results (for the 1-hour interval measurements), leading to the lack of the 24-hour weighted composite flow data for the study area. The 24-hour weighted composite flow is used to determine the current average loading to the WSP that will be used to predict the future long-term wastewater flows to the WSP.
- The collection of samples by unprofessional and unregistered personnel.

- The non-feasibility to perform the statistical analysis due to insufficient influent and effluent water quality test results.
- The unavailability of the as-built information, including the installation of flow meters and design capacity, to determine the geometry of the ponds.
- The inaccessibility to the interior of the septic tank and ponds to obtain accurate dimensions (i.e. bottom width, length, and depth).
- The poor data recording given that most authorities perform these tests to comply with the DWS requirements and maintain their Water Use License (WUL).

All the seven pond sites, namely the 14 SAI Battalion, Bedford Hospital, Mthatha Prisons, St Patricks School, Marselle township, FCC, and TARDI, which were identified as the initial study areas in the Eastern Cape, had similar issues as listed above.

The following must be considered to limit the data errors in pond design and performance:

- The performance of the WSP must be determined and analysed before considering the upgrading of the WSP.
- The necessary equipment and instruments, namely floating digital thermometers, electrochemical or optical sensor to measure the dissolved oxygen, must be installed inside the WSP system to monitor the performance of each WSP unit.

3.6 METHOD OF DATA ANALYSES

The experimental data analyses were handled by independent laboratories, and the present study focuses on the evaluation of the test results provided by the laboratories, including the comparisons with the typical characteristics of domestic wastewater. Further, mathematical models that virtually replicate the treatment process at the WSP system were created as tools that can be used in sizing the system.

3.6.1 **Development of models that can be applied for the feasibility design of WSP – Scenario 1**

Models were tested and used to predict the effluent of both the A-F-M and F-M systems during the feasibility stage referred to as Scenario 1 in the present study. The A-F-M system consists of the AP, FP, and MPs, while the F-M system consists of the FP and MPs. The composition of the two systems is shown in Figure 3.7 and Figure 3.8 below.

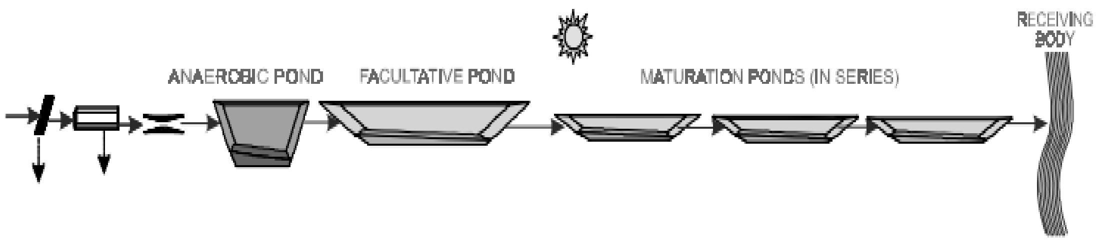


Figure 3.7: The A-F-M system, extract from (von Sperling, 2007b).

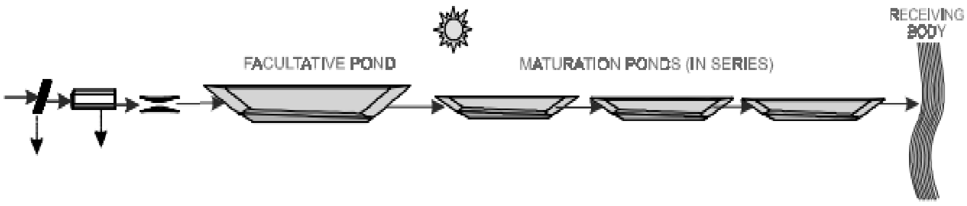


Figure 3.8: The F-M system, extract from (von Sperling, 2007b).

The number of models considered for each pond unit in both the A-F-M and F-M systems are briefly indicated below.

The A-F-M system consisted of:

- Three models of the AP (A-F-M system) considered for the AP;
- Four models of the SFP (A-F-M system) used for the sizing of the SFP; and
- Three models of the MP (A-F-M system) used for the design of the MP.

The F-M system consisted of:

- Four models of the PFP (F-M system) used for the sizing of the PFP; and
- Three models of the MP (F-M system) used for the design of the MP.

The details of the models used on each pond unit are described in the subsequent sections.

3.6.1.1 **A-F-M system for Scenario 1**

The method of sizing the units of the A-F-M system are discussed in the following sub-sections.

AP for the A-F-M system for Scenario 1

Three models were considered, which assumed an influent BOD concentration and pond depth of 350 mg/l and 2 m, respectively (von Sperling, 2007a and Nozaic *et al.,* 2009). Equation 2.7 in Chapter 2 is used to calculate the volumetric BOD loading of the AP.

- Model 1 assumed a retention time of 2 days to calculate the volume of the AP. Equation 2.7 is then used to determine the volumetric BOD loading.
- Model 2 used an average volumetric BOD loading of 250 g BOD/m^3.d, as per Equation 2.7, to determine the pond volume. The retention time is calculated by dividing the pond volume by the influent flow.
- Model 3 used the volumetric loading formula, represented by 20T – 100 for temperature between 20 ^{0}C to 25 ^{0}C as indicated in Table 2.12, to calculate the volumetric BOD loading. Equation 2.7 is used to determine the pond volume.

The results of the above models are tabulated and discussed further in Section 4.3.1.

FP for the A-F-M system for Scenario 1

Four models were used to size the FP, which were based on (i) the Mara empirical equation, (ii) the first order kinetics, (iii) the McGarry and Pescod empirical procedure, and (iv) the Marais design sequence. The FP design models for Scenario 1 were done based on two WSP options, which are the WSP with and without APs. The pond length to breadth ratio used is 1.5, which is the ratio of 2:3 as recommended by Nozaic, *et.al.* (2009).

SFP of the FP with AP for the A-F-M system for Scenario 1

The influent BOD and FC concentrations to the secondary facultative pond (SFP) are the respective effluent concentrations of the AP. These concentrations (196 mg/l and 1.53 x 107 FC/100 ml respectively), from Model 3 shown in Table 4.2**,** are used to simplify the design of the SFP (A-F-M system).

The SFP (A-F-M system) was sized based on the four models indicated in the above sub-section.

- The Mara empirical equation was used to determine the required FP mid-depth area and the retention time of Model 1 as shown in Table 4.4.
- The first order kinetics, Model 2 in Table 4.4, were used to determine the area of the FP. This was done by using Equation 2.29 and Equation 2.30**.** The effluent BOD concentration was assumed to be 37.5 mg/l.
- The Model 3 used the McGarry and Pescod empirical procedure where the Arial loading rate was calculated, using Equation 2.23, to determine the FP area as indicated in Equation 2.23.
- The Marais design sequence is model 4, as shown in Table 4.4. Equation 2.27 was used to compute the pond retention time, which was then used to calculate the pond volume.

MP with AP for the A-F-M system for Scenario 1

The design of the MP (A-F-M system) was based on the model by both Marais (1974) and von Sperling (2007b). The retention of the MP was calculated by using Equation 2.33 for n equals to 1,2,3,4, and 5.

Three models were used as shown in Table 4.6 , Table 4.7, Table 4.8, and Table 4.9, in Chapter 4, and they were based on the literature mentioned in Sections 2.9.2.1d) and 2.9.3.

- Model 1:
    - The number of MPs is assumed until the retention time of approximately to 3 days is obtained.
    - The BOD loading in the first MP is calculated to check if it is higher than the BOD loading in the FP. When the BOD loading in the PMP is found to be higher than that in the FP, it is adjusted to the same BOD loading as in the FP.
    - The revised retention time is recalculated to get the volume of the PMP.
    - The first step is repeated to determine the number of Secondary Maturation Ponds (SMPs).
    - The effluent FC at the last SMP is checked by using Equation 2.32 and Equation 2.33.
- Model 2:
    - The retention time is assumed to be 10 days for all the MPs.
    - The Equation 2.32 and Equation 2.33 are used to calculate the effluent FC at the last SMP.
    - The Equation 2.31 and Equation 2.32 can also be used to calculate the effluent FC from each SMP.
    - The above step is repeated until the effluent FC at the last SMP meets the minimum DWS limits.
- Model 3:
    - The minimum retention time of the PMP is assumed to be 7 days for all the MPs.
    - The Equation 2.31 and Equation 2.32 are used to calculate the effluent FC from each SMP until the effluent FC meets the DWS limits.

3.6.1.2 **F-M System for Scenario 1**

The design considerations of the WSP units of the F-M system are discussed in the following Sections.

PFP of the FP without AP for the F-M system for Scenario 1

For the sizing of the PFPs, the influent BOD concentration to the FP and its depth were assumed to be 350 mg/l and 2 m respectively, as recommended in Chapter 2.

The PFP (F-M system) was sized based on the four models mentioned in Section 3.6.1.1 above.

- The Model 1 is based on the Mara empirical equation and the surface loading of 124 kg/ (ha. day), which is calculated by using Equation 2.26 or Table 2.13. Equation 2.28 is used to determine the PFP mid-depth area. The retention time is calculated by using the PFP volume and wastewater flow.
- The first order kinetics are used as Model 2 as per Table 4.6 of Chapter 4. Equation 2.29 and Equation 2.30 are used to compute the area of the PFP by using the effluent BOD concentration of 37.5 mg/l. The calculated retention time was found to be greater than the minimum of 4 days.
- The Model 3 uses the McGarry and Pescod empirical procedure where the arial loading rate is calculated using Equation 2.23 to determine the PFP area as indicated in Equation 2.23.
- The Marais design sequence is used as model 4 as shown in Table 4.6, and Equation 2.27 is used to compute the pond retention of 75.8 days, which is used to calculate the volume of 13 864 m^3.

MP without AP for the F-M system for Scenario 1

Three models were used, as shown in Table 4.9 and Table 4.10, and they are the same models as used in the MP for the A-F-M system. The same procedure, as indicated in Section 3.6.1.1, is followed for all three models, and the design parameters are presented in Table 4.9 to Table 4.12.

3.6.2 Selection and extension of the best feasibility design model for the completion of the WSP sizing – Scenario 2

The best of the models described in Section 3.6.1 was selected for further calculations that were used in the conceptual design of the WSP to determine the pond sizes and effluent characteristics of each pond unit. This stage is referred to as Scenario 2 in the present study.

3.6.2.1 Septic tank pond for Scenario 2

The length and width of the septic tank were determined on site by measuring the width and length of the top cover slab. The depth was determined by dipping the septic tank to the floor level using a wooden stick and the water depth determined at the level of the water mark.

Equation 2.6 is used to determine the volume of the septic tank pond (Nozaic and Freese [2009] and DWA [2009]).

3.6.2.2 PFP of the FP without the AP for the F-M system for Scenario 2

The sizing of the PFP (F-M system) was based on the Marais design sequence. The designs for Scenario 2 are done based on the existing pond configuration, i.e. the WSP without APs. This implies that there is no change in the BOD concentration of the inlet of the septic tank. Also, the BOD removal at the septic tank was assumed to be zero percent (0%). The Marais design sequence was used to calculate the volume of the primary pond. It should be noted that the calculations on this model do not take into account the surface loading (λs) and/or arial loading rate (λ_d).

3.6.2.3 MP without the AP for the F-M system for Scenario 2

The Equation 2.33 was used to design the MP (F-M system). The literature by Marais and Shaw (1961) recommends the design of two or more MPs, with 7 days of retention time, in a case where the effluent FC from the PFP does not meet the

DWS limit. This concept was used to design the primary and secondary MPs for the given study area.

3.7 DESIGN CRITERIA

Table 3.5 sets out the design criteria that were used for the study area.

Table 3.5: The design criteria for the sewer network and ponds.

ITEM No.	DESCRIPTION	QUANTITY	REFERENCE
1	Average Daily Flow		
1.1	Low Income Group	70 l /person/day 500 l /dwelling /day 100 l /person/day	Nozaic *et al.*,2009 Nozaic *et al.*,2009 DoH, 2000.
1.2	Middle Income Group	750 l /person/day	City of East London Guidelines, n.d.
2	Allowance of stormwater infiltration (%)	15	
4	Flow formula	Manning – 0,012	
5	Minimum pipe diameter (reticulation network, excluding connections)	110 mm diameter	
6	Velocity in pipes		
6.1	Minimum	0,7 m/s	DoH, 2000.
6.2	Maximum	2,5 m/s	DoH, 2000.
6.3	Optimum	1,0 m/s	DoH, 2000.

von Sperling (2007a), van Niekerk *et al.* (2009), and Nozaic *et al.* (2009) tabulated the typical municipal domestic wastewater qualities found in South Africa, as shown in Table 2.2 in Section 2.4, some of which were adapted from Ekama (1984) and used for the WSP design purposes in the present study.

The Microsoft Excel Software is used to model the proposed WSP sizes and designs as discussed in Section 4.

3.8 DESIGN ASSUMPTIONS

The following assumptions for pond design were made:

- The ponds are completely mixed, which means that the concentration inside the pond is the same as the effluent and BOD do not settle with sludge (Marais and Shaw [1961] and Adhikar and Fedler [2019]). This is for the simplified design model. However, for more complex dynamic models, a computational fluid dynamic modelling (CFD) approach should be used.

- The organic matter degradation follows the first-order kinetics, as explained in Section 2.9.2.
- The existing pond geometry (depth, bottom and top widths, bottom and top lengths, side slopes, water depths, and others) is estimated due to the unavailability of as-built drawings. This is deemed acceptable for a comparative study as the present study.
- The sludge blanket is not taken into consideration when determining the effective depth, hence, the sludge blanket depth is not measured on site at the time of study. Also, the assumption is made simple for modelling purposes.
- The design flows to the existing ponds are calculated based on 85% of the water demand (City of East London Guidelines, n.d.), although Nozaic *et al*. (2009) recommends approximately 70% of domestic water consumption to be expected to reach the sewer.
- The net evaporation is also assumed to be zero to simplify the process design of the WSP.
- The growth rate of 0.5% per annum is used to predict the future water demands and design flows.

3.9 DESIGN APPROACH

Two design horizons are considered for the provision of water for both the short-term and long-term demands. The short-term horizon is based on the water demand, which is less than 5 years, and the long term is selected as 11 to 20 years. The sewer network, including the wastewater treatment works (WWTWs), is to be designed to meet the long-term projection, which is 20 years.

The population figures supplied by TARDI are used to determine the above-mentioned water demands.

3.10 CONCLUSIVE SUMMARY

Seven sites in the EC Province were initially selected for the present study. These sites are the 14 SAI Battalion, Bedford Hospital, Mthatha Prisons, St Patricks School, Marselle township, FCC, and TARDI. The water quality data was collected by three distinct organisations/groups (i.e. Local Authorities, DWS, and independent or private laboratories) during the years 2016 up to 2021. The study area was narrowed down from seven sites to three sites viz. Marselle township, FCC, and TARDI. Since, among other things, the geometry of the ponds, and other parameters as indicated in Section 3.2, were unknown, the selection of the study area was further refined to the TARDI.

The detailed comparison between the typical domestic wastewater and influent test results of untreated wastewater was done and presented in Table 3.3, for TARDI only, due to the financial and time constraints of the present study area. The effluent water quality of the WSP was used solely to check if the effluent met the requirements of the DWS. Two sites viz. FCC and TARDI were found to have the records of both the influent and effluent water quality tests. TARDI was then selected, out of the seven WSP sites, given that its WSP had the measurable parameters and characteristics of the WSP as discussed in detail in Section 3.2. These measurable parameters are the wastewater concentrations and flow data, for both the inlets and outlets of the pond units, existing pond geometry, and design capacity of the existing WSP. The environmental data were used to carry out the present study.

Due to the unavailability of data, assumptions were made for Scenarios 1 and 2 stages, which are discussed in detail under Section 3.8. The description and location of the study area are briefly discussed under Section 3.3.1.

The TARDI provided the information on the land use and building occupancy, including the number of livestock, students, future upgrades, and others. The information is shown in Figure 3.3. Equation 3.1 as suggested by von Sperling (2007a) and Umara, *et al.* (2010), was used to forecast the population, and the growth rate of 0.5% over 20 years was assumed, hence, the expansion of the institution was anticipated to be negligible. The forecast method used for the study area was based on the geometric growth method, which is the mathematical method discussed briefly in Section 2.12.

The water resources and associated infrastructure at TARDI are discussed under Section 3.3.3. Two water sources, namely the Xokonxa River, which is a surface water source, and two BHs, which are the ground water sources, were identified within the study area. The sanitation infrastructure at TARDI primarily consists of full waterborne sanitation and flushing toilets. The wastewater at TARDI is collected by the sewer mains, which discharge the liquid waste into the WSP. The units of the WSP at TARDI comprises the septic tank, FP, and six MPs. The actual dimensions, including the capacity of the ponds, are determined as shown in Figure 3.5 under Section 3.3.4. The field investigations related to both the water and sanitation systems are also noted and recorded.

The water demand of 100 l/person/day (DoH, 2000) is used to size the water infrastructure components for the in-house connection, of which 85% of the water demand are expected to reach the sewer (City of East London Guidelines, n.d.). The calculated average flow of 510 l/d/dwelling is found to be close to the typical wastewater flow estimate for a full waterborne system of 500 l/dwelling, as suggested by van Niekerk *et al.* (2009) and Nozaic and Freese (2009).

The SANS were used to ascertain the sampling methods, preservation and handling of samples, and testing procedures, as discussed in Section 3.4. The discussions of the experimental data analyses and methods, including the verification of the water sample data, flows, and temperatures, in present study, were performed as discussed in Section 3.4.10. The verification of the data was to ensure that the influent concentration data, used for modelling Scenario 2 in Chapter 4, are within the acceptable limits as discussed in Section 2.4.

The unavailability of the historical data, namely the water quality test results and inflow data, and the as-built drawings, were found to be the major challenges encountered during the collection, preparation, and analysis of the data. The experimental data analyses were handled by independent laboratories, and the present study focuses on the evaluation of the test results provided by the laboratories. The Excel spreadsheet was used to determine the minimum, average, median, and maximum concentrations of the wastewater parameters obtained from the sample points, shown in Figure 3.6.

The mathematical models that virtually replicate the treatment process of the WSP system were conceived as tools for sizing the system. These models were tested and used to predict the effluent of both the A-F-M and F-M systems during the feasibility stage referred to as Scenario 1 in the present study.

The two types of WSP configuration, namely the A-F-M and F-M systems, are considered during the Scenario 1 and 2 stages. The A-F-M system is the WSP containing an Anaerobic Pond (AP), Facultative Pond (FP), and Maturation Pond (MP), whereas the F-M system is the A-F-M system without AP. The details of the models considered for each pond unit in both the A-F-M and F-M systems for Scenario 1 are summarised in Table 3.6 below.

Table 3.6: The summary of the models used for the A-F-M and F-M systems for Scenario 1

		A-F-M System		
Pond Unit	**Model 1**	**Model 2**	**Model 3**	**Model 4**
AP (A-F-M system)	Detention time of 2 days	volumetric BOD loading of 250 g BOD/m^3.d	volumetric BOD loading of 140 g BOD/m^3.d	
SFP (A-F-M system)	Mara Empirical Equation	First-order kinetics	McGarry & Prescod Empirical procedure	Marais design sequence
MP (A-F-M system)	The retention time of about 3 days	The retention time of 10 days to all the MPs	The retention time of 7 days to all the MPs	
		F-M System		
PFP (F-M system)	Mara Empirical Equation	First-order kinetics	McGarry & Prescod Empirical procedure	Marais design sequence
MP (F-M system)	The retention time of about 3 days to all MPs	The retention time of 10 days to all the MPs	The retention time of 7 days to all the MPs	

The three models are considered for the AP during Scenario 1 stage. The assumed influent BOD concentration and pond depth of 350 mg/l and 2 m, respectively, are used to design the APs. The FP design models for Scenario 1 are done based on two WSP options, which are the WSP with and without APs, namely SFP (A-F-M system) and

PMP (F-M system), respectively. The pond length to breadth ratio used is 2:3, as recommended by Nozaic, *et.al.* (2009). For the sizing of the PFPs, the influent BOD concentration and PFP depth are assumed to be 350 mg/l and 2 m, respectively, as recommended in Chapter 2. The MP are designed by using Equation 2.33.

The best of the models described in Section 3.6.1 are selected for further calculations to design the WSP to determine the pond sizes and effluent characteristics of each pond unit during the Scenario 2 stage. The summary of the models used for Scenario 2 is shown in Table 3.7.

Table 3.7: The summary of the models used for the F-M system for Scenario 2

Pond Unit	Model used	Comments
PFP	Marais design sequence	The calculations on this model do not consider the surface loading (λ_s) and, or arial loading rate (λ_d).
MP	Retention time of 7 days to all the MPs	The design of two or three maturation pond in cases where the effluent FC from the primary facultative meet the DWS general limit.

The design criteria were developed based on the literature by Nozaic *et al.*, 2009 DoH, 2000, and they are discussed in Section 3.7. The design assumptions used for the models are highlighted in Section 3.8. The Microsoft Excel Software is used to model the WSP sizes and designs during the Scenario 1 and 2 stages as discussed in Section 4.

Two design horizons are considered for the provision of water for both the short-term and long-term demands. The term horizon is discussed in detail in Section 3.9. The WSP is designed to meet the long-term projection, which is 20 years.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 INTRODUCTION

The water quality test results of the TARDI WSP were generated and presented by the independent laboratories, and the data were analysed in detail in Section 3.4.2. The data of the influent test results were further compared with the typical domestic wastewater characteristics, as highlighted in Section 3.4.3, to confirm the acceptance of the data which were provided by the independent laboratories. In addition, the effluent test results of the WSP system were compared with the limit guidelines of both the DWS and WHO as indicated in Section 3.4.4.

The water test results were used for modelling the WSP for Scenario 2, as discussed in Section 4.4 below, whereas the design inputs or parameters used during the feasibility stage (Scenario 1) were the typical domestic wastewater characteristics obtained from Nozaic *et al*., (2009), von Sperling (2007a), and others, used for sizing the WSP.

The calculations and results of both Scenarios 1 and 2 are presented and discussed in the following Sections.

4.2 SCENARIOS 1 AND 2

During the Scenario 1 (feasibility design) stage, the information, namely the water flow rates (to and out of the pond system), water quality results, and pond depths are assumed based on Nozaic *et al*., (2009) and von Sperling (2007b), and these are tabulated in Table 2.2 and Table 2.17 in Chapter 2. The design approach and different models are discussed in detail in Chapters 2 and 3.

The assumptions regarding the pond geometry (depth, bottom and top widths, side slopes, and others) for the Scenario 1 stage are confirmed during the Scenario 2 stage in Section 4.3.5.1 below.

4.3 SCENARIO 1 STAGE

Table 4.1 presents the design inputs, which are the guidelines used for Scenario 1 to investigate the performance of the existing WSP at TARDI.

The ratio of COD/BOD (2:1) in domestic wastewaters is used to calculate the BOD of the influent to the WSP (van Niekerk *et al.*, 2009). In the present study, only the bacteria reduction (FC) and BOD loading are considered for the process design of the WSP.

Table 4.1: The design parameters used for Scenario 1 at TARDI WSP

Parameter	Unit	Quantity	Reference
Temperature, minimum	°C	12	("Umtata Climate Weather..." n.d.)
Temperature, maximum	°C	31	("Umtata Climate Weather..." n.d.)
Influent BOD concentration	mg/l	350	van Niekerk *et al.*, 2009
Effluent COD concentration	mg/l	75	DWA, 2013
Effluent BOD concentration	mg/l	37.5	
Influent faecal coliforms	FC/100 ml	4×10^7	van Niekerk *et al.*, 2009
Average Dry Weather Flow	kl/day	183	
(Q_{ADWF})	l/s	2.1	
Peak dry weather factor		3.5	Nozaic *et al.*, 2009
PDWF (Q_{PDWF})	kl/day	640	
	l/s	7.4	
Infiltration		15%	Nozaic *et. al.*, 2009
PWWF (Q_{PWWF})	kl/day	737	
	l/s	8.5	

4.3.1 AP for the A-F-M System for Scenario 1

Given that there is no existing AP in the study area, the calculations in Table 4.2 are done to check whether the AP would be required at the TARDI WSP and to determine the benefits thereafter. Three models of the AP (A-F-M system) were used for Scenario 1, and they are discussed in detail in Section 3.6.1.1.

(i) AP Model 1 for the A-F-M System

The assumed retention time of 2 days and ADWF of 183 kl/d are used to determine the pond volume of 366 m^3. Equation 2.7 is used to determine the volumetric BOD loading of 175 g BOD/m^3.d which is found to be between the acceptable range of 100–400g BOD/m^3.d.

(ii) AP Model 2 for the A-F-M System

When the average volumetric BOD loading of 250 g BOD/m^3.d is used in Equation 2.7, the volume is 256 m^3 for a retention time of 1.4 days. Both retention times are within the acceptable retention time range of 1–4 days for APs as recommended in Table 2.17 (Nozaic and Freese, 2009).

(iii) AP Model 3 for the A-F-M System

The volumetric BOD loading in Model 3 is found to be 140 g BOD/(m^3.d), and Equation 2.7 is used to determine the pond volume of 458 m^3. The retention time is calculated to be 2.5 days, which is between the acceptable retention time range of 1–4 days for APs.

4.3.1.1 BOD removal and faecal coliform reduction during Scenario 1 of the AP

The literature recommends a BOD removal between 60–85%, as summarised in Section 2.13. Table 2.11 shows the percentages of the BOD removals for the APs loaded with 250g BOD/m^3.d, and Table 2.12 table also indicates the BOD removal at various temperatures. Based on the above literature by Pescod and Mara (1988), Nozaic and Freese (2009), De Souza and Jack (2010) and Butler *et al*. (2015), the BOD removals (%) used in Model 1, Model 2, and Model 3 are 60%, 53 %, as interpolated from the values in Table 2.11, and 44 % (2T + 20) as shown in Table 2.12, respectively. The input and output parameters, and the parameters of the AP are indicated in Figure 4.1 below. The results of the three models used to design the AP for Scenario 1 are tabulated in Table 4.2.

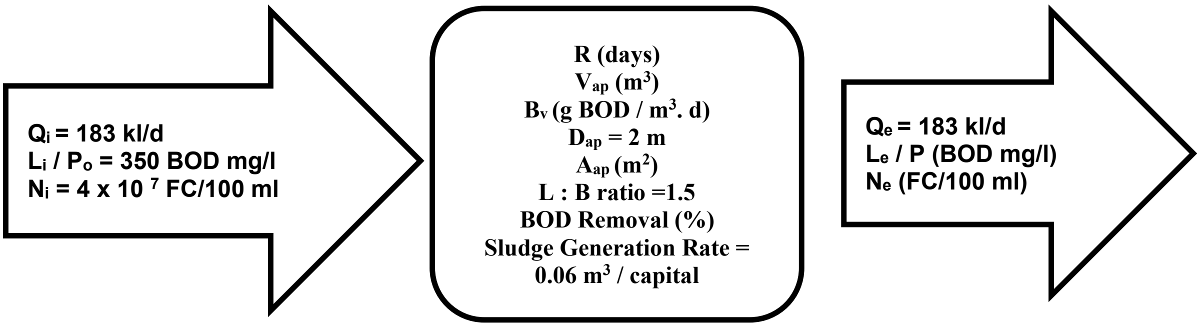


Figure 4.1: The schematic layout of the AP for the A-F-M system during Scenario 1.

Table 4.2: The design models for the AP during Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Reference
Retention time (R)	day	2*	1.4	**2.5**	
Influent BOD concentration	mg/l	350*	350*	**350***	
Volume (V_p)	m^3	**366**	256	**458**	
Volumetric BOD loading, (B_v)	g BOD/m^3 d	175	250*	**140**	Nozaic *et al.,* 2009
Depth (D_{ap})	m	2*	2*	**2***	Boyd and Mbelu, 2009
Area (A_{ap})	m^2	183	128	**229**	
Length to breadth ratio	m/m	1.5	1.5	**1.5**	Boyd and Mbelu, 2009
Breadth (b_p)	m	11.0	9.2	**12.3**	
Length (l_p)	m	16.6	13.9	**18.5**	
BOD removal	%	60*	52.7	**44**	Table 2.4, Table 2.11 and Table 2.12
Effluent BOD concentration	mg/l	140	165.7	**196**	
Sludge generation rate	m^3/ capita /year	0.06			Mara, 2018
Sludge generated / accumulated	m^3 /year	129.2			
Frequency of desludging	year	1.42	0.99	**1.77**	
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml	4×10^7	4×10^7	**4×10^7**	
1st order rate constant for FC removal	K_b	0.647	0.647	**0.647**	Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	1.74×10^7	2.10×10^7	**1.53×10^7**	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.

The following are the comments regarding the above three models for the AP (A-F-M system):

- The volumetric BOD loading of 200 g BOD/ (m^3 day) is the acceptable limit to prevent odour, and it can be determined based on the minimum retention time of 1.75 days. This retention time yields a pond volume of 320 m^3, which is obtained by making use of Model 2, and the required volumetric BOD loading changes to 200 g BOD/ (m^3 day).
- The above is done to investigate the maximum retention time and pond volume to the minimum volumetric BOD loading of 100 g BOD/ (m^3 day). The

maximum retention time and pond volume are found to be 3.5 days and 640 m^3, respectively.

- The two models (Models 1 and 3) are recommended to be used when designing and sizing the APs, hence it is important to ensure that the volumetric BOD loading is in the limit range of 100 – 400g BOD/m^3.d and to take note of the acceptable limit of 200g BOD/m^3.d to prevent odour.
- All models do not yield results that meet the DWS minimum requirement of 37.5 mg BOD/l, therefore the secondary treatment of the effluent from the AP is required.
- The above modelling of AP is done as a desk study whereby there is limited information on the pond geometry (length, width, depth, and side slope), design flows, and water quality test results. This information should be checked during the subsequent design stages of the WSP.

4.3.2 **Septic tank – Scenario 1**

The information, particularly the length and width of the septic tank, is not known for Scenario 1 and should be determined during the Scenario 2 stage. The future waste flow of 183 kl/d is used to determine the contribution population of 2 153. This is calculated by converting the average flow per dwelling of 510 l/dwelling, as calculated in Section 3.3.5, to 85 l/c/d, by considering 6 people per dwelling.

Equation 2.6 is used to size the septic tank, and the years between consecutive desludging is assumed to be 10 years, as shown in Table 4.3. The maximum desludging interval of 10 years has been chosen because the existing septic tank has not been de-sludged ever since it was commissioned. The minimum depth of 2 m, for a significantly large population, is used for the design of the septic tank as recommended by Nozaic, *et.al.* (2009).

Table 4.3: The design parameters of the septic tank for Scenario 1

Parameter	Unit	Quantity	Reference
Contribution Population	Capita	2153	
Flow per Capita	m^3/c/d	0.085	
Years between desludging	years	10	
Volume (V_{ST})	m^3	864	
Depth (d)	m	2	Boyd and Mbelu, 2009
Area (A_{ST})	m^2	432	
Length to breadth ratio	m/m	0.67	Boyd and Mbelu, 2009
Breadth (b_p)	m	12	
Length (l_p)	m	36	

4.3.3 PFP and SFP for Scenario 1

The results of the four models used to size the FP are discussed in the following sub-sections. These methods are discussed in detail in Section 2.9.2 and highlighted in Sections 3.6.1 and 3.6.1.2.

4.3.3.1 SFP of the FP with AP for the A-F-M system

The influent BOD and FC concentrations of the SFP (A-F-M system) are the respective effluent concentrations of the AP. These effluent concentrations (196 mg/l and 1.53 x 10^7 FC/100 ml respectively), from Model 3 in Table 4.2, are used to simplify the design of the SFP, and they are indicated in Figure 4.2 below.

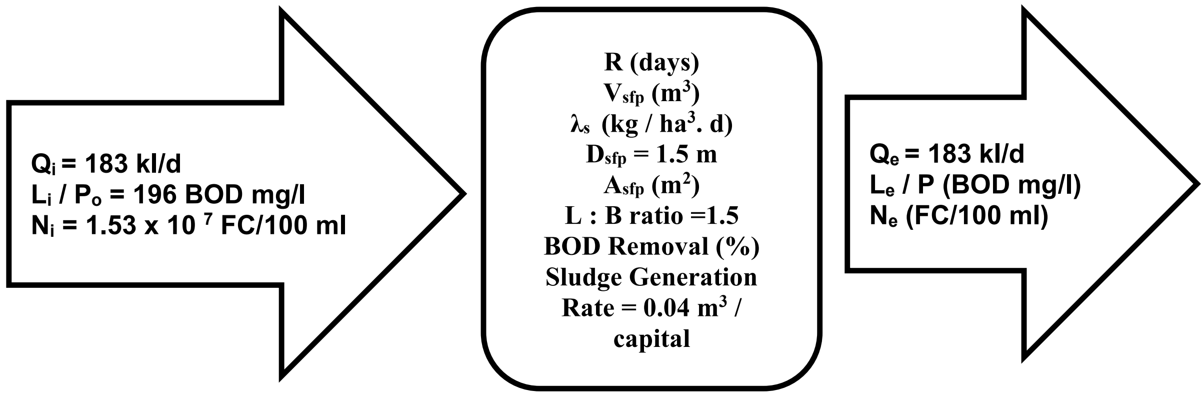
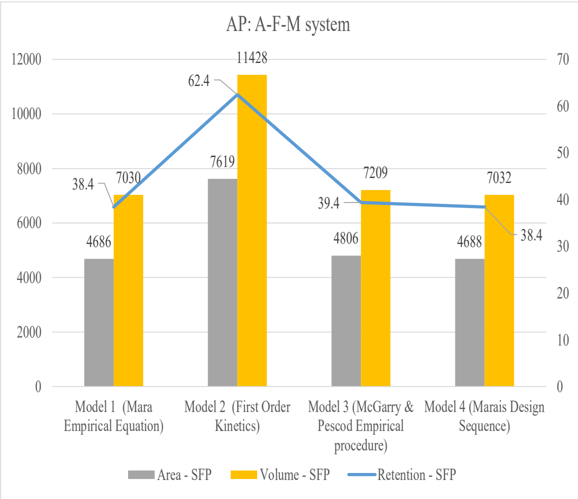


Figure 4.2: The schematic layout of the SFP for the A-F-M system for Scenario 1.

Table 4.4: The SFP of the FP with AP for the A-F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Model 4	Reference
Minimum Temperature (T)	^{0}C	12	12	12	12	("Umtata Climate Weather..." n.d.)
Influent BOD concentration	mg/l	196				
Surface loading (λ_s) or Arial Loading rate (λ_d)	kg/ ha/ day	124.1	N/A	120	N/A	Table 2.13
Area (A_{sfp})	m^2	2 889	7 619	2 989	4 688	
Depth	m	1.5*				
Volume (V_{sfp})	m^3	4 334	11 428	4 484	7 032	
Retention time (R)	day	23.7	62.4	24.5	38.4	
First order rate constant (k_{1T})	day^{-1}	0.068			N/A	Equation 2.11 and Equation 2.30
Effluent BOD concentration (L_e)	mg/l	75.3	37.5	73.7	37.5	Equation 2.29
BOD removal	%	70.3	86.2	71.0	79.3	Table 2.13
Length to breadth ratio	m/m	1.5*				Boyd and Mbelu, 2009
Breadth (b_p)	m	43.9	71.3	44.6	55.90	
Length (l_p)	m	65.8	106.9	67.0	83.9	
Sludge generation rate	m^3/ capita /year	0.04				
Sludge generated / accumulated	m^3 /year	86.1				
Frequency of desludging	year	25.2	66.4	26.0	40.8	
Bacterial Reduction						
Influent faecal coliforms	FC/100 ml	1.53 x 10^7				
1st order rate constant for FC removal	K_b	0.647				Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	9.37 x 10^5	3.70 x 10^5	9.08 x 10^5	5.92 x 10^5	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature in Section 2.13.



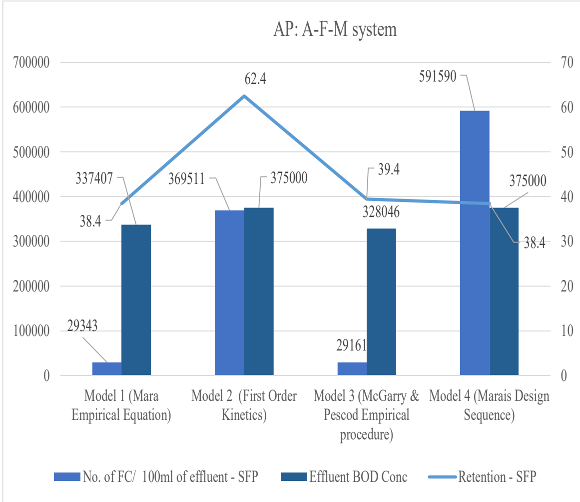


Figure 4.3: The comparisons of the area, volume, effluent concentrations, and retention time of the SFP, of the four different models, for the A-F-M system for Scenario 1.

The following are the comments regarding the above Models used for sizing the SFP (A-F-M system):

- The calculated retention times of all four models are found to be greater than the minimum of 4 days.
- The Model 1 (Mara Empirical Equation) and Model 3 (McGarry and Pescod Empirical procedure) do not meet the DWS minimum requirements of 37.5 mg/l when the volumes presented in Table 4.4 are used. Therefore, the retention time required should be increased, which, in the present study, resulted in the increase in the pond volume (refer to Table 4.5).
- There will be a requirement for further treatment to meet the DWS effluent minimum requirements when Models 1 and 3 are used. This means the designer will require additional FPs to reduce the BOD in the effluent before discharge.

    - Model 1:
        - Two additional ponds are required to ensure that the effluent concentration of 37.5 mg/l is met, as shown in Table 4.5.
        - The total pond area, volume, and retention time are calculated to be 4 686 m^2, 7 030 m^3, and 38.4 d, respectively. The sizing of Model 1 is similar to Model 4.

- The construction costs of Model 1 will be greater, compared with those of Models 2 and 4, as there will be a requirement to construct the three FPs.

- Model 3:
  - Two additional ponds are required to ensure that the effluent concentration of 37.5 mg/l is met, as shown in Table 4.5.
  - The total pond area, volume, and retention time are calculated to be 4 806 m^2, 7 209 m^3, and 39.4 d, respectively. The pond area, volume, and retention time are found to be larger than their Model 1 equivalents.
  - The effluent BOD and faecal coliforms (FC) also play a role in pond sizing. Table 4.5. below shows that the effluent BOD and FC of Models 1 and 2 are 33.7 mg BOD/l and 2.93 x 10^4 FC/100 ml and 32.8 mg BOD/l and 2.92 x 10^4 FC/100 ml, respectively. This implies that the greater the above parameters the better the water quality.
  - The construction costs of Model 3 will be greater, compared with those of Models 2 and 4, as there will be a requirement to design and build the three FPs.
  - In addition, the construction costs of Model 3 will be greater because the required volume is the higher, compared with those of Model 1, although the effluent water quality is better in Model 3.

- Figure 4.3 shows that:

  - Model 3 produces the best effluent quality, followed by Model 1, and then Models 2 and 4.
  - Models 1 and 4 have the same total retention time of 38.4 days, but different volumes of 7 030 m^3 and 7 032 m^3 and areas of 4 686 $m^2$and 4 688 m^2, respectively. However, Model 1 produces a better effluent quality than Model 4, which indicates that a series of pond units provides a better effluent quality compared with a single pond unit (Marais and

Shaw, 1961). This benefit of having the ponds in series is discussed briefly in Sections 2.8 and 2.9.2.1d).

- Model 4 (Marais design sequence) is the preferred model to be used for design as it shows an economical volume of 7 032 m^3, compared with the volume of Model 1 with three FPs of 7 030 m^3 in total, Model 2 of 11 428 m^3, and Model 3 with three FPs of 7 209 m^3 in total.
- There is a requirement for further treatment of the faecal coliforms if the assumed depth and width, indicated in Table 4.4 and Table 4.5, are found to be the same on site. Therefore, MPs are required. The pond geometry requires checking during the analysis of Scenario 2.

Table 4.5: The designs of the SFP with AP for the A-F-M system for Scenario 1 using Model 1 and Model 3

Parameter	Unit	Model 1			Total	Model 3			Total	Reference
		SFP 1	SFP 2	SFP 3		SFP 1	SFP 2	SFP 3		
Minimum Temperature (T)	^0C	12				12				
Influent BOD concentration	mg/l	196	75.3	46.6		196	73.7	45.4		
Surface loading (λ_s) or Arial Loading rate (λ_d)	kg/ ha/ day	124.1				120.0				Table 2.13
Area (A_{sfp})	m^2	2 889	1 110	687	**4 686**	2 989	1 124	692	**4 806**	
Depth (D_{sfp})	m	1.5*				1.5*				
Volume (V_{sfp})	m^3	4 334	1 665	1 030	**7 030**	4 484	1 687	1 039	**7 209**	
Retention time (R)	day	23.7	9.10	5.63	**38.4**	24.5	9.22	5.68	**39.4**	
First order rate constant (k_{1T})	day^{-1}	0.068				0.068				Equation 2.11 and Equation 2.30
Effluent BOD concentration (L_e)	mg/l	75.3	46.6	33.7		73.7	45.4	32.8		Equation 2.29
BOD removal	%	70.3	47.6	36.0		71.0	48.0	36.2		Table 2.13
Length to breadth ratio	m/m	1.5*				1.5*				Boyd and Mbelu, 2009
Breadth (b_p)	m	43.9	27.2	21.4		44.6	27.4	21.5		
Length (l_p)	m	65.8	40.8	32.1		67.0	41.1	32.2		
Sludge generation rate	m^3/ capita /year	0.04				0.04				
Sludge generated / accumulated	m^3 /year	86.1				86.1				
Frequency of desludging	year	25.2	9.67	5.98	**40.8**	26.0	9.79	6.03	**41.9**	
Bacterial Reduction										
Influent faecal coliforms	FC/100 ml	1.53×10^7	9.37×10^5	1.36×10^5		1.53×10^7	9.08×10^5	1.35×10^5		
1st order rate constant for FC removal	K_b	0.647				0.647				Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	9.37×10^5	1.36×10^5	2.93×10^4		9.08×10^5	1.35×10^5	2.92×10^4		Equation 2.15 , Equation 2.30 and Equation 2.33

* Value assumed based on literature in Section 2.13.

4.3.3.2 **PFP without AP for the F-M system**

The sizing of the PFPs is done and designed given that there is no AP. The influent BOD concentration and depth of the PFP (F-M system) are assumed to be 350 mg/l and 2 m, respectively. Figure 4.4 below shows both the required parameters and quantities used to perform the PFP sizing during Scenario 1 stage.

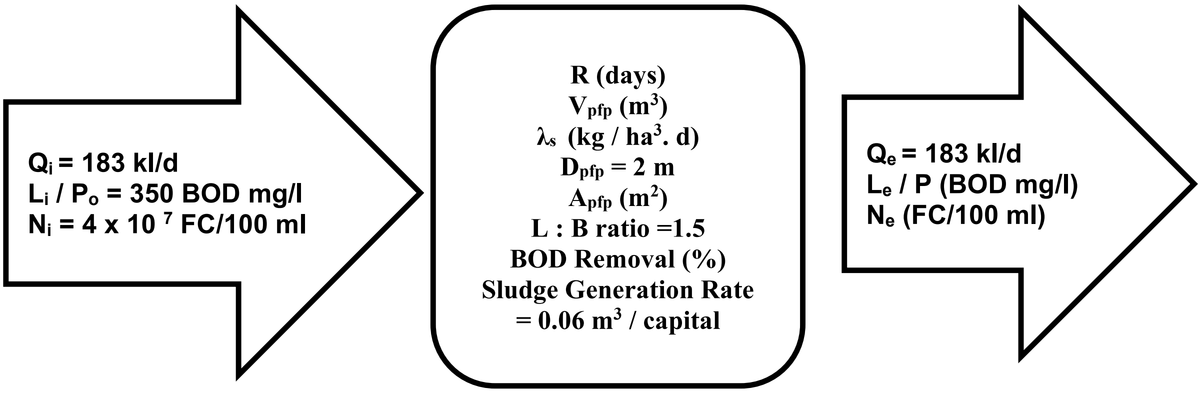
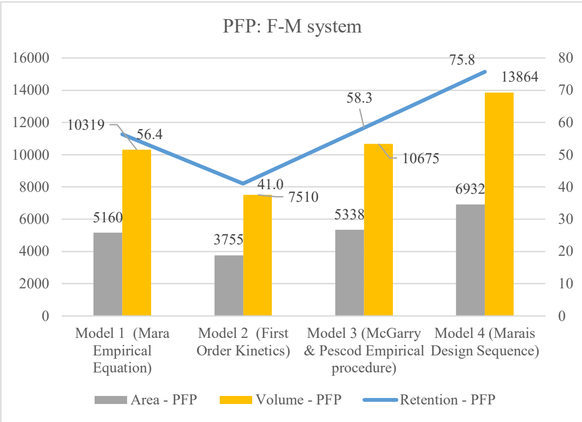


Figure 4.4: The schematic layout of the PFP for the F-M System for Scenario 1.

Table 4.6: The PFP of the FP without AP for the F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Model 4	Reference
Minimum Temperature (T)	^{0}C	12				
Influent BOD concentration	mg/l	350*				
Surface loading (λ_s) or Arial Loading rate (λ_d)	kg/ ha/ day	124.1	N/A	120	N/A	Table 2.13
Area (A_{pfp})	m^2	5 160	3 755	5 338	6 932	
Depth	m	2*	2*	2*	2*	
Volume (V_p)	m^3	10 319	7 510	10 675	13 864	
Retention time (R)	day	56.4	41.0	58.3	75.8	
First order rate constant (k_{1T})	day^{-1}	0.203	0.203	0.203	N/A	Equation 2.11 and Equation 2.30
Effluent BOD concentration (L_e)	mg/l	28.1	37.5	27.3	37.5	Equation 2.29
BOD removal	%	94.4	92.5	94.6	95.8	Table 2.13
Length to breadth ratio	m/m	1.5*	1.5*	1.5*	1.5*	Boyd and Mbelu, 2009
Breadth (b_p)	m	58.6	50.0	59.7	68.0	
Length (l_p)	m	88.0	75.1	89.5	102.0	
Sludge generation rate	m^3/ capita /year	0.06				
Sludge generated / accumulated	m^3 /year	129.2				
Frequency of desludging	year	39.94	29.07	41.32	53.66	
Bacterial Reduction						
Influent faecal coliforms	FC/100 ml	4×10^7	4×10^7	4×10^7	4×10^7	
1st order rate constant for FC removal	K_b	0.647	0.647	0.647	0.647	Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	1.07×10^6	1.45×10^6	1.03×10^6	8.00×10^5	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.



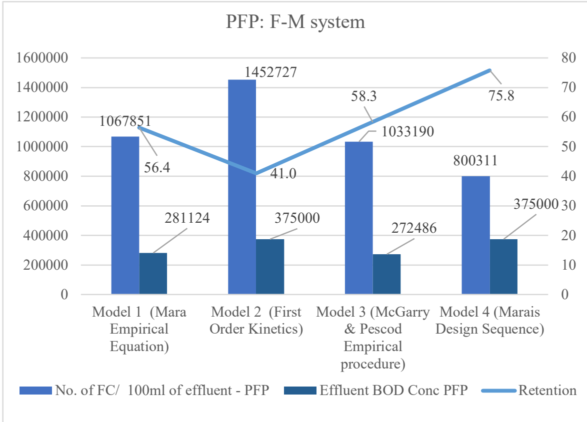


Figure 4.5: The comparisons of the area, volume, effluent concentrations, and retention times of the four different models for the PFP for the F-M System for Scenario 1.

The following are the comments regarding the models used in Table 4.6 for sizing the PFP (F-M system):

- All models meet the DWS minimum effluent BOD requirement of 37.5 mg BOD/l when the volume tabulated above is used for each model. Therefore, no further effluent treatment is required to reduce the BOD loading.
- Model 4 (Marais design sequence) is the preferred model to be used for design given that it yields the greatest design volume of 13 864 m^3, during the winter season, of all four models.
- The maximum temperature should not be used when sizing the primary FPs. For example, Model 1 (Mara Empirical Equation) will require an area of 1 405 m^2 during the summer season. This pond design area will not meet the required pond area of 5 160 m^2, which is calculated based on the winter season temperature, given that it will be small. Hence, the pond will discharge high BOD concentrations that will require further treatment.
- Figure 4.5 shows that:
    - Model 4 has the least effluent FC, followed by Model 3, then Models 1 and 2. On the other hand, Model 3 has the least effluent BOD concentration, followed by Model 1, then Model 2 and 4.
- The PFP will be overloaded if the pond design is based on the summer season. Also, this will influence the performance of the subsequent pond units.

- There is no requirement for further treatment of the effluent BOD from the PFP if the assumed depth and width indicated in Table 4.6 are found to be same on site. This will require checking for Scenario 2.

4.3.4 MP for Scenario 1

The design of the MP is based on the model by Marais (1974), von Sperling (2007b), and Nozaic and Freese (2009). The retention of the MP is calculated by using Equation 2.33 where n is equal to 1,2,3,4, and 5.

4.3.4.1 MP with AP for the A-F-M system

Three models as used for sizing the MPs, as discussed in detail in Section 3.6.1.1. Figure 4.6 and Figure 4.8 indicate the input and output parameters used for pond sizing. The influent parameters of the PMPs are the effluent parameters of the SFP (A-F-M system) of Model 4. The results of the three models used for the design of the MPs for Scenario 1 are tabulated in Table 4.7, Table 4.8, and Table 4.9.

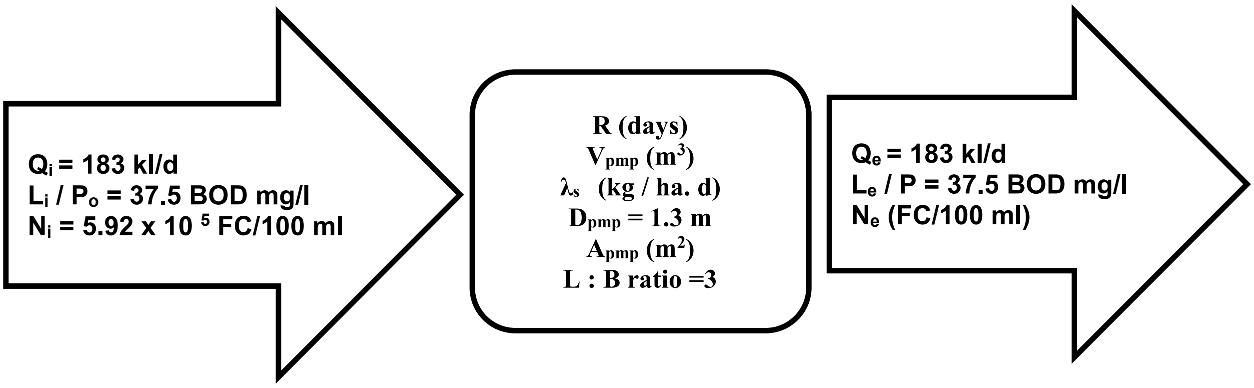


Figure 4.6: The schematic layout of the PMP for the A-F-M System for Scenario 1.

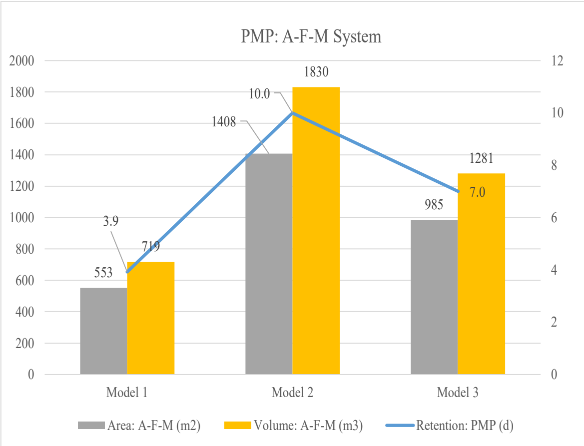
Three models are used as shown in Table 4.7 and Table 4.8, and these models are briefly discussed in Section 3.6.1.1.

- Model 1:
    - The BOD loading in the first MP is calculated to be 166.1 kg/ ha/ day and found to be higher than the BOD loading of 124.1 kg/ ha/ day in the SFP.
    - The BOD loading in the PMP is adjusted to 124.1 kg/ ha/ day given that the calculated BOD loading is found to be higher than the BOD loading in the SFP. The adjustment is only done when the BOD load in the PMP is found to be higher than the loading in the SFP upstream.
    - The revised retention time is recalculated to be 3.9 days, and it is used to get the volume of 719 m^3 of the PMP.
    - The first step of using Equation 2.33 is repeated to determine the number of the SMPs.
    - The retention time of 3 days is used to determine the required number of five SMPs.
    - The effluent FC (761.5 FC/100 ml) at the last SMP is compared with the limit guideline, as specified by the DWS, by using Equation 2.32 and Equation 2.33.

- Model 2:
    - The assumed retention time of 10 days, for all the MPs, is used in Equation 2.32 and Equation 2.33 to calculate the effluent FC at the last SMP.
    - The Equation 2.31 and Equation 2.32 are used to calculate the effluent FC of each SMP.
    - The effluent FC of 190.5 FC/100 ml at the last SMP is found to meet the required minimum limit guideline of the DWS.

- Model 3:
    - The minimum retention time of 7 days for the MP is assumed similar for all the MPs.

- The Equation 2.31 and Equation 2.32 are used to calculate the effluent FC of each MP until the effluent FC of the last SMP meets the DWS limit guidelines.
- The fourth MP is found to have an effluent FC of 634.5 FC/100 ml.



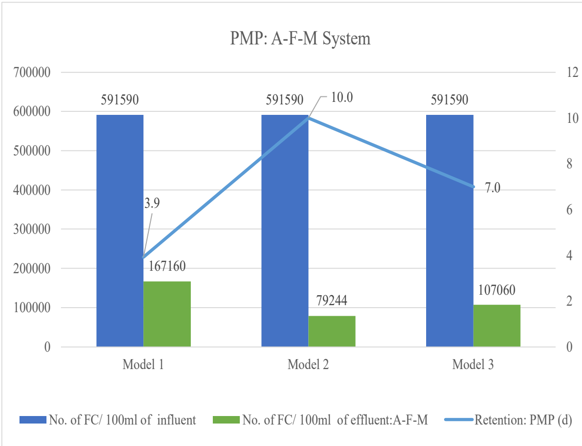


Figure 4.7: The designed area, volume, and retention time, and the influent and effluent FC of the PMP for the A-F-M system for Scenario 1.

Figure 4.7 shows that Model 1 requires the least area, volume, and retention time although the effluent concentration is higher compared with the other two models (Models 2 and 3). This means that a longer retention time will require greater volume and area for the PMP (A-F-M system), and the effluent concentration (FC) will be improved.

Table 4.7: The PMP of the MP with AP for the A-F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Reference
Minimum Temperature (T)	^{0}C	12	12	12	("Umtata Climate Weather..." n.d.)
Surface loading (λ_m) or Arial Loading rate (λ_d)	kg/ ha/ day	124.1	N/A	N/A	Table 2.13
Retention time (R)	day	3.9	10*	7*	
Volume (V_{pmp})	m^3	719	1 830	1 281	
Depth	m	1.3*			
Area (A_{pmp})	m^2	553	1 408	985	
Length to breadth ratio	m/m	3*			Boyd and Mbelu, 2009
Breadth (b_p)	m	13.6	21.7	18.1	
Length (l_p)	m	40.7	65.0	54.4	
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml	5.92×10^5			
1st order rate constant for FC removal	K_b	0.647			Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	1.67×10^5	7.92×10^4	1.07×10^4	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.

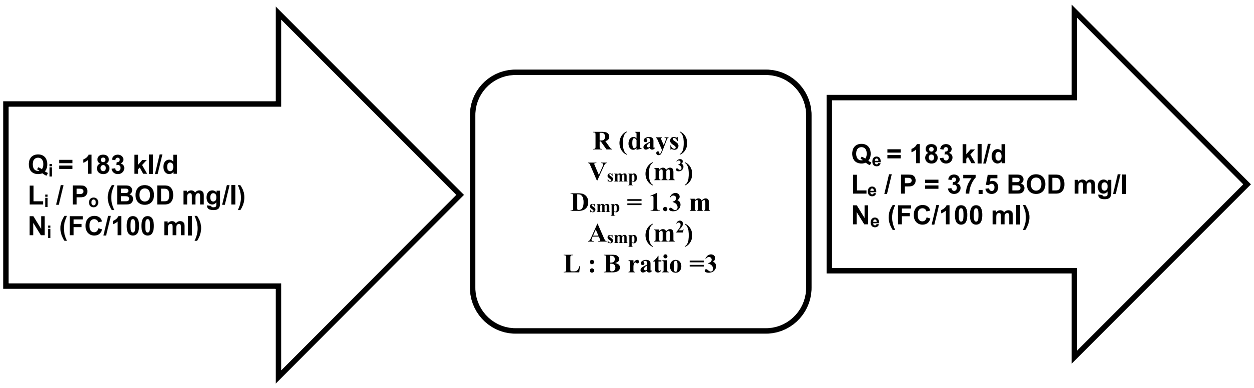
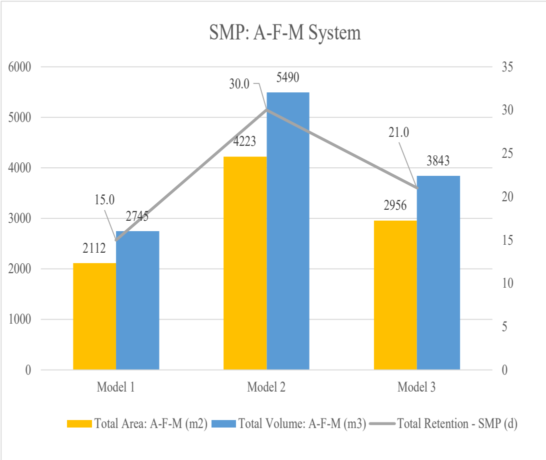


Figure 4.8: The schematic layout of the SMP for the A-F-M system for Scenario 1.



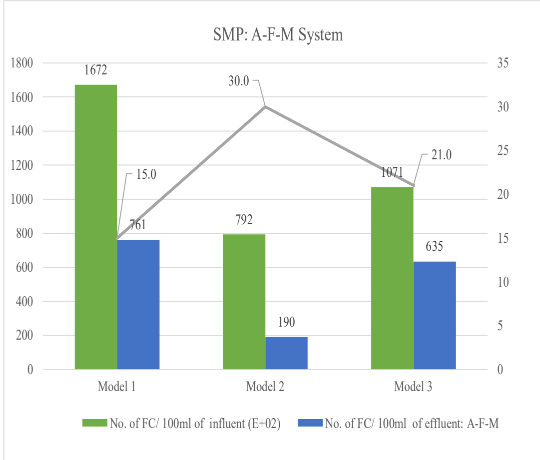
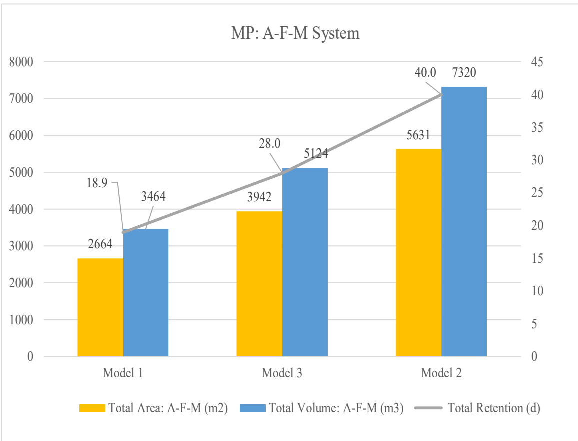


Figure 4.9: The required total area, volume, and retention time, and the influent and effluent FC of the SMP for the A-F-M system for Scenario 1.

Figure 4.9 above, Table 4.8, and Table 4.9 indicate that Model 1 of the SMP (A-F-M system) requires a smaller land (2 112 m^2) compared with the other two models, but the effluent qualities of Model 1 of 761.5 No. FC/100 ml are worse than those of Models 2 and 3 of 190.5 and 634.5 No. FC/100 ml, respectively. The SMP (Model 2), with a higher retention time than Models and 3, produces the best effluent quality in terms of FC.



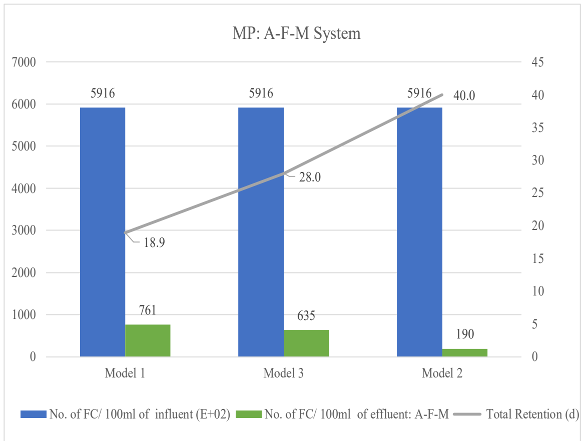


Figure 4.10: The required total area, volume, retention time, and the influent and effluent FC of the MP for the A-F-M system for Scenario 1.

It is found inFigure 4.9 and Table 4.9 that Model 1 of the MP (A-F-M system) requires a smaller land (2 664 m^2) compared with the other two models. Model 1 of the MP yields an effluent quality of 761.5 No. FC/100 ml, as discussed above.

Table 4.8: The SMP of the MP with AP for the A-F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Reference
Minimum Temperature (T)	^{0}C	12	12	12	
Retention time (R)	day	3	10*	7*	
Number of secondary MPs required	n	5	3	3	
Volume (V_{pmp})	m^3	549	1 830	1 281	
Depth	m	1.3*			
Area (A_{pmp})	m^2	422	1 408	985	
Total Area (A_{smp})	m^2	2 112	4 223	2 956	
Length to breadth ratio	m/m	3*			Boyd and Mbelu, 2009
Breadth (b_p)	m	11.9	21.7	18.1	
Length (l_p)	m	35.6	65.0	54.4	
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml	1.67×10^5	7.92×10^4	1.07×10^4	
1st order rate constant for FC removal	K_b	0.647			Equation 2.17 and Equation 2.31
Effluent faecal coliforms at the last pond	FC/100 ml	761.5	190.5	634.5	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.

Table 4.9 The summary of the MP with AP for the A-F-M system for Scenario 1

Parameter		Unit	Model 1	Model 2	Model 3
PMP	Retention time (R)	day	3.9	10	7
	Volume (V_{pmp})	m^3	719	1 830	1 281
	Area (A_{pmp})	m^2	553	1 408	985
SMPs	Retention time (R)	day	15	30	21
	Total Volume (V_{smp})	m^3	2 745	5 490	3 843
	Total Area (A_{smp})	m^2	2 112	4 223	2 956
TOTAL	Retention time (R)	day	18.9	40	28
	Total Volume (V_{mp})	m^3	3 464	7 320	5 124
	Total Area (A_{mp})	m^2	2 664	5 631	3 942

Table 4.9 above indicates that Model 1 requires a smaller land (2 664 m^2) compared with the other two models, implying that Model 1 is cost effective and therefore will be used for MP sizing during Scenario 2 in Section 4.4.5.

4.3.4.2 **MP without AP for the F-M system**

Three models are used as shown in Table 4.10, Table 4.11, and Table 4.12, and they are found to be the same models as used for the MP with AP, as discussed in Section 4.3.4.1. The same procedure indicated in the previous sub-section is followed for all three models, including the design parameters that are presented in Table 4.10 to Table 4.12. Figure 4.11 and Figure 4.13 indicate the input and output parameters used for the MP sizing. The influent parameters of the PMPs are the effluent parameters of the PFP (F-M system) of Model 4.

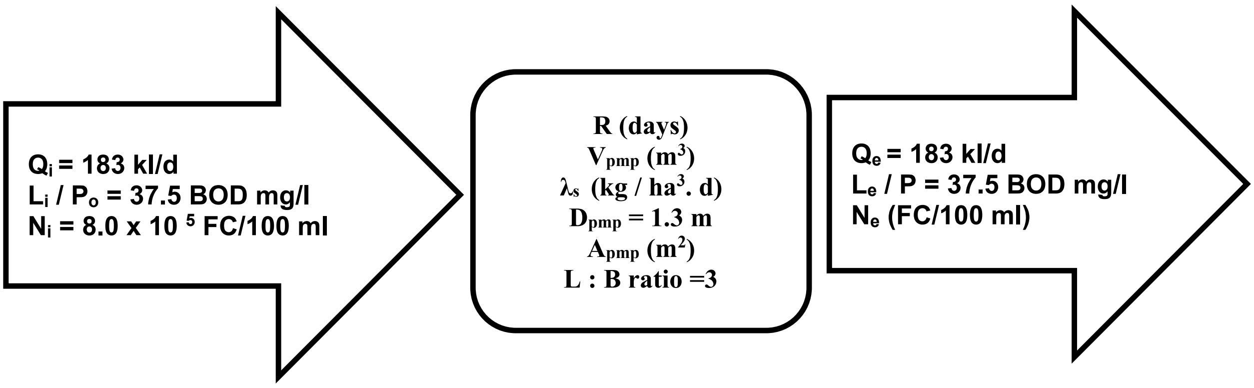
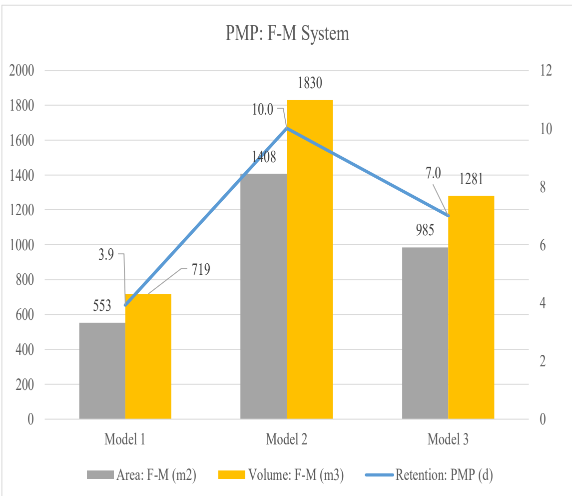


Figure 4.11: The schematic layout of the PMP for the F-M System for Scenario 1.



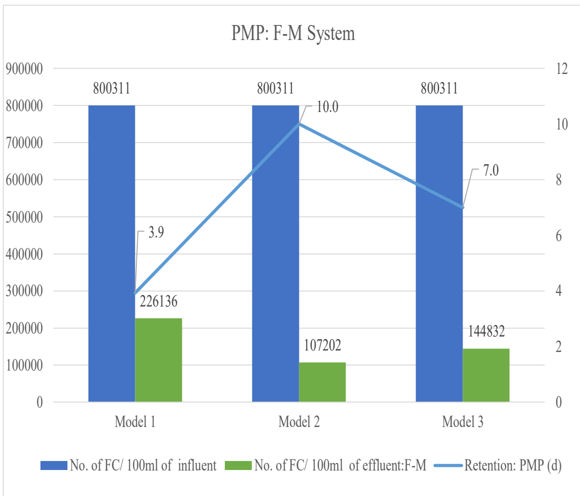


Figure 4.12: The designed pond area, volume, retention time, and the influent and effluent FC of the PMP for the F-M system for Scenario 1.

Figure 4.12 above shows that Model 1 requires the least area, volume, and retention time, of all three models, although the effluent concentration is higher compared with the other two models. This follows the same trend as the PMP for the A-F-M system viz. the longer the retention time the larger the pond volume and area, which will also improve the FC effluent concentration.

Table 4.10: The PMP of the MP without AP for the F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Reference
Surface loading (λ_m) or Arial Loading rate (λ_d)	kg/ ha/ day	124.1	N/A	N/A	Table 2.13
Retention time (R)	day	3.9	10*	7*	
Volume (V_{pmp})	m^3	719	1 830	1 281	
Depth	m		1.3*		
Area (A_{pmp})	m^2	553	1 408	985	
Length to breadth ratio	m/m		3*		Boyd and Mbelu, 2009
Breadth (b_p)	m	13.6	21.7	18.1	
Length (l_p)	m	40.7	65.0	54.4	
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml		8.00×10^5		
1st order rate constant for FC removal	K_b		0.647		Equation 2.17 and Equation 2.31
Effluent faecal coliforms	FC/100 ml	2.26×10^5	1.07×10^5	1.45×10^5	Equation 2.15, Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.

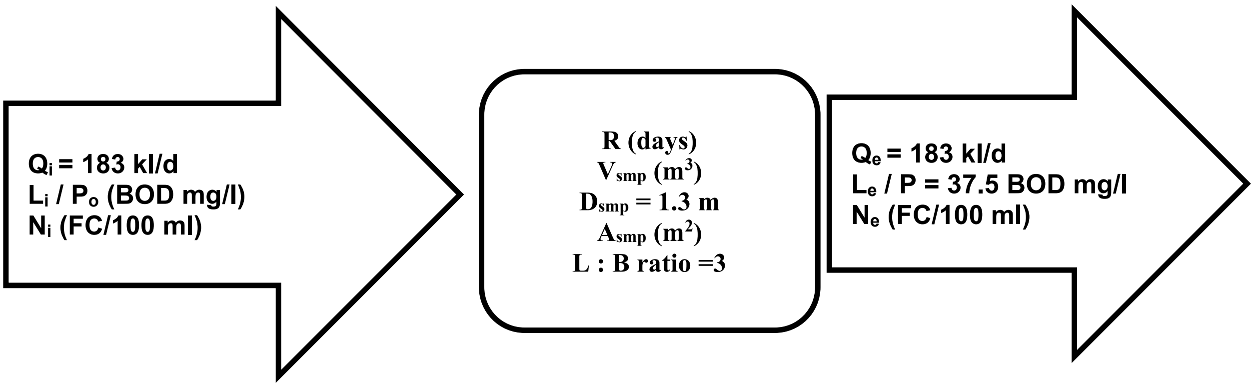


Figure 4.13: The schematic layout of the SMP for the F-M System for Scenario 1.

Table 4.11: The SMP of the MP without AP for the F-M system for Scenario 1

Parameter	Unit	Model 1	Model 2	Model 3	Reference
Minimum Temperature (T)	^{0}C	12	12	12	
Retention time (R)	day	3.03	10*	7*	
Number of secondary MPs required	n	5	3	3	
Volume (V_{smp})	m^3	555	1 830	1 281	
Depth	m	1.3*			
Area (A_{smp})	m^2	427	1 408	985	
Total Area (A_{smp})	m^2	2 133	4 223	2 956	
Length to breadth ratio	m/m	3*			Boyd and Mbelu, 2009
Breadth (b_p)	m	11.9	21.7	18.1	
Length (l_p)	m	35.8	65.0	54.4	
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml	2.26 x 10^5	1.07 x 10^5	1.45 x 10^5	
1st order rate constant for FC removal	K_b	0.647			Equation 2.17 and Equation 2.31
Effluent faecal coliforms at the last pond	FC/100 ml	996.8	257.7	858.4	Equation 2.15 , Equation 2.30 and Equation 2.33

* Value assumed based on literature discussed in Section 2.13.

Table 4.12: The design summary of the MP without AP for the F-M system for Scenario 1

Parameter		Unit	Model 1	Model 2	Model 3
PMP	Retention time (R)	day	3.9	10	7
	Volume (V_{pmp})	m^3	719	1 830	1 281
	Area (A_{pmp})	m^2	553	1 408	985
SMP	Retention time (R)	day	15.2	30	21
	Total Volume (V_{smp})	m^3	2 772	5 490	3 843
	Total Area (A_{smp})	m^2	2 133	4 223	2 956
TOTAL	Retention time (R)	day	19.1	40	28
	Total Volume (V_{mp})	m^3	3 491	7 320	5 124
	Total Area (A_{mp})	m^2	2 686	5 631	3 942

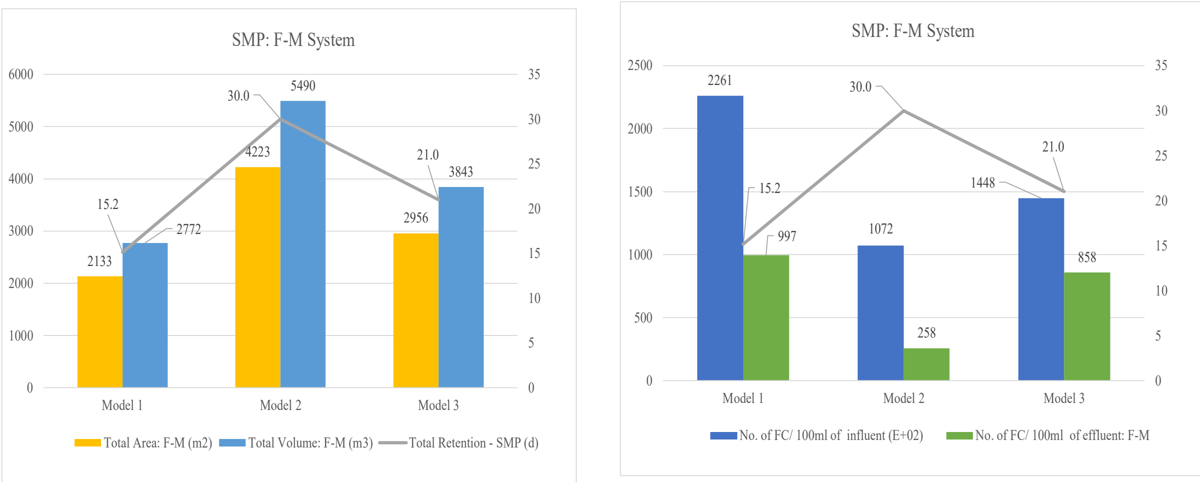


Figure 4.14: The total pond area, volume, retention time, and the influent and effluent FC of the SMP without AP for the F-M system for Scenario 1.

Figure 4.14, Table 4.11, and Table 4.12 indicate that Model 1 of the SMP (F-M system) yields the smallest area of 2 133 m^2 of all three models, but the effluent quality of Model 1 of 996.8 No. FC/100 ml is worse than those of Models 2 and 3 of 257.8 and 858.4 No. FC/100 ml, respectively. The SMP (Model 2), with a higher retention time of 30 days, produces the best FC effluent quality.

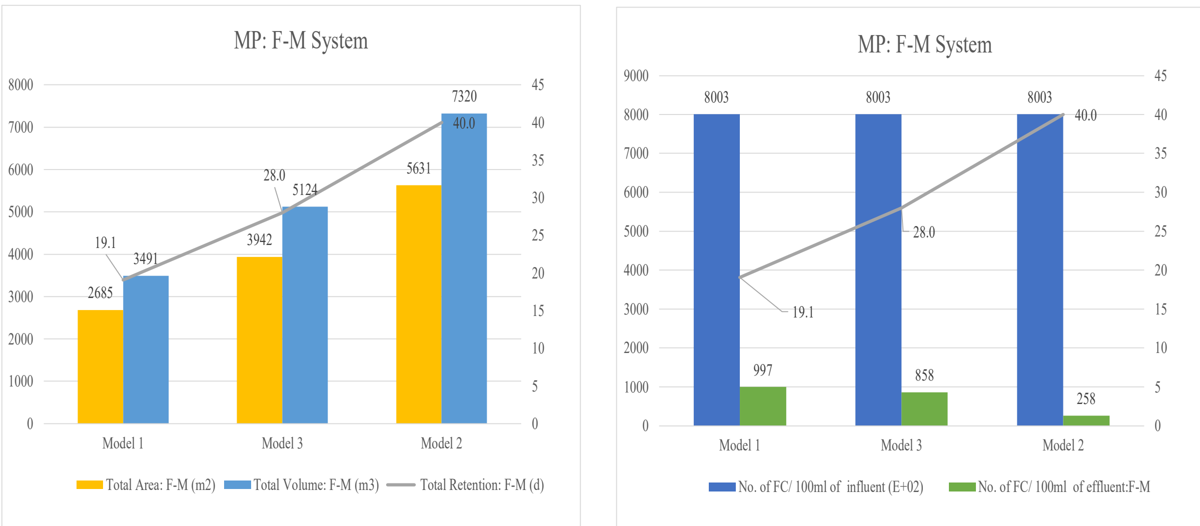


Figure 4.15: The total pond area, volume, retention time, and the influent and effluent FC of the MP without AP for the F-M system for Scenario 1.

It is found in Figure 4.15 and Table 4.12 that Model 1 of the MP for the F-M system requires a smaller land (2 685 m^2) compared with the other two models. The comparison of the footprints of the MPs for the A-F-M and F-M systems is discussed further in Sections 4.3.5.3 to 4.3.5.5.

4.3.5 Summary of pond sizing for both the A-F-M and F-M systems for Scenario 1

Figure 4.16 and Table 4.13 give a summary of pond sizing for Scenario 1 of both the A-F-M and F-M systems, which is based on Models 1 for MPs, 3 for AP and 4 for FP. The relationship and mechanisms of operation of the A-F-M and F-M systems are summarised below and detailed in Sections 4.3.5.1 to 4.3.5.5.

1. The modelling for Scenario 1 of both the A-F-M and F-M systems is undertaken as a desk study whereby there is limited information on pond geometries (length, width, depth, and side slope), design flows, and water quality tests.
2. The total pond area, volume, and retention time required for both the A-F-M and F-M systems, from Figure 4.16 and Table 4.13, are calculated to be 7 580 m^2, 10 950 m^3, and 60 d, and 9 620 m^2, 17 360 m^3, and 95 d, respectively.
3. The effluent BOD is found to be 37.5 mg BOD/l for both the A-F-M and F-M systems. However, the effluent faecal coliforms concentration (Effluent concentrations from the Scenario 1 models of both the A-F-M and F-M systems) for Model 1 of the MP (A-F-M system) is found to be 761.5 FC/100 ml, which is less than the effluent faecal coliforms concentration of the MP (F-M system) of 996.8 FC/100 ml, see Figure 4.10, Figure 4.15 and Figure 4.23.
4. The A-F-M system is found to be the cost-effective model, based on the required pond volume indicated above, although most operators find it hard to operate and maintain the AP in practice. The result of the AP being compromised given that it does not operate as per the designs will lead to a rise in the sludge blanket, which if not de-sludged, will lead to the effluent quality not meeting the DWS discharge guidelines.
5. The operation and maintenance issues indicated above are to be considered when choosing between the A-F-M and F-M systems.

Table 4.13: The total pond footprint, capacity, and effluent quality of the A- F-M and F-M systems for Scenario 1

Parameter	Unit	A-F-M System	F-M System
Inflow and outflow (Q)	kl/day	183	183
Influent BOD concentration	mg/l	350	350
Influent faecal coliforms	FC/100 ml	4×10^7	4×10^7
Area (A_p)	m^2	7 581	9 617
Volume (V_p)	m^3	10 953	17 355
Retention time (R)	day	59.9	94.8
Effluent BOD concentration	mg/l	37.5	37.5
Effluent faecal coliforms	FC/100 ml	761.5	997
BOD removal	%	89.3	89.3
Faecal coliforms removal	%	99.998	99.998

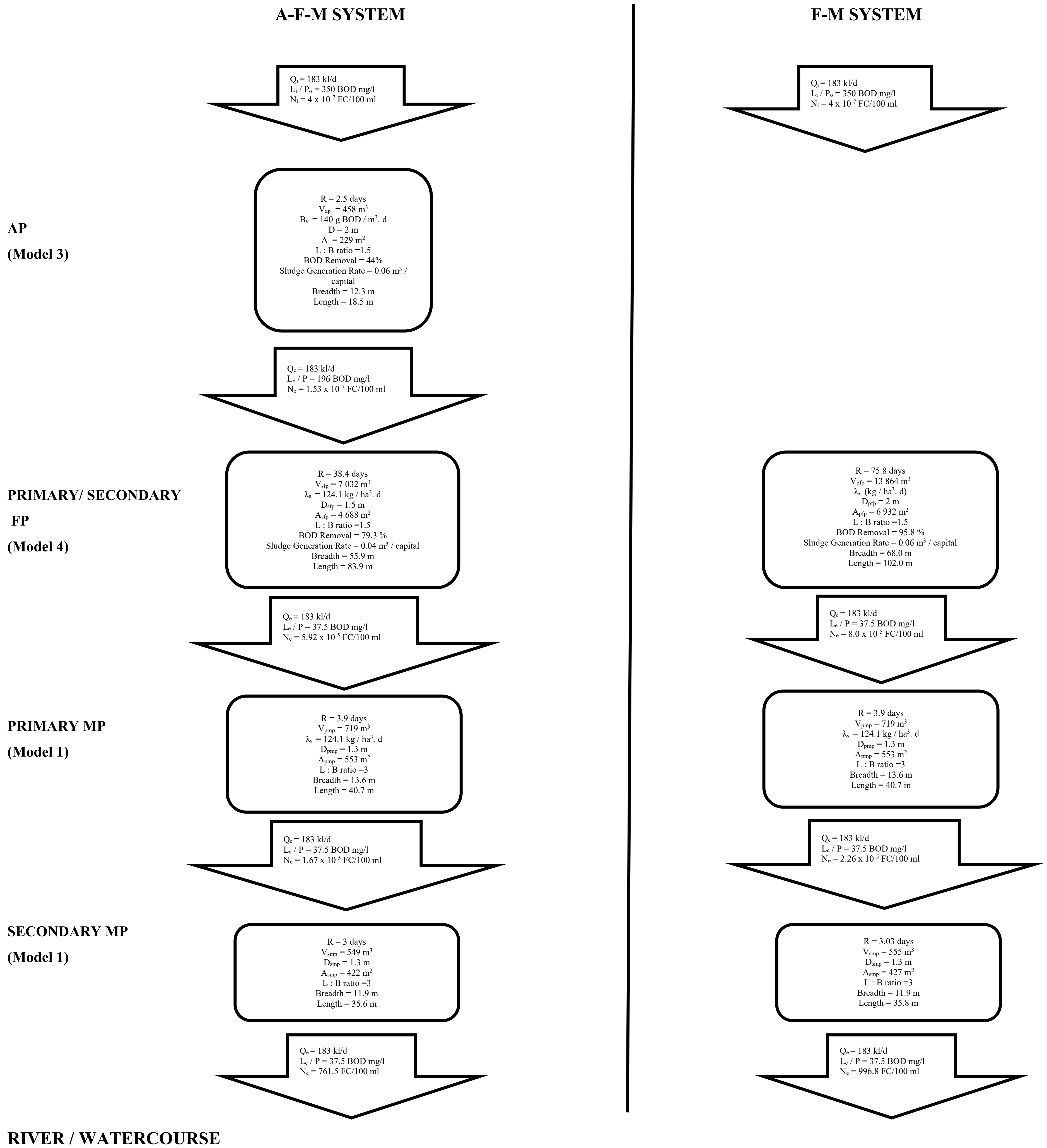


Figure 4.16: The schematic layout indicating the pond sizes of the units and differences between the A- F-M and F-M Systems for Scenario 1.

4.3.5.1 **APs during Scenario 1**

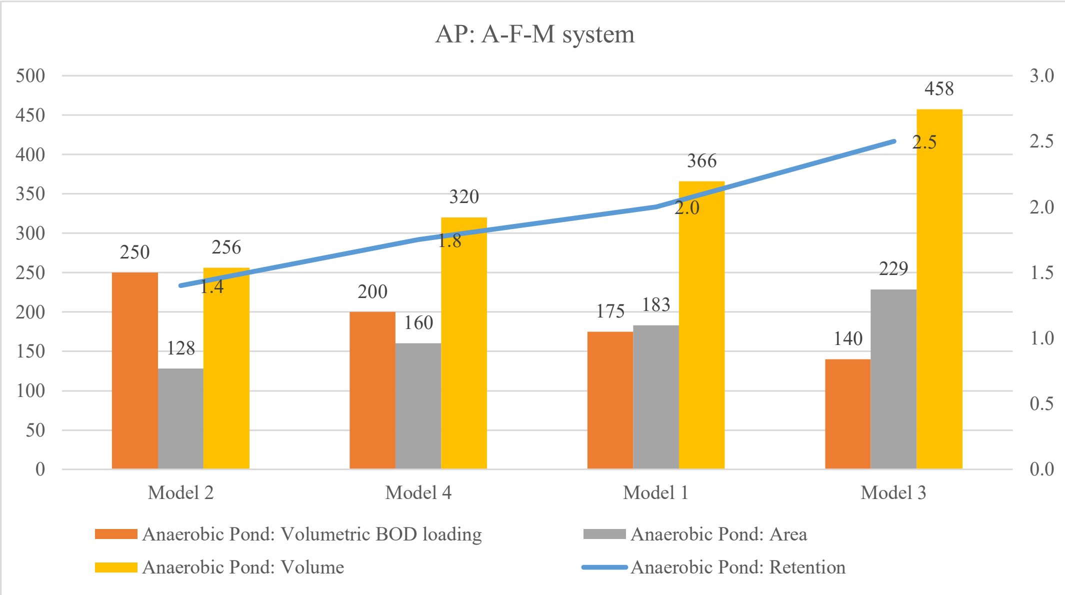


Figure 4.17: The relationship between the retention time, area, volume, and volumetric loading of the APs for the different models for Scenario 1.

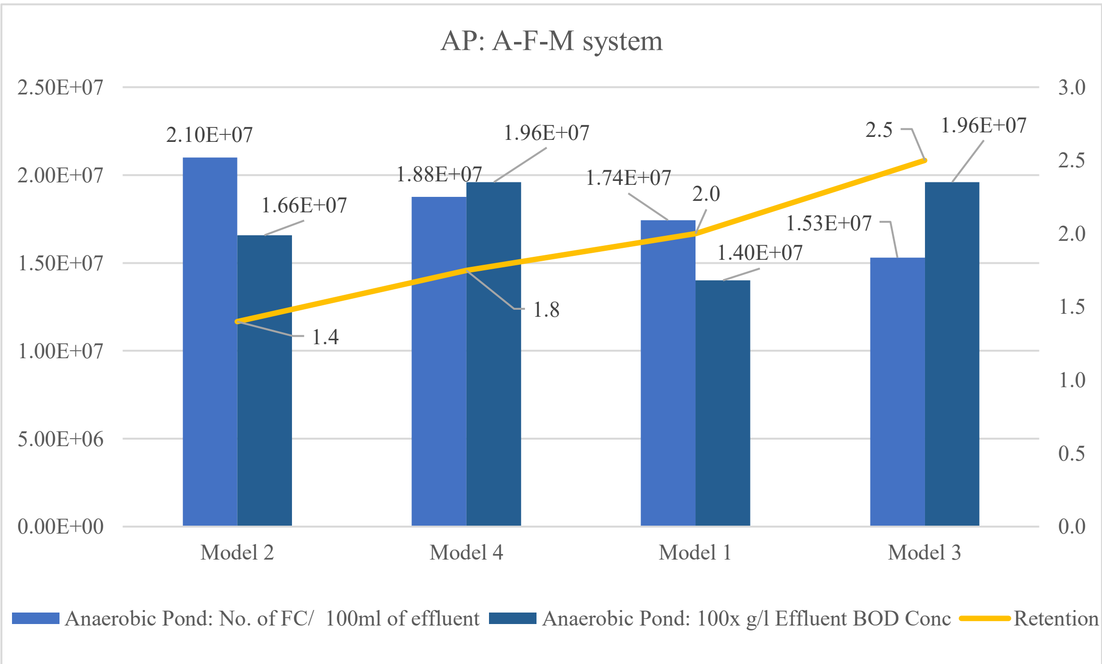


Figure 4.18: The relationship between the retention, effluent BOD in 100 x g/l, and FC of the AP.

Figure 4.17 indicates that the increase in the retention time of the AP will decrease the volumetric BOD loading, and there will be an increase in both the pond area and volume. Figure 4.18 shows that an increase in the retention time of the AP improves the effluent BOD and FC qualities, whereas the effluent BOD concentration depends on the surrounding temperature. The percentages of the BOD removal with respect to Models 1, 2, and 3 are discussed in detail in Section 4.3.1.1.

4.3.5.2 **PFPs and SFPs for Scenario 1**

This section focuses on the pond geometry for both the PFPs and SFPs, using the four models, for Scenario 1. It also focuses on the influence on the effluent quality of both A-F-M and F-M systems (Figure 4.19 and Figure 4.20). The depths of 1.5 m and 2 m of the SMP and PMP, respectively, are used.

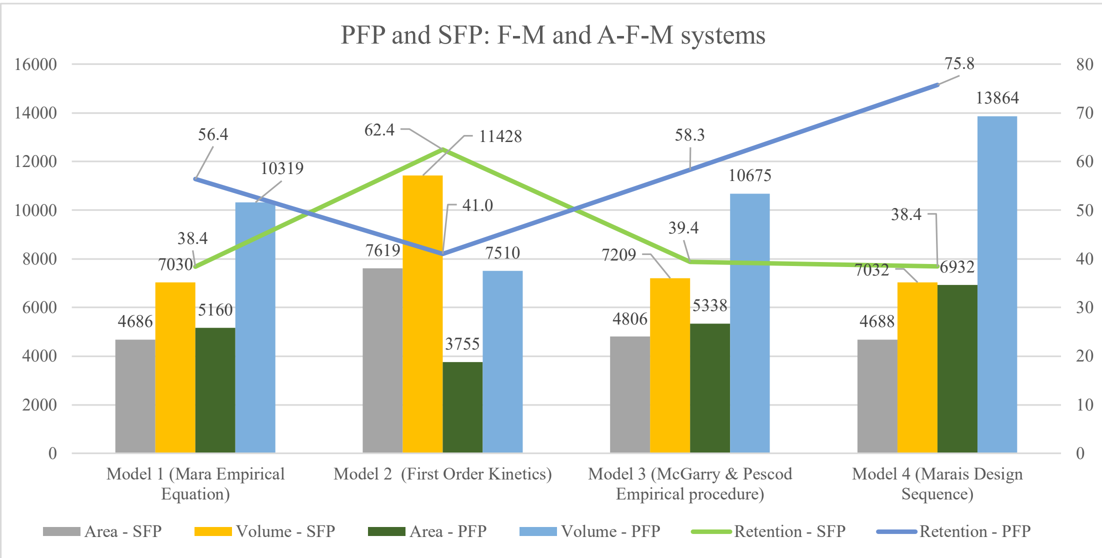


Figure 4.19: The comparison of the pond geometry (area and volume) between the PFP and SFP for Scenario 1.

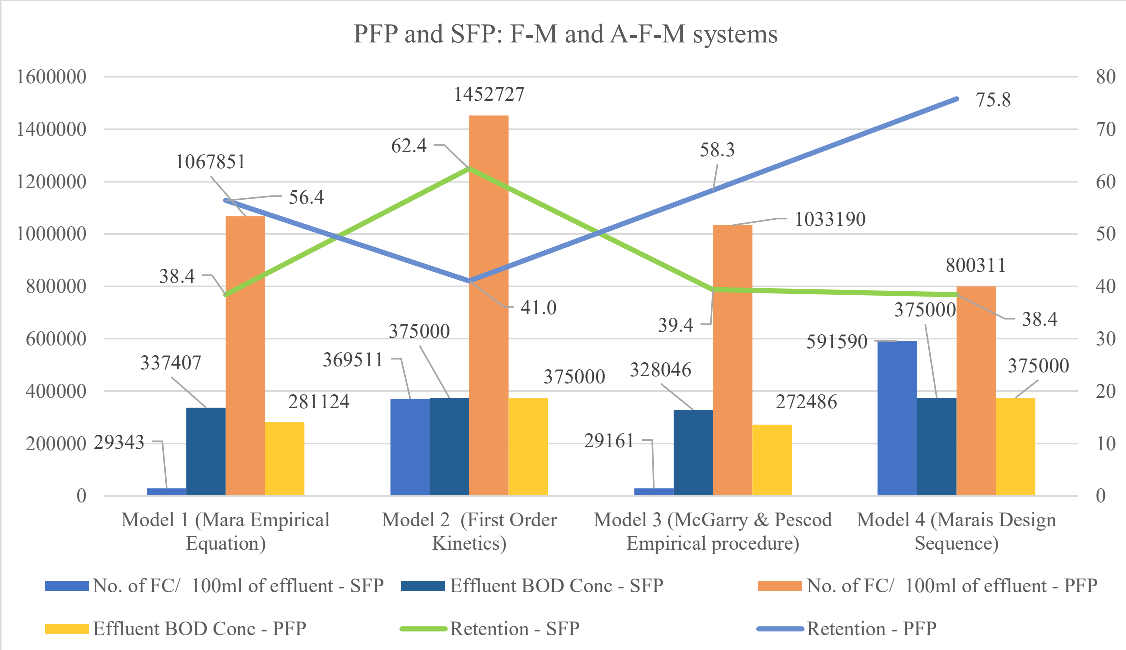


Figure 4.20: The relationship of the retention time, effluent BOD in 100 x g/l, and FC between the PFP and SFP for Scenario 1.

The following are the comparisons, between the PFP (F-M system) and SFP (A-F-M system) for Scenario 1, including the findings, which refer to Figure 4.19 and Figure 4.20 above:

- The retention times for Models 1, 3 and 4 of the PFP, of the F-M system, are found to be longer compared with those of the SFP, of the A-F-M system, except for Model 2 where the trend is reversed.
- The effluent faecal coliforms of the PFP is lower than that of the SFP due to the longer retention time implying that the bacteria removal or die off is higher for the PFP.
- The effluent BOD concentration of the PFP is lower, in terms of COD concentration, compared with that of the SFP due to the longer retention time. The trend is similar for the faecal coliforms.
- The SFP shows that most solids settle in the AP, and the AP removes approximately 60–80% of the BOD loading, as discussed in Section 4.3.1.1.
- The above indicate that the process designer should pay attention to the BOD loading, i.e. the influent BOD concentrations and wastewater inflows. This also

includes other parameters, including FC and Nitrates, which should meet the DWS limit guidelines.

Figure 4.19 indicates that the longer the retention time the larger the area required for both the PFP and SFP, for Models 1, 3, and 4, except for Model 2 where the trend is reversed. The pond volume required for both the PFP and SFP, for Models 1, 2, 3, and 4, follows the same trend as indicated above.

4.3.5.3 **PMPs for both the A-F-M and F-M systems for Scenario 1**

Figure 4.21 below shows the comparison of the PMPs for the A-F-M and F-M systems for Scenario 1.

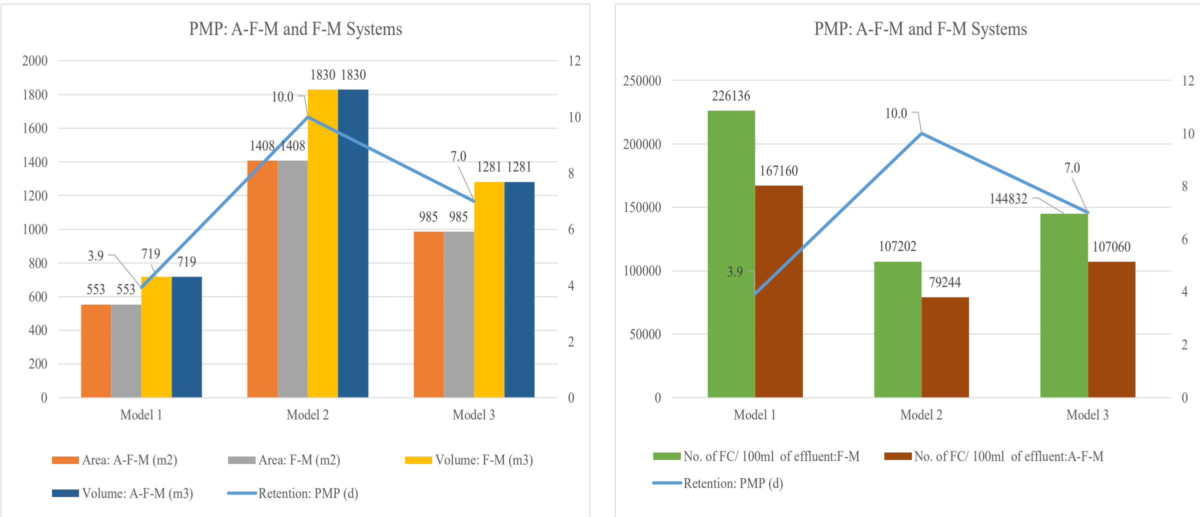
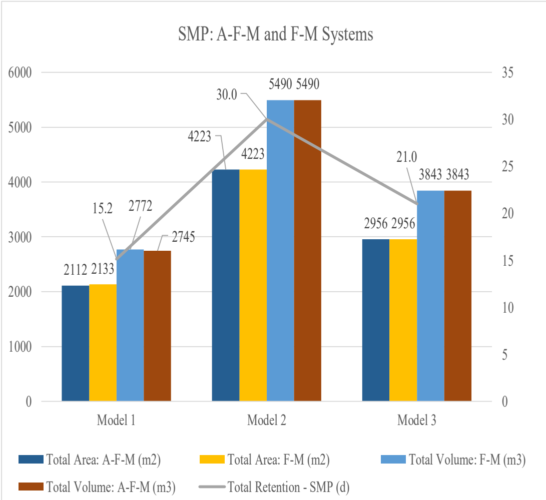


Figure 4.21: The comparison of area, volume, and effluent concentration between the PMPs of the A-F-M and F-M systems for Scenario 1.

The findings of the PMPs for both the A-F-M and F-M systems, as indicated in Figure 4.21, are:

- The PMPs of both the A-F-M and F-M systems require the same or similar pond footprints (pond volume and area) for all three models.
- The inclusion of the AP in the F-M system improves the effluent FC by approximately 26%.
- The preferred model for the PMP is Model 1, which should be used for Scenario 2 as it requires a small area and volume. Thus, it is cost effective.

4.3.5.4 **SMPs for both the A-F-M and F-M systems for Scenario 1**



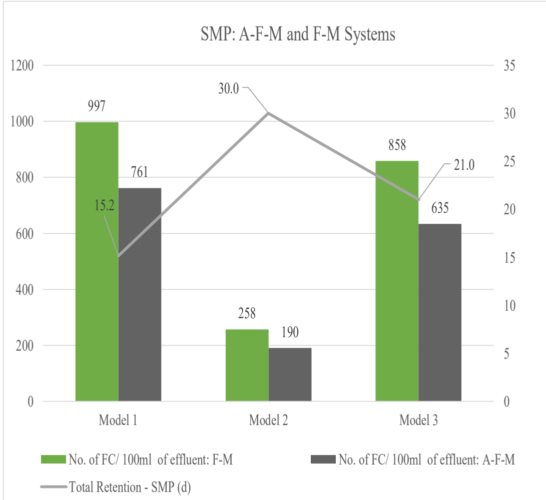


Figure 4.22: The designed area, volume, retention time, and effluent FC of the SMP for both the A-F-M and F-M systems for Scenario 1.

The findings on the pond sizing of the SMPs for Scenario 1 are based on the information presented in Figure 4.22, and they are highlighted below:

- The total volume and area of the SMP (Model 1), for the A-F-M system, are smaller due to total retention time of 15 days compared with 15.2 days of the SMP, for the F-M system.
- Models 2 and 3, for both the A-F-M and F-M systems, require the same total volume, area, and retention time for the SMP.
- The reduction in the retention time of the SMP will reduce the required pond area and volume, and this will also have an impact on the performance of the SMP, particularly in terms of the faecal coliforms.
- Model 1 of the SMP, for both the A-F-M and F-M systems, is found to have the least volume and area. Hence, the construction costs will also be lower than the other two models.
- When the AP is added to the WSP as an upgrade, less pond area will be required for the MP. This will also improve the effluent water quality (faecal coliforms and BOD) of the WSP system.

4.3.5.5 MPs for both the A-F-M and F-M systems for Scenario 1

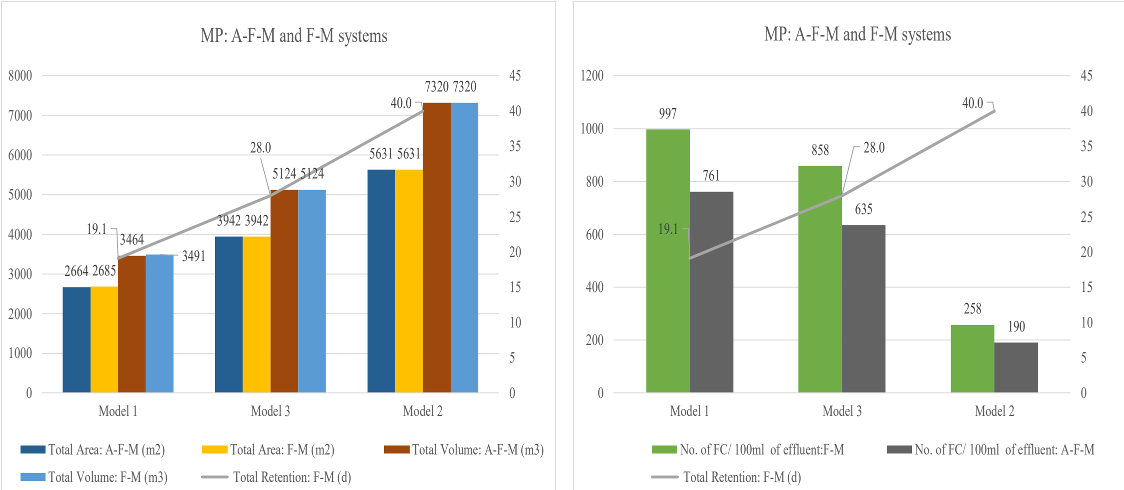


Figure 4.23: The required pond area, volume, retention time and effluent FC of the MP for the F-M and A-F-M systems for Scenario 1.

The following are comments and comparisons, between the MP for F-M and A-F-M systems for Scenario 1, which refer to Figure 4.23 above:

- Table 4.9, Table 4.12, and Figure 4.23 indicate that Model 1 yields a smaller area compared with Model 2 and Model 3, for both the A-F-M and F-M systems.
- The MP of both A-F-M and F-M systems requires a similar pond footprint (pond volume and area) for Models 2 and 3. Although there is approximately 0.8% increase in pond area and volume of the MP (Model 1) for the F-M system.
- It is evident that the inclusion of the AP improves the effluent quality, in terms of FC, as shown in Figure 4.23.
- There are not significant savings in terms of construction costs of the MP for the F-M system and A-F-M system.
- The reduction in the retention time of the MP will reduce the required area and volume, which has an impact in the performance of the MP in terms of the removal of faecal coliforms.
- When the AP is added to the WSP as an upgrade, the required area of the MP will be less, thus improving the effluent quality of the pond system in terms of faecal coliforms and BOD.

4.4 SCENARIO 2 STAGE

The information in Table 4.14 is used as input to perform the process design of the WSP at TARDI, for Scenario 2. The following information is used as the input to model the WSP units required for TARDI:

- Influent COD concentration: the values are obtained from Table 3.2 in Section 3.4.
- Effluent COD concentration: the values are obtained from Table 3.2 and the effluent limit guidelines of the DWS as shown in Table 3.3 in Section 3.4.
- Influent faecal coliforms: the values are obtained from Table 3.2 in Section 3.4.
- Effluent faecal coliforms: the values are obtained from Table 3.2 and the effluent limit guidelines of the DWS as shown in Table 3.2 in Section 3.4.
- Minimum mean temperature in winter: it is obtained from Section 3.4.9.
- Maximum mean temperature in summer: it is obtained from Section 3.4.9.
- Average Daily Flow: the value is obtained from Sections 3.3.5 and 3.4.10.
- Top surface area and depth of ponds: the values are obtained from Section 3.3.4.

Table 4.14: The design parameters used for the WSP at TARDI for Scenario 2

Parameter	**Unit**	**Quantity**
Temperature, minimum	ºC	12
Temperature, maximum	ºC	31
Effluent BOD concentration	mg/l	37.5
Average Dry Weather Flow (Q_{ADWF})	kl/day l/s	183 2.1
Untreated wastewater		
Influent COD concentration	mg/l	625
Influent BOD concentration	mg/l	312.5*
Effluent BOD concentration	mg/l	<75
Influent faecal coliforms	FC/100 ml	4 x 10^7

*COD/BOD ratio is 2:1.

Only the FC reduction and COD loading are considered for the process design of the WSP for Scenario 2.

4.4.1 <u>Pond arrangement and geometry</u>

The TARDI WSP has ponds in series and comprises a septic tank, PFP, PMP, and five SMPs. The existing dimensions, including the depths of the existing WSP units, are measured on site. Table 4.15 to Table 4.18 indicate the geometry of the existing ponds and minimum design requirements.

4.4.2 Determination and verification of the parameter concentrations used for the WSP design

The Microsoft Excel Software is used to determine the minimum, average, median, and maximum concentrations of the influent and effluent measured wastewater parameters that are calculated as shown in Table 3.3 and Table 3.4.

- Influent COD concentration:
    - The measured COD concentrations of 2 438 and 906 mg COD/l, as indicated in Table 3.2 and Table 3.3, are found to be several times the maximum COD concentration of 800 mg COD/l of typical domestic wastewater. Therefore, among other things, it is suspected that there was an analytical error, wrong sampling method or points and or the time at which the samples were taken, sampling after the honey sucker has discharged the sewage at SWP, which could affect the test results obtained on 22/01/2019 and 24/02/2021.
    - The measured COD concentrations of 26.2, 27.2, 32, 128, and 140 mg COD/l, as indicated in Table 3.2 and Table 3.3, are found to be several folds less than the minimum COD concentration of 450 mg COD/l of typical domestic wastewater. Therefore, it is suspected that there was an analytical error and other factors as listed above in the test results.
    - The above show that a single test result is insufficient for design purposes of the WSP as it can lead to the WSP being either overdesigned or undersigned.
    - The calculated average COD concentration is found to be 528.3 mg COD/l, which is acceptable being within the range of 450–800 mg COD/l of typical domestic wastewater.
    - The calculated average COD concentration of typical domestic wastewater is 625 mg COD/l, which will be used for sizing the TARDI WSP given that it is higher than the average of the measured COD concentrations of 528.3 mg COD/l and will yield a more conservative pond design.

- Effluent COD Concentration:

    - The effluent COD concentration is less than the limit guidelines of 75 mg COD/l and 100 mg COD/l of the DWS and WHO, respectively.
    - The effluent COD concentration limit guideline of 75 mg COD/l of the DWS will be used for sizing the TARDI WSP.

- Influent faecal coliforms:

    - The average of the measured influent faecal coliforms (5 780 No./100 ml), indicated in Table 3.2 and Table 3.3, is found to be several folds less than the minimum faecal coliforms of 10^6 No./100 ml of typical domestic wastewater. Therefore, it is suspected that there was an error in the test results, for instance, a dilution with clean water or stormwater.
    - The faecal coliforms of 4 x 10^7 FC/100 ml will be used for the WSP sizing, as advised by Nozaic *et al.* (2009), given that the water test results are not realistic.

- Effluent faecal coliforms:

    - The measured effluent faecal coliforms are discussed in Section 3.4.
    - The effluent FC limit guideline of 1000 FC /100ml mg COD/l of the DWS will be used for sizing the TARDI WSP.

4.4.3 Septic tank for Scenario 2

The design capacity of the septic tank is calculated using Equation 2.6 and the design parameters, which are compared with the existing geometry of the septic tank, are tabulated in Table 4.15. The design parameters used to determine the geometry of the septic tank are discussed in Section 4.3.2. Figure 4.24 shows the input and output data for sizing the septic tank.

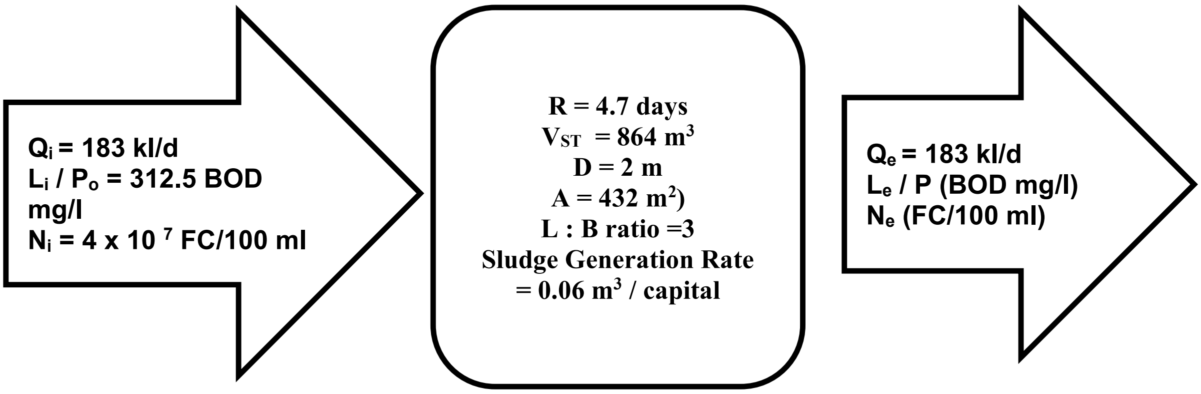


Figure 4.24: The schematic layout of the septic tank for Scenario 2.

Table 4.15: The comparison of sizes and geometries between the existing and newly designed septic tanks at TARDI

Parameter	Unit	Data of the existing septic tank	Minimum design requirements	Reference
Breadth (b_p)	m	3.0	12.0	
Length (l_p)	m	6.0	36.0	
Pond length to breadth ratio	m/m	2	3	l : b = 3: 1 to 4 : 1
Depth	m	1.94	2*	
Area (A_p)	m^2	18	432	
Volume (V_{ST})	m^3	34.9	864	
Water Flow, (Q)	m^3/s	NM	183	NM – Not Measured
Retention time (R)	day	0.2	4.7	
Outlet / inlet pipe.	mm	160	75	

The following are the findings of the mechanism of the septic tank:

- The length to breadth ratio of the existing septic tank is less that 3:1, as recommended by Nozaic, et.al. (2009).
- The existing septic tank is found to be suitable for a population of approximately 87 people and will be overloaded when the contribution population reaches 2 153. The population is determined by using the volume of the existing septic tank and Equation 2.6.
- The above calculation for the septic tank is based on a 10-year interval of desludging (refer to Table 4.3).
- The retention time is calculated to be 0.2 day when the existing dimensions of the septic tank are used. It is found that the retention time of the existing tank does not meet the minimum retention time as indicated in Table 2.17 in Section 2.13. Therefore, solids will not have enough time to settle.

4.4.4 **PFP for the FP without AP for the F-M system for Scenario 2**

The model used to size the PFP is based on Marais design sequence (model 4). The designs of Scenario 2 are done based on the existing pond configuration (F-M system), WSP without APs. This implies that there is no change in the BOD concentration, in the inlet of the septic tank, given that the existing septic tank is overloaded. Hence, the sizing of the PFP is done as there is no AP.

The influent BOD concentration and depth of the designed PFP are assumed to be 312.5 mg COD/l and 2 m, respectively. The pond depth is kept at 2 m for simplicity of comparison of Scenarios 1 and 2. Also, the Marais equation is used for the design of the PFP, as recommended in Section 4.3.3.2. Figure 4.25 presents the influent and effluent parameters at the PFP, including the design parameters required when sizing the PFP.

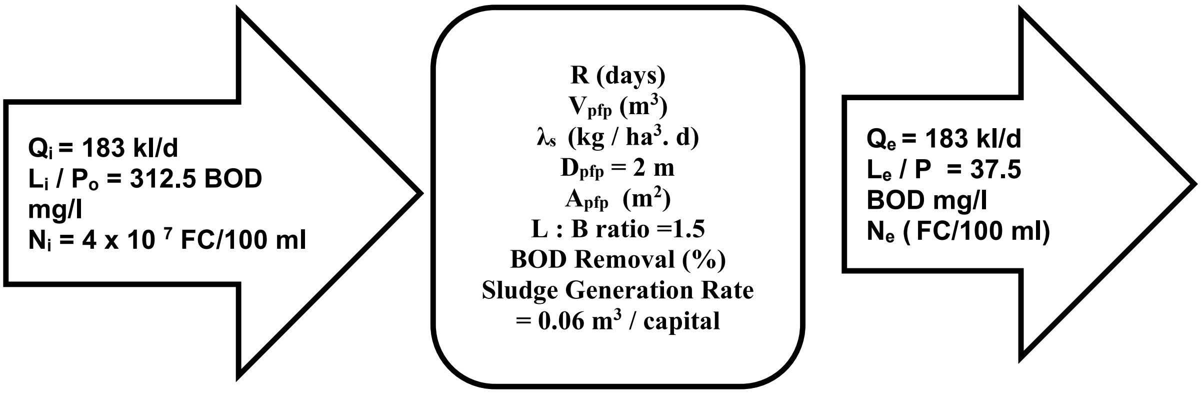


Figure 4.25: The schematic layout of the PFP for the F-M System for Scenario 2.

The following are the comparisons between the design requirements and existing pond parameters, including the other associated pond sizing or design aspects:

- The existing FP volume of 3 204 m^3 and inflow of 183 kl/d are used to compute the retention time of 17.5 days as shown in Table 4.16 as Model 4B.
- The calculated retention time of 17.5 days for the existing pond is found to be shorter than the retention time of 66.7 days for the new pond designed to receive a wastewater inflow of 183 kl/d, shown as Model 4C in Table 4.16. Hence, there will be overloading, bacteria washout, and effluent BOD non-compliance with the DWS guidelines.
- The performance efficiency of the existing PFP is discussed in the Section below.
- The literature review by Ashworth and Skinner (2011) indicated that the BOD removal of the MP is approximately 20% as indicated in Section 2.6.4.2 and Table 2.17. Thus, the FP can be designed or sized to discharge an effluent BOD concentration of not more than 45 mg BOD/l to meet the DWS limit guideline of 37.5 mg BOD/l (75 mg COD/l).
- The effluent number of FC per 100ml of 73.2 x 10^4 is found to be non-compliant with the DWS limit guideline. Therefore, there will be a requirement for further treatment to remove the FC.

Table 4.16: The sizes of the existing and newly designed PFP for the F-M system at TARDI for Scenario 2

Parameter	Unit	Existing PFP Model 4A	Model 4B	Design Requirements Model 4C	Reference
Breadth (b_p)	m	19.9	19.9	73.6	Figure 3.5
Length (l_p)	m	83	83	95.7	Figure 3.5
Pond length to breadth ratio	m/m	4.17	4.17	1.5	l : b = 2: 3 to 2 : 1
Depth	m	1.94	1.94	2.0	Measured on site
Area (A_p)	m^2	1 652	1 652	6 100	
Volume (V_p)	m^3	3 204	3 204	12 200	Figure 3.5
Influent flow (Q_i)	kl/day	38	183	183	$Q_i = V_p / R$
Retention time (R)	day	82.9	17.5	66.7	Equation 2.27
Influent COD concentration	mg/l	528.3	625	625	Calculated in Section 4.4.2
Influent BOD concentration	mg/l	264.2	312.5	312.5	
Effluent COD concentration	mg/l	52.1	213.6	75	Table 3.4 , Table 2.14
Effluent BOD concentration (L_e)	mg/l	26.1*	106.8* ++	37.5*	
BOD removal	%	96.1	84.0	95.2	Table 2.13
BOD removal	%	90.1	65.8	88	Equation 3.2
Bacterial Reduction					
Influent faecal coliforms	FC/100 ml	4×10^{7}	4×10^{7}	4×10^{7}	
Effluent No. of FC/ 100ml	FC/100 ml	73.23×10^{4}	324.7×10^{4}	907.0×10^{3}	
Faecal coliforms removal	%	98.2	91.9	97.7	Equation 3.2

*Assumption of no BOD removal in the subsequent MPs
++ Effluent BOD concentration does not meet DWS guideline

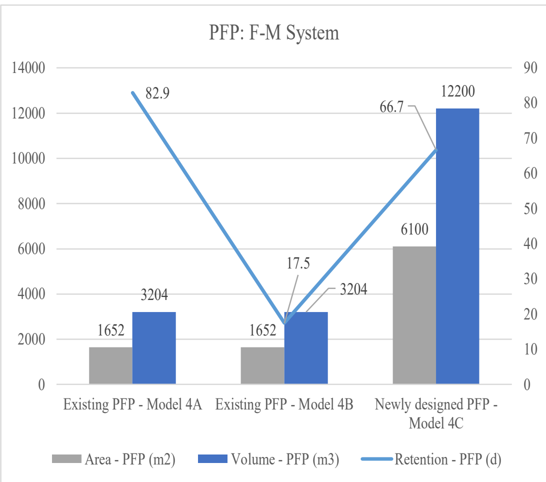
4.4.4.1 **Evaluation of the performance efficiency of the existing PFP**

Table 2.13 in Section 2.9.2 and Equation 3.2 in Section 3.4.5 are both used to evaluate the existing PFP. The determination of the performance is done to assess the mechanism of operation of the existing PFP, i.e. whether the PFP yields the removal percentages (%), for different parameters, as indicated in the literature.

The following are the findings on the performance of the existing pond:

- The BOD removals are found to be 84 % and 90.1 % when Table 2.13 and Equation 3.2 are used, respectively, as indicated for Model 4A in Table 4.16. The removals are for the case where the average influent and effluent BOD concentrations of 264.2 and 26.1 mg BOD/l, respectively, are used.
- The recommended equation to be used when determining the BOD or COD removal of the existing pond is Equation 3.2, hence, the retention time can be affected by the sludge blanket (accumulation of the sludge at the bottom of the pond). The sludge blanket reduces the pond volume.
- The percentage of the BOD removal is found to be within the acceptable limits when compared with the range of 75–85 % BOD removal stated in the literature in Section 2.7, Table 2.4 and Sections 2.13, Table 2.17.
- The maximum BOD concentrations of 84.2 and 63.1 mg BOD/ l in the PFP, for it to remain aerobic, are determined from Equation 2.18 and Equation 2.19, respectively, with the existing pond depth of 1.94 m (6.47 feet).
    - Marais suggested the pond to be designed at 75% to prevent failure, as shown in Figure 2.14 in Section 2.9.2.1d)2.9.2. Therefore, the pond will have the maximum design BOD concentration of 63.1 mg BOD/l in aerobic conditions.
    - The above also implies that the existing PFP is not aerobic, hence, both influent BOD concentrations of 312.5 and 264.2 mg BOD/l are found to be above 84.2 mg BOD/ l.
- The pond length ratios used for pond sizing, for Scenarios 1 and 2, are the same, and they differ from the pond length ratio of the existing PFP of 4.17. They are also greater than the recommended maximum ratio of 3. Therefore, the pond will tend to operate in plug flow conditions, as indicated in Section 2.6.2.1.
- The water test results of both the average influent and effluent BOD concentrations of 264.2 and 26.1 mg BOD/l are used in Equation 2.27 to calculate the retention time of 82.9 days for the PFP, shown as Model 4A in Table 4.16. This retention time and existing PFP volume of 3 204 m^3 yield an inflow of 38 kl/d. Therefore, based on the above BOD concentration results, the pond will be overloaded once the BOD loading exceeds 20.1 kg BOD/d.

- The BOD loading of 20.1 kg BOD/d is used to determine the average influent BOD concentration of 109.7 mg/l when the inflow is 183 kl/d. It is noted that the BOD concentration is below the minimum value of 250 mg/l indicated in Table 2.2 in Section 2.4.
- The BOD loading can be used to determine the maximum inflow to the PFP (capacity of the pond). This inflow is found to be 76 kl/d when the wastewater has average influent concentration of 264.2 mg BOD/ l.
- The retention time (82.9 d) of Model 4A and the calculated retention time (17.5 d) of Model 4B are different, as found in Figure 4.26 and Figure 4.32, when using the existing PFP volume of 3 204 m^3 and inflow of 183 kl/d. The measured BOD concentration indicates that the current inflow to the existing PFP is less than 183 kl/d for Model 4A, as indicated in Table 4.16 and Figure 4.26. The PFP discharges an average effluent BOD concentration of 26.1 mg BOD/l which is less than the DWS limit guideline of 37.5 mg BOD /l.
- The exact pond inflows when samples are taken should be known, given all the above findings.



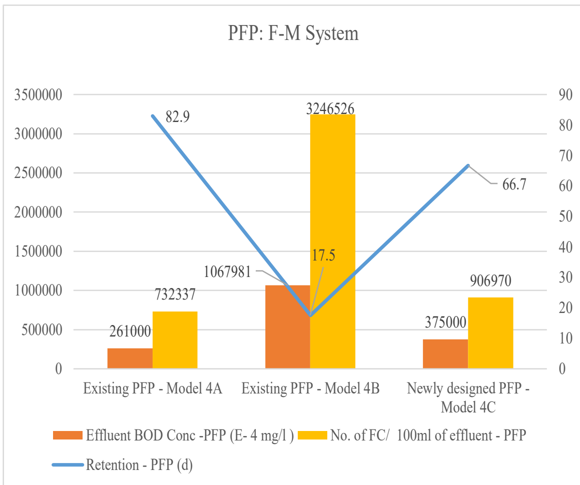


Figure 4.26: The pond geometry (area and volume) and effluent concentrations of the existing and newly designed PFPs for the F-M system for Scenario 2.

4.4.5 MP without AP for the F-M system for Scenario 2

The design of the MP is based on the model by Marais (1974), von Sperling (2007b), and Nozaic and Freese (2009), as discussed in Section 4.3.4.2 (Model 1). Table 4.17 below shows the comparison between the pond sizing of the existing and newly designed MPs. Figure 4.27 shows the influent and effluent concentrations, and the design parameters of the PMP.

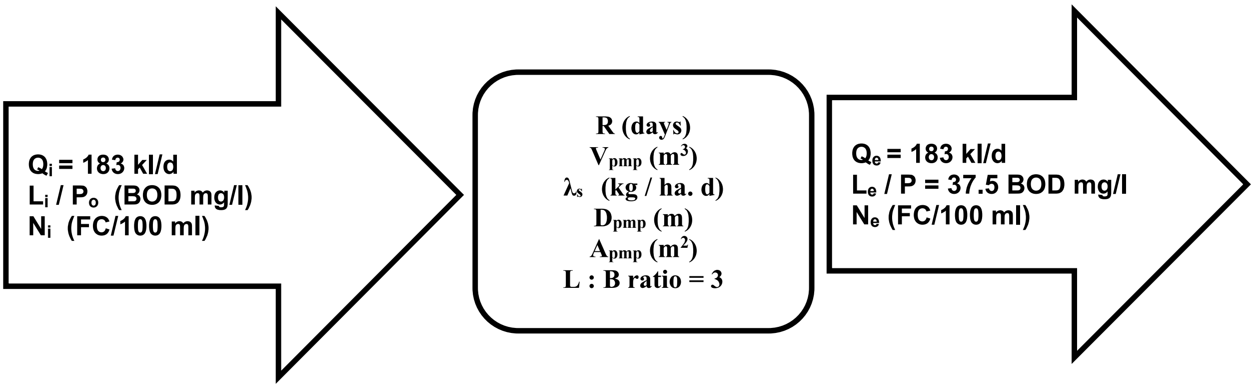


Figure 4.27: The schematic layout of the PMP for the F-M system for Scenario 2.

Table 4.17: The summary of the existing and newly designed PMPs for the F-M system at TARDI for Scenario 2.

Parameter	Unit	Parameters of the existing PMP	Minimum Design Requirements	Reference
Breadth (b_p)	m	14.4	13.6	Figure 3.5
Length (l_p)	m	43.9	40.7	Figure 3.5
Pond length to breadth ratio	m/m	3	3	l : b = 3 : 1
Depth	m	1.0	1.3	Figure 3.5
Area (A_p)	m^2	632	552.8	
Volume (V_p)	m^3	632	718.7	Figure 3.5
Retention time (R)	day	3.5	3.9	$Q_i = V_p /R$
Surface loading (λ_m)	kg/ ha/ day	124.1		Equation 2.26
Influent BOD concentration	mg/l	106.8	37.5	Assumed to be the same as the effluent from PFP
Influent No. of FC/ 100ml		324.7×10^4	9.07×10^5	
Effluent No. of FC/ 100ml		1.00×10^6	2.56×10^5	
Faecal coliforms removal	%	69.1	71.7	Equation 3.2

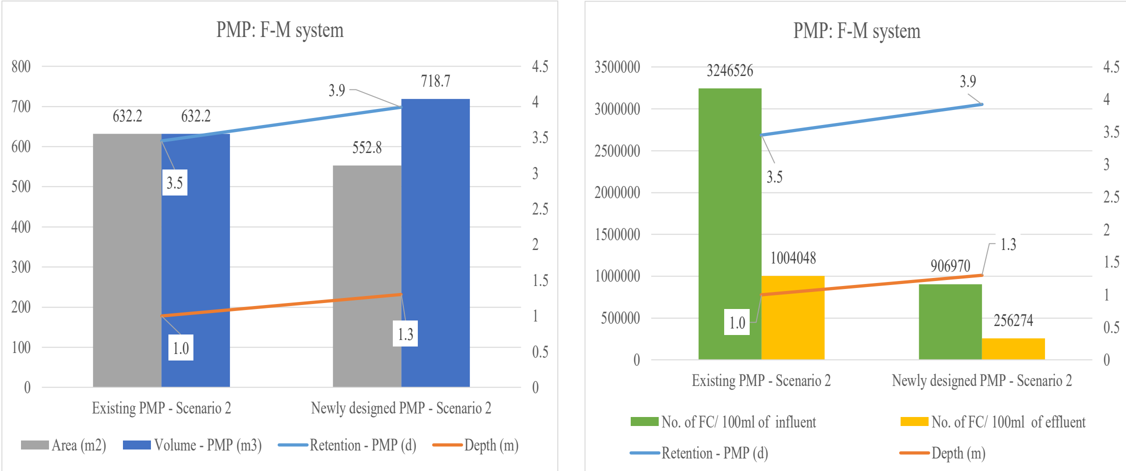


Figure 4.28: The pond geometry (depth, area and volume) with the retention time and effluent concentrations of the existing and newly designed PMP for Scenario 2.

Figure 4.28 and Table 4.17 show that the existing PMP has shorter retention time and volume, and also poor effluent quality (in terms of FC) when compared with the newly sized PMP.

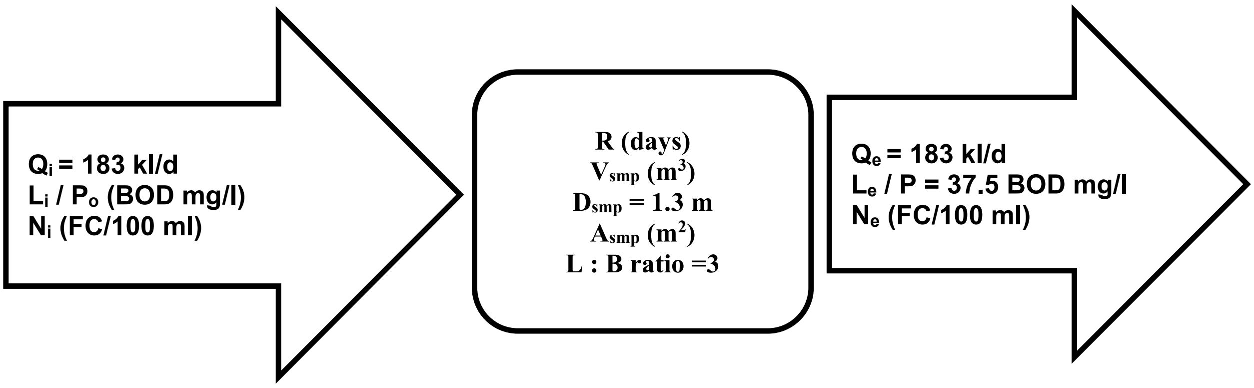
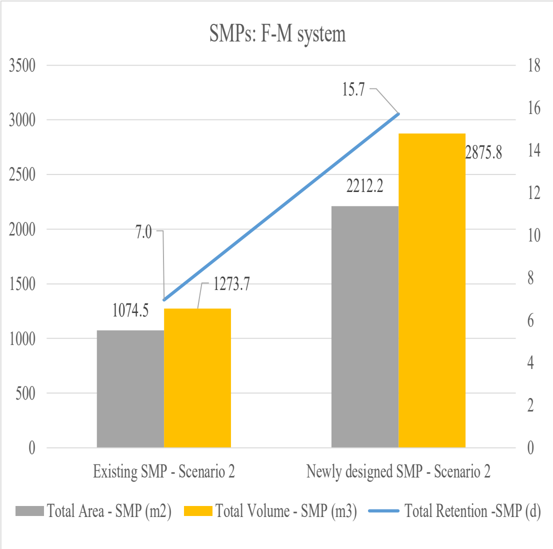


Figure 4.29: The schematic layout of the SMP for the F-M System for Scenario 2.

Figure 4.30 below, Table 4.18, and Table 4.19 indicate that the existing SMP requires smaller land (1 075 m^2) compared with the newly designed SMP (2 212 m^2). The benefit of this reduction in footprint should be compared with the effluent FC concentration which is higher than that of the newly designed SMP.



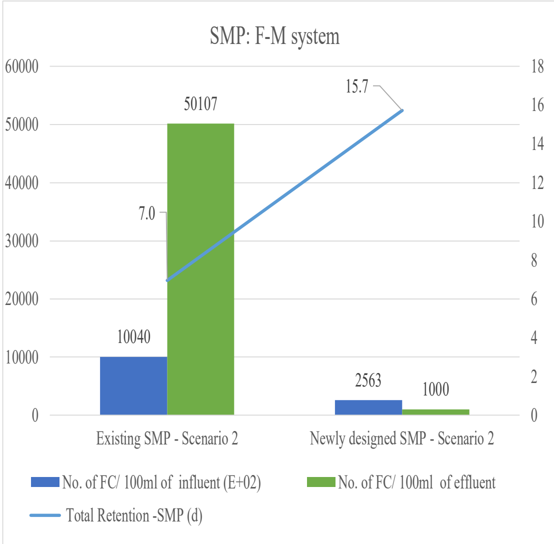
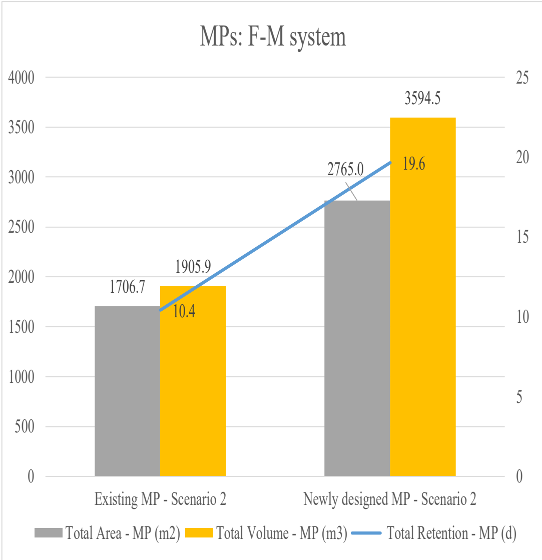


Figure 4.30: The pond geometry (total area and volume), total retention time, and influent and effluent FC of the existing and newly designed SMPs for the F-M system for Scenario 2.

The following are the comments and findings regarding the MP of the existing and newly designed MP (F-M system):

- The area, volume, and retention time of the existing and newly designed MPs are different, as indicated in Table 4.17 to Table 4.19 and Figure 4.31. The designed MPs have bigger area, volume, and retention time, compared with the existing MPs.
- The effluent faecal coliforms count of 5.01 x 10^4 FC No./100 ml for the existing MP does not meet the DWS effluent requirement of 1000 FC No./100 ml as shown in Table 4.18 and Figure 4.31.
- The reduction in the retention time of the MP undoubtedly also reduces the required area and volume, and this has an impact on the performance of the MP in terms of the treated faecal coliforms. This is also highlighted for Scenario 1 in Section 4.3.4.
- The newly design MP will meet the DWS effluent requirements, as shown in Figure 4.31. The required total area, volume, and retention time of the newly designed MP are 2 765 m^2, 3 595 m^3, and 19.6 days, respectively, as shown in Table 4.19.



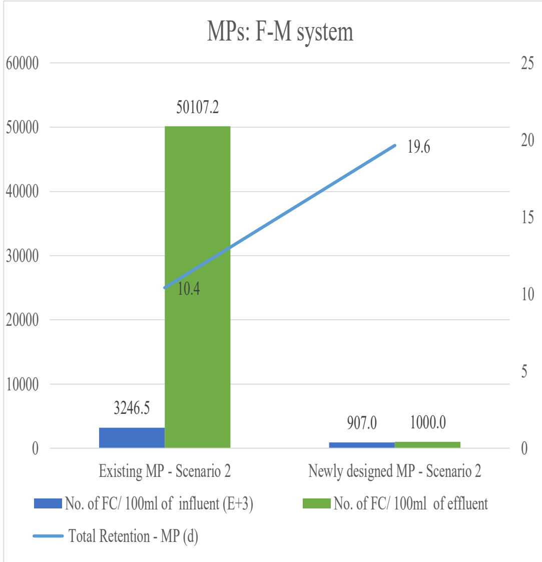


Figure 4.31: The total area, volume, total retention time, and the influent and effluent FC of the existing and newly designed MP for the F-M system for Scenario 2.

4.4.5.1 **Evaluation of the performance and efficiency of the existing MP**

The efficiency and performance evaluations of the existing MP (F-M system) are not done because of errors in the measured faecal coliforms, as presented in Table 3.2 to Table 3.4, which would otherwise be used to assess the mechanism of operation of the existing MP. Thus, the recommended removal percentage (%) in terms of faecal coliforms are not calculated and compared with the literature.

Table 4.18: The summary of the existing and newly designed SMPs for the F-M system at TARDI for Scenario 2

Parameter	Unit	Dimension of the existing SMP and Minimum Design Requirements									
Secondary MP (SMP)		SMP 1		SMP 2		SMP 3		SMP 4		SMP 5	
		Existing	Design	Existing	Design	Existing	Design	Existing	Design	Existing	Design
Breadth (b_p)	m	14.0	12.1	9.9	12.1	8.0	12.1	8.7	12.1	11.5	12.1
Length (l_p)	m	25.5	36.4	19.9	36.4	8.3	36.4	9.9	36.4	32.0	36.4
Pond length to breadth ratio	m/m	1.8	3.0	2.0	3.0	1.0	3.0	1.1	3.0	2.8	3.0
Depth	m	1.3	1.3	1.4	1.3	1.2	1.3	1.0	1.3	1.0	1.3
Area (A_p)	m^2	357	442	197	442	66.4	442	86	442	368	442
Volume (V_p)	m^3	464	575	276	575	79.7	575	86	575	368	575
Retention time (R)	day	2.54	3.14	1.51	3.14	0.44	3.14	0.47	3.14	2.0	3.14
Influent No. of FC/ 100ml		1.00×10^6	2.56×10^5	3.80×10^5	8.45×10^4	1.93×10^5	2.79×10^4	1.50×10^5	9.19×10^3	1.15×10^5	3.03×10^3
Effluent No. of FC/ 100ml		3.80×10^5	8.45×10^4	1.93×10^5	2.79×10^4	1.50×10^5	9.19×10^3	1.15×10^5	3.03×10^3	5.01×10^4	1000
Faecal coliforms removal	%	62.1	67.0	49.4	67.0	22.0	67.0	23.3	67.0	56.5	67.0

Table 4.19: The summary of the MP without AP for the F-M system for Scenario 2

Parameter		**Unit**	**Existing**	**Design**
PMP	Retention time (R)	day	3.5	3.9
	Volume (V_{pmp})	m^3	632	719
	Area (A_{pmp})	m^2	632	553
SMP	Retention time (R)	day	7.0	15.7
	Total Volume (V_{smp})	m^3	1 274	2 876
	Total Area (A_{smp})	m^2	1 075	2 212
TOTAL	Retention time (R)	day	10.4	19.6
	Total Volume (V_{mp})	m^3	1 906	3 595
	Total Area (A_{mp})	m^2	1 707	2 765

4.4.6 **Summary of pond sizing for the F-M systems for Scenario 2**

Figure 4.32 presents the pond design summary for Scenario 2 of the F-M system, which is briefly discussed below:

- The other measured wastewater parameters were found to be outliers when compared with the literature data, which could be due to the unprofessionalism of the sampling personnel.
- The inaccuracy of the sample data is found to impact the calculated results, and also the performance evaluation thereof. Thus, imperative information viz. the WSP retention time and % removals in terms of FC, COD, and BOD, cannot be determined. Also, the capacity of the WSP cannot be determined and verified.
- The preferred models to be used when sizing and determining the pond performance and efficiency in the Eastern Cape Province are Model 4 (Marais design sequence), for FPs, and Model 1, for MPs.

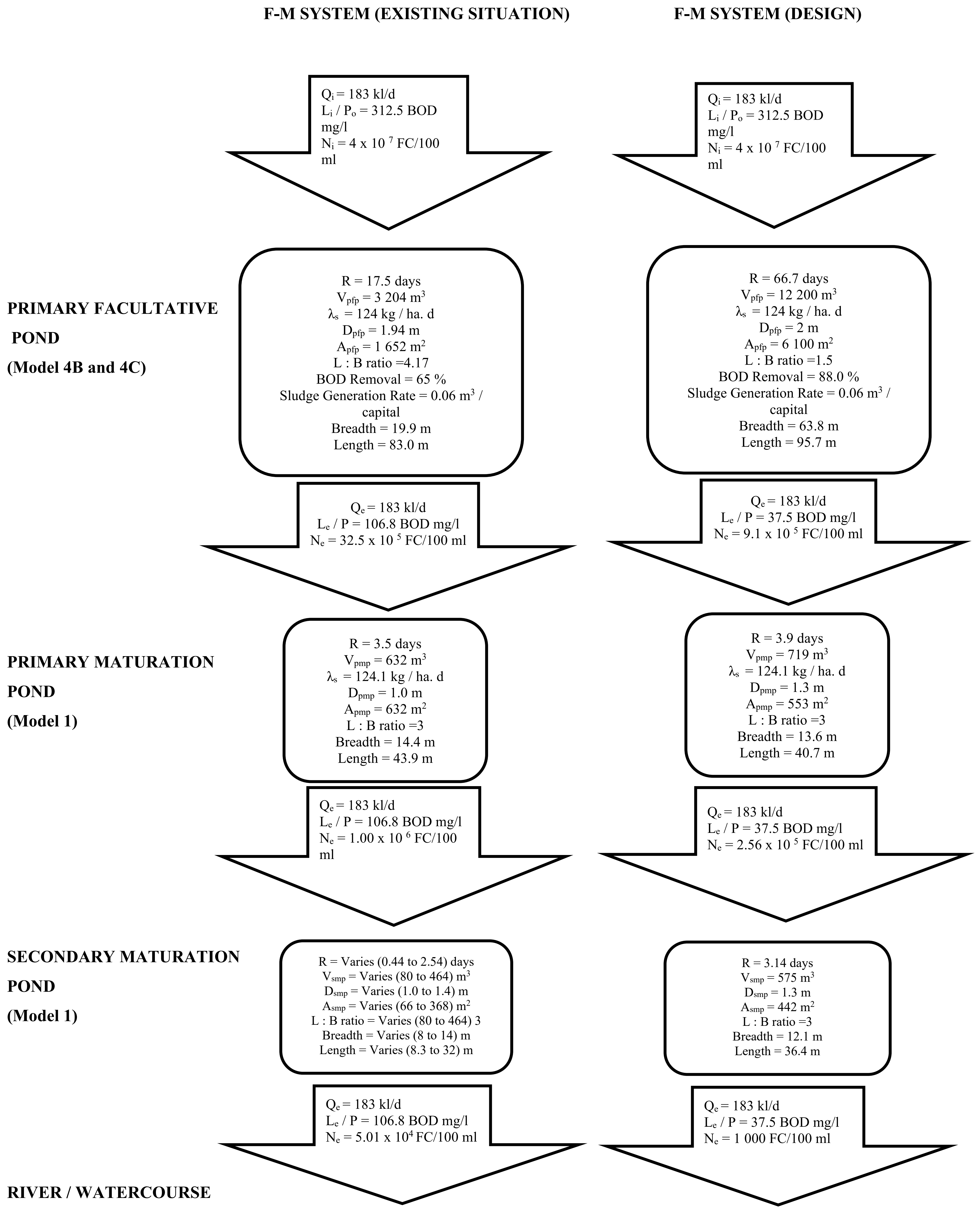


Figure 4.32: The schematic layout indicating the pond sizes of each unit and the difference between the existing F-M and designed F-M systems for Scenario 2.

4.5 COMPARISON OF THE POND SIZING FOR SCENARIO 1 AND SCENARIO 2 STAGES

The components of the WSP compared in Sections 4.5.1 to 4.5.3 are based on the modelling discussed in Sections 4.3 and 4.4. This section primarily compares the general findings and then considers the sizing and effluent comparisons between Scenarios 1 and 2 of each unit of the WSP, namely the AP, FP, and MP. Table 4.20 below indicates the pond parameters, and the influent and effluent concentrations for the best model identified for Scenarios 1 and 2.

Table 4.20: The total pond footprint, capacity, and effluent quality of the models used for Scenarios 1 and 2

Parameter	Unit	Scenario 1 Model	Scenario 2 Model
Inflow and outflow (Q)	kl/day	183	183
Influent BOD concentration	mg/l	350	312.5
Influent faecal coliforms	FC/100 ml	4×10^7	4×10^7
Area (A_p)	m^2	9 617	8 865
Volume (V_p)	m^3	17 355	15 795
Retention time (R)	day	94.8	86.3
Effluent BOD concentration	mg/l	37.5	37.5
Effluent faecal coliforms	FC/100 ml	997	1 000
BOD removal	%	89.3	88.0
Faecal coliforms removal	%	99.998	99.998

The general factors that can impact the modelling and performance of the ponds are:

- The water quality test results of the influent and effluent, including the influent flows, are insufficient to perform the statistical analysis and to analyse the performance of the existing ponds in detail.
- The information, namely the pond geometry (length, width, depth, and side slopes), design flows, and water quality tests data used for modelling do not

compare with those of the existing pond and typical domestic wastewater characteristics.

- The performance efficiency evaluation of the existing WSP is important before considering its upgrade.
- The water samples are to be taken periodically, during both the winter and summer seasons, to ensure that the existing pond geometry, and the water quality and flows meet the design requirements.
- The influent BOD concentrations of 350 and 312.5 mg BOD/l are used for modelling Scenario 1 and 2, respectively.
- The pond volume and area have approximately 9% and 8% variances when the BOD concentration is reduced from 350 to 312.5 mg BOD/l, respectively, as indicated in Table 4.20. The change in pond geometry is only applicable to the PMP.
- The WSP users should ensure that there are flow measuring devices installed at the influent entry and effluent exit points of the WSP. Also, water samples should be taken of the influent and effluent of each WSP unit so that the operator is able to check the performance of the WSPs.
- The recording of the information discussed above will help in the monitoring of the pond operation, particularly the desludging, before the pond is overloaded, and effluent compliance with the DWS guidelines.

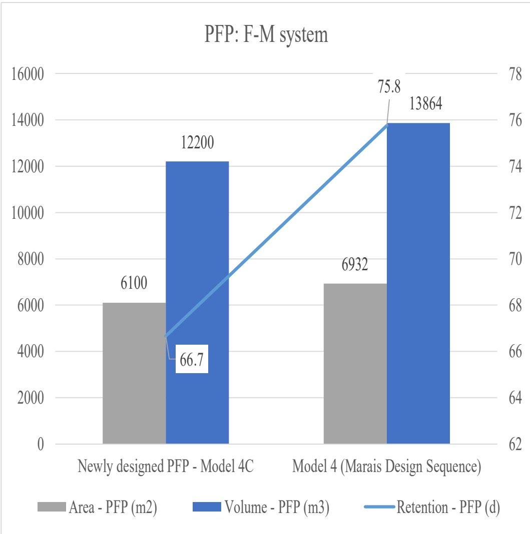
4.5.1 **AP**

There is no comparison to be done between Scenario 1 and Scenario 2 of the AP, hence, the sizing of the AP during the Scenario 2 stage was not performed. Based on the findings of the literature review in Chapter 2, the retention time and AP volume should be between 1.75–3.5 days and 320–640.5 m^3, respectively, to keep the volumetric loading between 100–200 g BOD/m^3.d, as discussed in Section 4.3.1.

Therefore, the sizing of the AP should take into consideration the recommended BOD loading range of 100–200 g BOD/m^3.d and pond volume range of 320–640.5 m^3 to limit odours and reduce construction costs.

4.5.2 FP

In general, the PFP (FP without AP [F-M system]) is found to have a longer retention time compared with the SFP (FP with AP [A-F-M system]), and this also resulted in it requiring a larger surface area and volume, as shown in Table 4.13 and Figure 4.16. The effluent BOD concentration of 37.5 mg BOD/l and pond depth of 2 m, for the PFP of the F-M system, were kept constant during Scenarios 1 and 2 for pond sizing purposes, as shown in Figure 4.33. This was to ensure the effluent compliance with the DWS guidelines. Hence, the performance comparison of the ponds can be done.



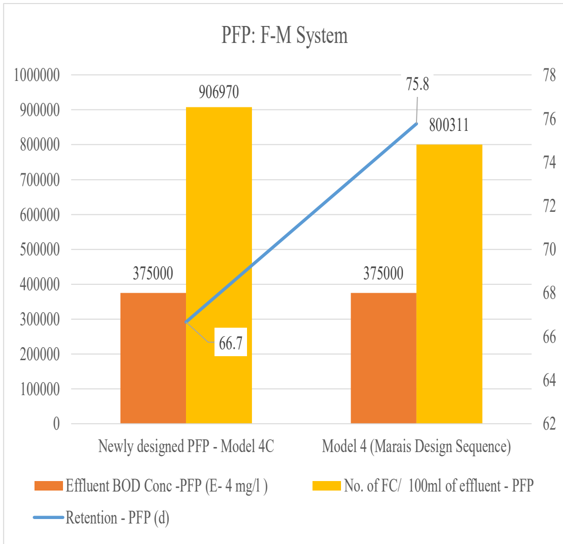


Figure 4.33: The pond geometry (area and volume) and effluent concentrations of the newly designed PFP for Scenario 1 and Scenario 2.

The findings and comparison of the PFP for Scenario 1 and Scenario 2 are as follows:

- The retention time: it is found that the retention time used for Scenario 1, of the PFP in the F-M system, is longer than that obtained for Scenario 2.
- The pond area and volume: they are found to be greater for Scenario 1, thus improve the effluent faecal coliforms, compared with the model (Model 4) used during Scenario 2 stage.
- The present study has proved that the pond parameters and wastewater quality can be used with confidence for Scenario 1, and there will be marginal changes in the pond geometry of the PFP for Scenario 2.

- The assumed depth of 2 m of the PFP for Scenario 1 is found to be approximately equal to the 1.94 m depth measured on site. Although the assumed BOD concentration of 350 mg BOD/l (700 mg COD/l) used during Scenario 1 stage was found to vary between 13.1–1219 mg BOD/l (26.2–2438 mg COD/l) on site as shown in Table 3.3, it indicates the inaccuracy of the site data as compared to literature data used for Scenario 1. Hence, the PFP sizing that is based on the site data will lead to over designed for Scenario 2. In this case, the verification of the data is vital as discussed in Section 4.4.2.

4.5.3 <u>MP</u>

Figure 4.34 to Figure 4.36 below show the sizing of the MPs (PMPs and SMPs), including the retention time and influent and effluent concentrations computed for Scenario 1 and Scenario 2.

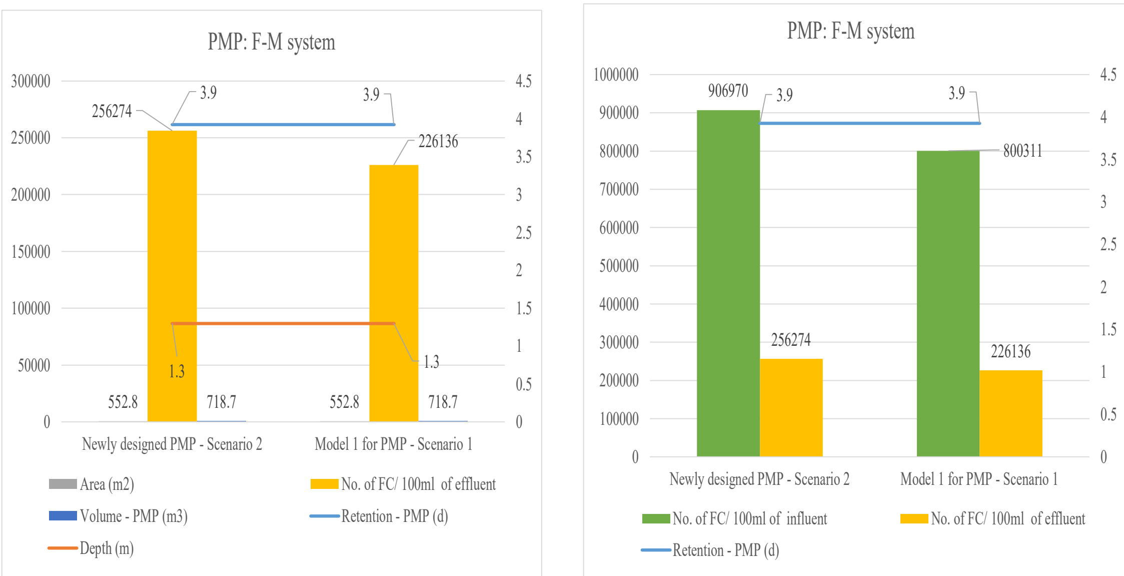
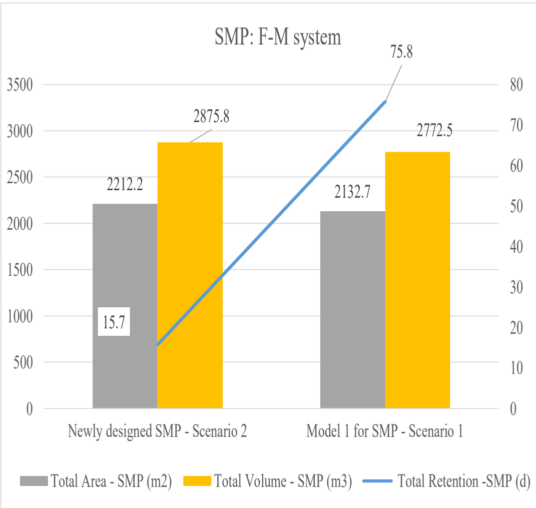


Figure 4.34: The pond geometry (depth, area, and volume) and effluent concentrations of the designed PMP for Scenarios 1 and 2.

Figure 4.34 above shows that there is no change in the retention time, depth, area, and volume of the PMP for Scenarios 1 and 2. Nevertheless, there is a difference in the quality of both the influent and effluent, in terms of the FC, during both scenarios as the retention time, volume, and area of the PFP for Scenario 1 were greater than those of the PFP for Scenario 2. This is due to the change in BOD concentration from 350 mg BOD/ l for Scenario 1 to 312.5 mg BOD/ l for Scenario 2, which is discussed in Sections 4.4.4 and 4.5.2.

The use of the typical influent BOD concentration for Scenario 1, which is considered to be significantly high, results in a larger PFP volume (Scenario 1) as compared with PFP for Scenario 2. This does not compromise the effluent concentration when the pond sizes are used for Scenario 2 (Figure 4.35 below). There will be a benefit of having a better effluent quality, namely the removal of FC by the PFP. This is also indicated in Figure 4.36 below, where the model used to size SMP for Scenario 1 is found to be the best model as it requires a smaller area and volume, and it also has a better effluent quality compared with the model used for sizing the SMP for Scenario 2. It can be concluded that the oversizing of the FP for Scenario 1 will help reduce the effluent FC.



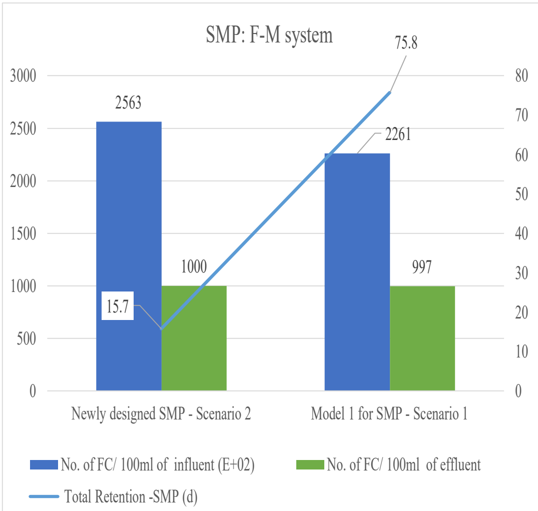
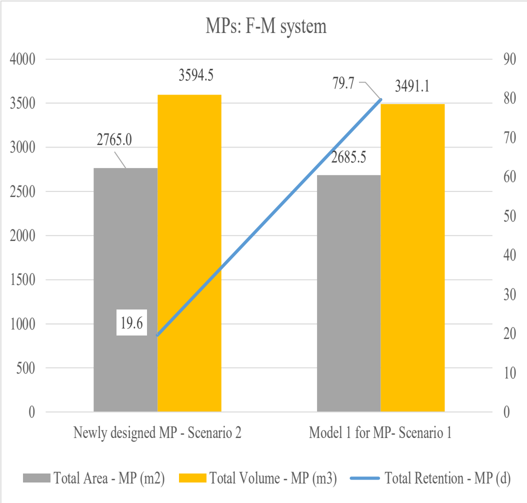


Figure 4.35: The pond geometry (total area and volume), total retention time, and the influent and effluent concentrations of the designed SMP for Scenario 1 and Scenario 2.



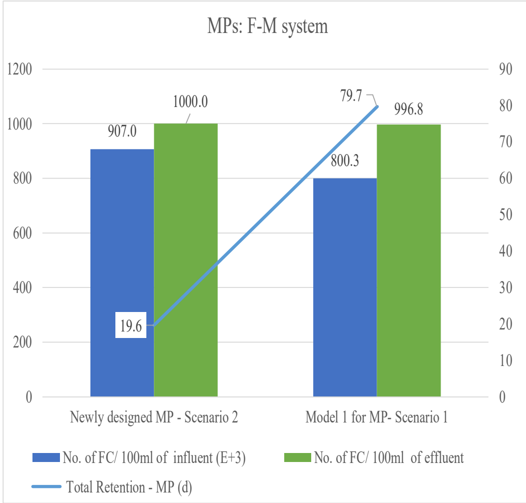


Figure 4.36: The pond geometry (total area and volume), total retention time, and the influent and effluent concentrations of the MP for Scenario 1 and Scenario 2.

4.6 CONCLUSIVE SUMMARY

Given the variations in wastewater quality, the pond performance depends on, among other things, the seasonal temperature. The proper assessment of a WSP performance requires the availability of data, including the flow measurements and pollutant concentrations. Otherwise, the design of the WSP will be invalid. The comparison of pond sizing for Scenarios 1 and 2, discussed in Section 4.5, show that the pond sizing of the FP and MP is affected by the BOD loading and faecal coliforms of the influent to the WSP. Hence the following:

- The recording of data was poor given that most authorities perform these tests to comply with the DWS requirements and maintain their Water Use License (WUL).
- The National Water Act requires the water sampling and testing to be done by SANAS accredited laboratory. This is done to gain the confidence in terms of acceptance of the water test results and to meet the requirements of National Water Act, as discussed in Section 2.13.
- The water quality test results of the influent and effluent were insufficient to perform the statistical analysis for the TARDI WSP.

- The as-built information to determine the geometry of the ponds was unavailable, notably the installed flow meters and pond design capacity, as discussed in Chapter 3.
- The seven initially selected pond sites, namely the 14 South African Infantry (SAI) Battalion, Bedford Hospital, Mthatha Prisons, St Patricks School, Marselle township, FCC, and TARDI, which were identified in the Eastern Cape as the initial study areas, had similar issues as those listed above.
- The performance of the existing WSP is important before considering upgrading it. The work of upgrading the WSP will assist in determining the pond units that are under performing, as indicated in Chapter 2.
- The measured wastewater parameters that were outliers were COD, Ammonia, TKN, N and TSS, as compared with the literature.
- The weather season in which the samples are taken are important to appropriately monitor the performance of the WSP. This includes, among other things, the diurnal flow fluctuations and treated wastewater dilutions.
- The literature review has shown the evidence of the inclusion of the AP in cases where there is an increase in loading, thus saving on land. The inclusion of the AP should be considered when an upgrade of the WSP is required.

CHAPTER 5: CONCLUSIONS

The literature has indicated that different countries, particularly due to different climatic zones, use different methods of pond designs and wastewater parameters. The determination or verification of the degradation constants in Waste Stabilisation Ponds (WSPs) will assist researchers, practitioners, and designers in better understanding wastewater treatment through the WSP process.

This study considered two scenarios, namely Scenario 1 and 2. During the Scenario 1 (feasibility design) stage, the information, namely the water flow rates (to and out of the pond system), water quality results, and pond depths are assumed based on literature. The information used for the Scenario 1 stage are confirmed during the Scenario 2 stage (conceptual stage).

The literature review has shown the evidence of the inclusion of the Anaerobic Pond (AP) in cases where there is an increase in loading, thus saving on land. The inclusion of the AP should be considered when an upgrade of the WSP is required, thus a is found to be the cost-effective model or pond system. This is shown during the Scenario 1 stage when comparing the land requirements for both the A-F-M system and F-M system.

The two types of WSP configuration, namely the A-F-M and F-M systems, are considered during the Scenario 1 stage, and the F-M system is only considered during the Scenario 2 stage. The A-F-M system is the WSP containing an Anaerobic Pond (AP), Facultative Pond (FP), and Maturation Pond (MP), whereas the F-M system is the A-F-M system without AP.

The three models are used for pond sizing of both A-F-M and F-M systems during Scenario 1 stage. The best of the models is selected for further calculations to design the WSP to determine the pond sizes and effluent characteristics of each pond unit during the Scenario 2 stage. The Marais design sequence is found to be the simplest model during Scenario 1 stage and can be used for both sizing the pond units and checking the performance of the WSPs.

The comparison of pond sizing of F-M system for Scenarios 1 and 2, discussed in Section 4.5, show that the pond sizing of the FP and MP is affected by the BOD loading and faecal coliforms of the influent to the WSP.

The following are the findings when the comparing the pond designs during the Scenarios 1 and 2 stages:

- The retention time used during the Scenario 1 stage, of the PFP in the F-M system, is longer than that obtained for Scenario 2.
- The pond volume and area of the PFP have approximately 9% and 8% variances when the BOD concentration is reduced from 350 to 312.5 mg BOD/l, respectively. The change in pond geometry is only applicable to the PMP.
- The present study has proved that the pond parameters and wastewater quality can be used with confidence for Scenario 1, and there will be marginal changes in the pond geometry of the PFP for Scenario 2.
- The assumed depth of the PFP for Scenario 1 is found to be approximately equal to the depth measured on site. Although the assumed BOD concentration of 350 mg BOD/l (700 mg COD/l) used during Scenario 1 stage was found to vary between 13.1–1219 mg BOD/l (26.2–2438 mg COD/l) on site.
- There is no change in the retention time, depth, area, and volume of the PMP for Scenarios 1 and 2
- The use of the typical influent BOD concentration for Scenario 1, which is significantly high, results in a larger PFP volume (Scenario 1) as compared with PFP for Scenario 2. This does not compromise the effluent concentration when the pond sizes are used for Scenario 2.
- There will be a benefit of having a better effluent quality, namely the removal of FC by the PFP.
- It can be concluded that the oversizing of the FP for Scenario 1 will help reduce the effluent FC.

In present study, the modelling of a WSP in the Eastern Cape region of South Africa was considered as a case study. It was noted, during the research study, that the recording of the inflows, outflows, and water quality parameters are vital to monitor the performance of the pond. Also, the water quality tests have to be done, as required by the DWS, for determining the performance of the existing pond and monitoring the sludge accumulation inside the WSP system. Furthermore, it was noted that the maximum temperature should not be used when sizing the FPs, as this can result in a smaller pond size, and subsequently in an effluent quality not complying with the required discharge guidelines, particularly during the winter season. Also, this would influence the performance of the subsequent pond units.

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APPENDICES

APPENDIX A : RESULTS AND LOCATION OF SEVEN WSP IN EASTERN CAPE

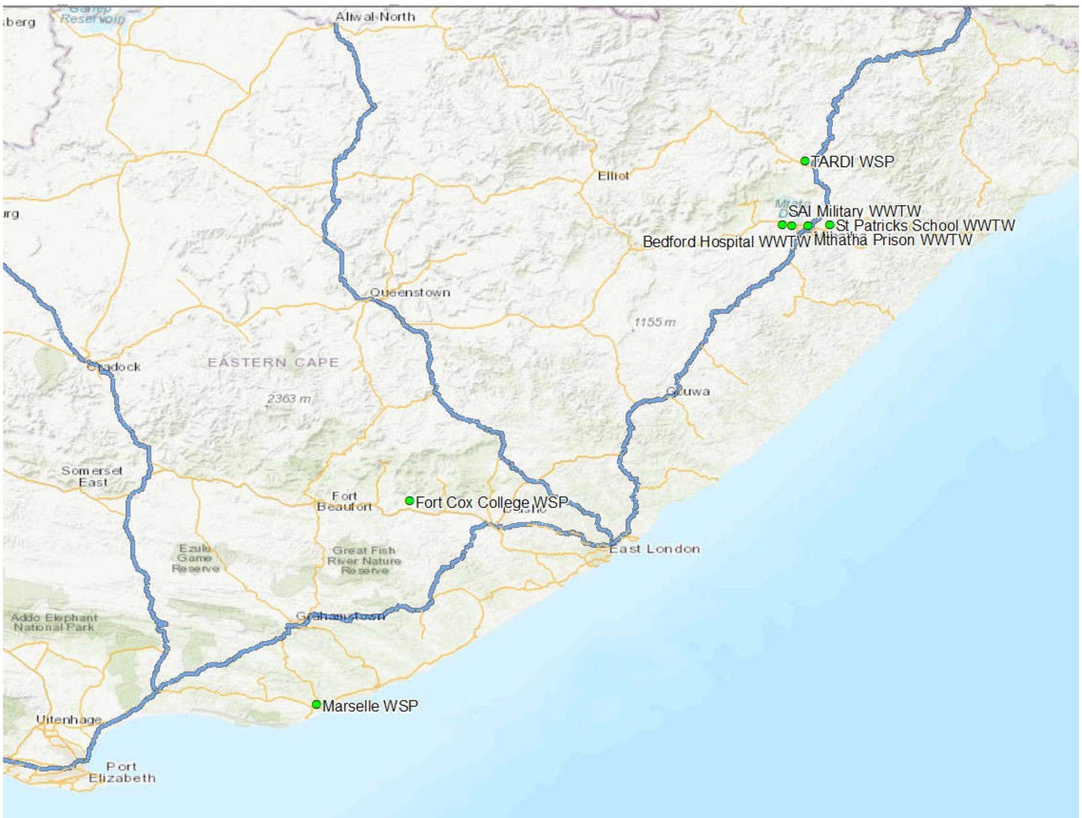

Figure A.1: Map indicating the location of the seven sites.

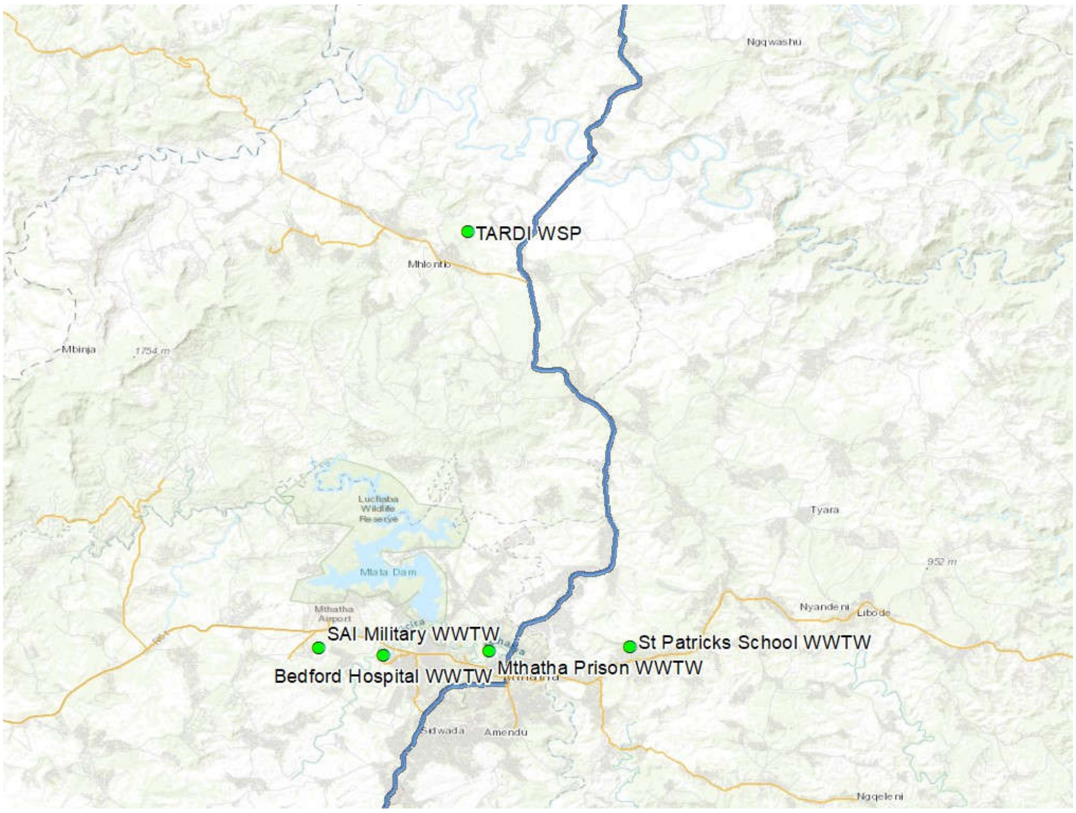

Figure A.2: Four WSP sites located in Mthatha (water test results provided by DWS).

Figure A.3: SAI Military WWTW.

Figure A.4: Bedford Hospital WWTW.

Figure A.5: Mthatha Prisons WWTW layout.

Figure A.6: St Patricks School WWTW layout.

Figure A.7: Marselle township WSP layout.

Figure A.8: Fort Cox College WSP layout.

Figure A.9: TARDI WSP layout.

Table A.1: Comparison between the effluent limit guidelines of treated wastewater and effluent data from four different pond systems near Mthatha

Parameter	DWS Guidelines	WHO Guidelines	14 SAI Military WWTW	Bedford Hospital WWTW	Mthatha Prisons WWTW	ST Patricks School WWTW
Faecal Coliforms	<1 000/100 ml	<1 000/100 ml	6	480	4100	54 000
E. coli			0	220	1600	32 000
pH at 25 ^{0}C	5,5 – 9,5	5,5 – 9,0	7.6	6.8	6.5	6.8
Ammonia (NH_4 – N)	<6 mg/l		0.26	16.9	24.6	16
Nitrate/nitrite (as N)	<15 mg/l		0.21	<0.18	<0.18	<0.18
Suspended Solids	<25 mg/l	<100 mg/l	6	62	108	47
Ortho-phosphates (as P)	<10 mg/l		0.416	1.74	4.20	1.54
Turbidity	NTU		10.7	31.2	89.0	40

Above the DWS and WHO guidelines limits

Above the DWS guidelines limits