HYDRAULIC MODELING AND OPTIMIZATION OF WASTE STABILIZATION POND DESIGN FOR DEVELOPING NATIONS

By

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A Ph.D. THESIS SUBMITTED

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DOCTOR OF PHILOSOPHY (Ph.D.) DEGREE IN CIVIL ENGINEERING OF THE DEPARTMENT OF CIVIL ENGINEERING (WATER RESOURSES AND ENVIRONMENTAL ENGINEERING UNIT), COLLEGE OF SCIENCE AND TECHNOLOGY, COVENANT UNIVERSITY, OTA, NIGERIA

APRIL, 2011

DECLARATION

I, Olukanni David Olatunde, declare that this work was done completely by me under the supervision of Professor K. O. Adekalu (Main Supervisor) of the Department of Agricultural Engineering, Obafemi Awolowo University, Ile-Ife, Osun State, Nigeria and Professor J. J. Ducoste (Co-Supervisor) of the Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, North Carolina, U.S.A. The thesis has not been presented, either wholly or partly for any degree elsewhere. Appropriate acknowledgment has been given where reference has been made to the work of others.

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Olukanni, D. O.

CERTIFICATION

This thesis titled ***Hydraulic Modeling and Optimization of Waste Stabilization Pond Design for Developing Nations*** carried out by Olukanni, David Olatunde under our supervision meets the regulation governing the award of the degree of Doctor of Philosophy (Ph.D) in Civil Engineering of the Covenant University, Ota, Ogun State, Nigeria. I certify that it has not been submitted for the degree of Ph.D or any other degree in this or any other University, and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to my family, Dorcas Eyiyemi (wife), and our son, David Olaoluwa Peace, who since birth has been patient and cooperative with us through the period of our studies both home and abroad. I love you so much and I know that you are the joy to your generation. Remain forever blessed!

Acknowledgements

Praise and glory be to the Almighty God from whom all blessings flow. He has enabled me to come through to the end of another phase of my educational career. I have received numerous help, assistance and guidance from a large number of people over the three years that I have worked on this thesis. Of these, I will only be able to mention a few.

I appreciate my supervisors: Professor Kenneth Adekalu (Obafemi Awolowo University, Ile-Ife, Nigeria) and Professor Joel Ducoste (North Carolina State University, Raleigh, USA) who find time to scrutinize this work and make useful comments and suggestions. Professor Adekalu was instrumental to the course to take at the beginning of this research. He prayed alongside with me for the successful award of the Fulbright scholarship while Professor Ducoste was very supportive on the CFD models of the laboratory-scale waste stabilization ponds that were undertaken. Thank you for the high standard you set for this thesis. Without your support and advice, this thesis could not have been produced the way it is. The benefits of being able to travel to meet and work with leading researchers around the world have been tremendous.

I wish to express my profound gratitude to the Chancellor, Dr. David Oyedepo for His love, support and prayers towards the completion of this program. You have been a blessing to me and I will extend this gesture to generations to come. I salute your love for God and humanity.

I also acknowledge the Vice Chancellor of Covenant University, Prof. (Mrs.) Aize Obayan, Deputy Vice-Chancellor, Prof. Charles Ogbulogo and the Registrar, Dr. Daniel Rotimi for their endless drive and support in making this program a success. Your word of encouragement and love has been tremendous. I cannot forget Pastor Yemi Nathaniel, the former registrar for his fatherly love and prayers. May God bless you all.

I appreciate the effort of the former Dean of College of Science and Technology, Prof. James Katende for supporting my candidacy for the Fulbright scholarship, without which this work would not have been completed. Many thanks to the Dean of Postgraduate School and the current Dean of the College of Science and Technology, Prof. C. O. Awonuga and Prof. F. K. Hymore for their support to the Postgraduate program at

Covenant University. I am equally grateful to the Head of Department of Civil Engineering, Prof. A.S. Adedimila and Prof. J.B. Adeyeri for their efforts towards the success of this program. My heartfelt gratitude goes to Prof. K. O. Okonjo and Prof. J. O. Bello who gave their time and support to see that this work gets completed in good time. My appreciation also goes to the entire staff of my department, both academic and non-academic for their contributions towards the successful completion of this study.

My acknowledgement will be incomplete without mentioning the input of the Department of State and Cultural Affairs, United States of America who at the crucial stage of this research, awarded me the Fulbright scholarship (administered by Institute of International Education) to two universities in United States of America: University of Oklahoma, Norman, where I attended a summer intensive study program for 3 months under the supervision of Associate Director, Donna Deluca and her entire team. I appreciate your love and kindness. The second university I attended was North Carolina State University, Raleigh, where I carried out 9 months intensive study and research for the completion of my Ph.D thesis. I will not forget to mention my first contact at Raleigh, John and Jean Hawes for picking me at the airport and hosting me in their home in Raleigh for few days before getting my apartment. These elderly couples are so much appreciated for the love and sacrifice shown to me.

My research work crosses a number of areas: Leslie and Vicky were very helpful in their approach to the teaching of technical report writing at the University of Oklahoma. Rosario, Sumeet and Chang have been invaluable with regard to setting me up for the modeFRONTIER optimization tool. Professors Duncan, Agunwamba and Shilton are recognized authority on waste stabilization pond technology. They provided literature materials to me that helped in understanding design of the model waste stabilization pond. I appreciate you so much.

I also acknowledged my parents Mr. and Mrs. OLUKANNI for their support in prayers and endurance of taking care of our son during the period when my wife and I were studying. I thank them for giving me a good foundation in education. I pray that they will reap the fruits of their labor abundantly. Amen. I also acknowledge the prayers of my inlaws, Mr. and Mrs. BAYODE who call quite often while I was studying abroad. I express my gratitude to my immediate family, particularly my wife, Eyiyemi and son, David Olaoluwa who supported me while I was away for complete one year in the United States of America for the Fulbright program. My gratitude goes to my siblings; Biodun, Bukky, Funmi, Wole, Wumi and also my loved ones; Oye, Diran and Olaitan for being part of the wonderful family. I appreciate the role of Pastor Mike Ogboluchi at getting the WEST software at the beginning of this work and the duo of Dr. Adebayo and Lorenzo Watson for their kindness while I was away in the United States of America.

I sincerely say thank you to all and those whose names have not been able to mention both home and abroad who have contributed to the success of this program. God bless you all.

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Abbreviations and Symbols

ABU	Ahmadu Bello University, Zaria
APHA	American Public Health Association
ASCII	American Standard Code for Information Interchange
ASS	Atomic Absorption Spectrophotometry
A_f	area of facultative pond/reactor
A_m	area of maturation pond/reactor
BOD_5	biochemical oxygen demand for 5 days
B_e	number of FC/100ml of effluent
B_i	number of FC/100ml of influent
c	tracer concentration
CEC	cation exchange capacity
CFD	computational fluid dynamics
CLMT	Canaan Land Mass Transit
COD	chemical oxygen demand
CSTR	complete stared treatment reactor
DOS	disk operating system
DO	dissolved oxygen
DOE	design of experiment
D_i	diffusion coefficient
D_a	depth of anaerobic pond/reactor
D_f	depth of facultative pond/reactor
D_m	depth of maturation pond/reactor
DF	dispersed flow

d	dispersion number
dx, dy, dz	differential change in distance
dt	differential change in time
D	coefficient of longitudinal dispersion
E	eddy viscosity coefficient
FC	faecal coliform
F	volume force in the x and y direction
FEPA	Federal Environmental Protection Agency
Fr	Froude number
g	acceleration due to gravity
GA	genetic algorithm
H^2S	hydrogen sulphide gas
HRT	hydraulic retention time
JAMB	joint admission and matriculation board
$K_B(T)$	first-order rate constant removal of pollutant at temperature T
$K_1(20)$	first-order rate constant removal of pollutant at temperature T
K_{BODP}	BOD removal constant rate in the plug flow pond model
$K_{BOD\,D}$	BOD removal constant rate in the dispersed flow pond model
$K_{BOD\,C}$	completely mixed first-order rate constant for BOD removal
L_e	effluent BOD5 concentration
L_i	influent BOD5 concentration
l	length of flow path
L	pond length
L_o	baffle opening
L_r	Scale ratio

MOGA	multi-objective genetic algorithm
NH_3-N	Ammonia- nitrogen
OAU	Obafemi Awolowo University, Ile-Ife, Nigeria
p	pressure
pH	activity of hydrogen ions = log10(hydrogen ion concentrations)
POTW	public owned treatment works
PPDU	physical planning and development unit
PVC	polyvinyl chloride
Q_m	waste water flow rate in model
Q_p	waste water flow rate in prototype
Q	mean design wastewater flow
Q^r	recycled flow rate
RBC	rotating biological contactors
R(c)	rainfall/evaporation rate.
R^2	coefficient of correlation
Re	Reynolds number
RTD	residence time distribution
$S\phi$	source term of scalar variable ϕ
SO_4	sulphate
SS	suspended solids
SSE	sum of squares of errors
SL	scaling factor
T	temperature
t	time taken by tracer from the inlet to outlet
t*	retention time in any pond

TN	Total Nitrogen
$\bar{t}$	mean hydraulic retention time obtained from tracer experiments
TSS	total suspended solid
UN	United Nations
USA	United States of America
USEPA	United State Environmental Protection Agency
USAID	United States Agency for International Development
UDF	user defined function
UNN	University of Nigeria, Nssuka
u	velocity
UASB	upflow anaerobic sludge blanket
u,v,w	velocity in x,y and z directions
UV	ultraviolet radiation
V_m	velocity of model
V_p	velocity of prototype
WW	wastewater
w	baffle spacing, pond width
WA	Western Australia
WHO	World Health Organization
WSP	waste stabilization pond
WWTP	wastewater treatment plant
θ	theoretical hydraulic retention time
λ_s	surface BOD loading
λ_v	volumetric organic loading rate

^0C	degree Celsius
∞	infinity
ρ	wastewater density
μ	dynamic viscosity
ϕ	scalar variable
σ	source/ sink of constituent
2D	two dimensional
3D	three dimensional
Γ	gamma function
ξ	empirical wind shear coefficient
ψ	wind direction
ω	rate of earth`s angular rotation
φ	local latitude
η	dynamic viscosity
δ_{st}	time-scaling coefficient
υ	kinematic viscosity

ABSTRACT

Wastewater stabilization ponds (WSPs) have been identified and are used extensively to provide wastewater treatment throughout the world. It is often preferred to the conventional treatment systems due to its higher performance in terms of pathogen removal, its low maintenance and operational cost. A review of the literature revealed that there has been limited understanding on the fact that the hydraulics of waste stabilization ponds is critical to their optimization. The research in this area has been relatively limited and there is an inadequate understanding of the flow behavior that exists within these systems. This work therefore focuses on the hydraulic study of a laboratory-scale model WSP, operated under a controlled environment using computational fluid dynamics (CFD) modelling and an identified optimization tools for WSP.

A field scale prototype pond was designed for wastewater treatment using a typical residential institution as a case study. This was reduced to a laboratory-scale model using dimensional analysis. The laboratory-scale model was constructed and experiments were run on them using the wastewater taken from the university wastewater treatment facility. The study utilized Computational Fluid Dynamics (CFD) coupled with an optimization program to efficiently optimize the selection of the best WSP configuration that satisfy specific minimum cost objective without jeopardizing the treatment efficiency. This was done to assess realistically the hydraulic performance and treatment efficiency of scaled WSP under the effect of varying ponds configuration, number of baffles and length to width ratio. Six different configurations including the optimized designs were tested in the laboratory to determine the effect of baffles and pond configurations on the effluent characteristics. The verification of the CFD model was based on faecal coliform inactivation and other pollutant removal that was obtained from the experimental data.

The results of faecal coliform concentration at the outlets showed that the conventional 70% pond-width baffles is not always the best pond configuration as previously reported in the literature. Several other designs generated by the optimization tool shows that both shorter and longer baffles ranging between 49% and 83% for both single and multi-objective optimizations could improve the hydraulic efficiency of the ponds with different variation in depths and pond sizes. The inclusion of odd and even longitudinal baffle arrangement which has not been previously reported shows that this configuration could improve the hydraulic performance of WSP. A sensitivity analysis was performed on the model parameters to determine the influence of first order constant (k) and temperature (T) on the design configurations. The results obtained from the optimization algorithm considering all the parameters showed that changing the two parameters had effect on the effluent faecal coliform and the entire pond configurations.

This work has verified its use to the extent that it can be realistically applied for the efficient assessment of alternative baffle, inlet and outlet configurations, thereby, addressing a major knowledge gap in waste stabilization pond design. The significance of CFD model results is that water and wastewater design engineers and regulators can use CFD to reasonably assess the hydraulic performance in order to reduce significantly faecal coliform concentrations and other wastewater pollutants to achieve the required level of pathogen reduction for either restricted or unrestricted crop irrigation.

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study.

The practice of collecting, treating and proper management of wastewater prior to disposal has become a necessity in developing and modern societies. This is because the consequences due to poor management of wastewater treatment systems have become enormous. Moreover, the need to minimize waste and make the most valuable use of nutrients present in wastewater is receiving global interest. Bixio et al. (2005) pointed out that the world's freshwater resources are strained; therefore, reuse of wastewater, combined with other water conservation strategies can lessen anthropogenic stress arising from over-extraction and pollution of receiving waters.

As reported by the World Health Organization (2000), "despite tremendous efforts in the last two decades to provide improved water and sanitation services for the poor in the developing world, 2.4 billion people world-wide still do not have any acceptable means of sanitation, while 1.1 billion people do not have an improved water supply". This indicates that less than 1% and 15% of the wastewaters collected in sewered cities and towns in Africa and Latin America, respectively, are treated in effective sewage treatment plants. Mara (2001) also reported that out of the world population of just over 6 billion, 4 billion lack wastewater treatment and this is expected to rise to 7.8 billion by 2025. Most of these live in developing countries where energy-intensive electro-mechanical wastewater treatment (the type favoured in industrialized countries) is too expensive and too difficult to operate and maintain.

Wastewater is the water that has been adversely affected in quality by anthropogenic influence. It comprises liquid waste discarded by domestic residences, commercial properties, industry, and/or agriculture. Banda (2007) emphasized the significant composition of wastewater as the degradable organic compounds and these form an excellent diet for bacteria and are exploited in biological treatment of wastewater. In order to reduce the transmission of the excreta-related diseases and the damage to aquatic biota, Mara (2004) express the necessity to treat wastewater to meet the consent requirements of the effluent quality set by the regulatory agencies.

The safe disposal of wastewater has been a great concern in developing nations, most especially in Nigeria. In 2008, estimated population of Nigeria was 151.5 million (UN, 2008) yielding an average density of 151 persons per sq km covering an area of 923,768 sq km (356,669 sq miles). The population is projected to grow to 206 million by 2025 (Microsoft Encarta Encyclopedia, 2005). Treatment ponds serve thousands of communities around the world. In many cases, they are the only form of environmental protection that stands between raw sewage and natural waterways. Unfortunately, ignorance and lack of knowledge are responsible for poor wastewater treatment in many of these nations (Mara, 2004).

Among the current, globally available processes used for wastewater treatment, waste stabilization pond (WSP) has been identified as the ideal treatment of municipal wastewaters in the tropics. This technology is well known for its simplicity of construction and operation (Mara, 2004). Shilton and Bailey (2006) noted that the only thing standing between raw sewage and the environment, into which it is ultimately discharged, is a waste stabilization pond. WSP has been emphasized to be the first choice wastewater treatment facilities in developing countries as these operate extremely well in tropical regions at low-cost (Mara et al. 1992; Mara, 1997; Mara and Pearson, 1998). The WSP system typically consists of a series of continuous flow anaerobic, facultative, and maturation ponds. The anaerobic pond, which is the initial treatment reactor, is designed for eliminating suspended solids and some of the soluble organic matter. The residual organic matter is further removed through the activity of algae and heterotrophic bacteria in the facultative pond. The final stage of pathogens and nutrients removal takes place in the maturation pond. These three types of ponds when used in series, have demonstrated up to 95% removal of BOD and fecal coliform. (Mara, 2004; Hamzeh and Ponce, 2007).

In Nigeria, WSPs have been installed in some universities for domestic wastewater treatment. From preliminary investigation, the design population for most of these Nigerian ponds has been exceeded and has lead to serious overloading problems from deficient hydraulic designs (Agunwamba, 1994 and 2001; Oke and Akindahunsi, 2005; Olukanni and Aremu, 2008). Despite abundant lands, large surface area ponds and favourable temperatures, the waste stabilization ponds are not able to produce treated effluents that meet the minimum standard limits set by Nigeria's Federal Environmental Protection Agency (FEPA, 1991). Most waste stabilization ponds in residential academic institutions discharge their effluents into the environment without adequate treatment which cause great negative impact on the aquatic life of the receiving streams.

Research is therefore needed to address the weakness in the pond`s treatment capacity. It is believed that processes exist within ponds that, if understood better, could be optimized to enable effective treatment of wastewater with limited cost of construction and maintenance. This is the main focus of the research project. Previous studies have shown that the WSP treatment efficiency is often hydraulically compromised (Shilton and Mara, 2005; Shilton and Harrison 2003a). The majority of the hydraulic studies on WSPs have been performed on full-scale field ponds, which have transient flows and large surface areas exposed to wind and temperature variations (Marecos and Mara, 1987; Moreno, 1990; Agunwamba, 1992; Fredrick and Lloyd, 1996; Adewumi et al, 2005).

Researchers have been challenged to reliably predict the impact of pond design modifications, such as placement and number of baffles and different flow rates on the pond performance using field scale WSPs. Antonini et al., (1983) and later Shilton and Bailey (2006) noted that operation and weather variations that occur during field experimental tests limit the study of retention time distribution only with scale models studied under controlled conditions.

Banda (2007), Shilton and Harrison (2003a), Sperling et al. (2002), Muttamara and Puetpailboon (1996,1997) and Kilani and Ogunrombi (1984) all observed higher fecal coliform removal in WSPs that were fitted with baffles of various configurations than unbaffled WSPs. These researchers concluded that the 70% pond width baffle is the optimal pond configuration that provides the best WSP treatment efficiency. A number of previous studies have also discussed the idea of optimizing the cost of treatment plant construction and operation of WSPs (Oke and Otun, 2001; Shilton and Harrison, 2003b; Bracho et al, 2006). These researchers have concluded that while additional baffles produce better hydraulic efficiency, the inclusion of construction cost may provide a better understanding on the cost effectiveness of increasing baffle number on the WSP treatment efficiency.

Some research studies have also shown that pond depth could be increased beyond the generally accepted value of 2.5m for anaerobic, 1.5m for facultative and 1m for maturation particularly in tropical countries with an abundance of sunlight (Agunwamba, 1991; Mara and Pearson, 1998; Mara, 2004). Agunwamba, (1991) applied a graphical method to cost minimization in WSPs subject to area, depth, and efficiency constraints. Although shallow ponds have been reported to produce better effluents than deeper ponds at equal retention time, the former require a larger area to treat waste of equal strength (Agunwamba, 1991; Hamzeh and Ponce, 2007). Hence, there is a need to balance efficiency and pond area during WSP design process. Such a compromise may be developed through a rigorous optimization technique, which yields a design at a minimum cost while satisfying the constraints imposed by land requirement and efficiency.

Based on the situation already presented, it becomes important to investigate and improve further on the functioning and performance of waste stabilization pond currently in use. WSP has been designed, operated and evaluated in many developed and African countries for the treatment of wastewater. However, research on the hydraulic modeling and optimization of the pond configuration is still limited. It is worth noting that WSP models proposed in manuals are to serve as guide and local experience from pilot and full scale plants of a particular type is needed. Pond hydraulics in Nigeria has been scantily researched because of climatic variation, low velocity and long residence time, and the systems are difficult to systemically study in the field. An alternative is to undertake research on scale-model ponds operated under controlled conditions in a laboratory.

Limited studies have been performed using numerical models to help quantify and elucidate the WSP performance. Recently, researchers have extensively become interested in the use of computer simulation using the PHOENICS, FLUENT, FIDAP, MIKE 21, FLOWWIZARD, POLYFLOW, and GHENT Computational Fluid Dynamics (CFD) modelling tools to investigate potential hydraulic improvements due to different inlet, outlet and baffling configurations which allow quantitative evaluation of treatment performance given by any pond shape or configuration. In addition to predicting the hydraulics of ponds, it is relatively easy for these models to incorporate first order kinetics.

The use of COMSOL Multiphysics model building and dynamic simulation in this study has given a better insight to the behavior of the system, so that optimal performance of the system through the use of modeFRONTIER optimization tool is ensured, resulting in cost reduction without jeopardizing the treatment efficiency. These approaches use mathematical models that give a reliable image of the existing and optimized systems respectively. The optimization program utilizes a single objective (Simplex) that is based on minimizing cost alone and a multi-objective function that utilizes genetic algorithm and is based on simultaneously minimizing cost and maximizing the treatment efficiency. The results of this study will serve as a guide for the design of treatment and reuse systems that will boost the environmental protection drive of the Nigerian Government.

1.2 Problem Statement

The efficiency of existing ponds system is often compromised by hydraulic problems and it is necessary to reliably predict how various modifications of pond design might affect pond performance. In addition, no rigorous assessment of WSPs that account for cost in addition to hydrodynamics and treatment efficiency has been performed. The research in this area has been relatively limited. Hence, there is need to balance efficiency and pond configuration during WSP design process.

1.3 Aim of the Research

This study is aimed at developing a CFD-based optimization model as an innovative tool for the design of waste stabilization ponds that incorporates the effects of different pond footprint and number, length, and placement of baffles on the WSP treatment performance.

1.4 Objectives

The principal objectives to achieve the aim of this research are as follows:

1. To determine the total water supply, the per capita demand and the initial physico-chemical and biological characteristics of raw wastewater in a typical residential institution.
2. To design, construction and evaluation of a laboratory-scale treatment plant model to study how to improve on the existing treatment system designs.
3. To calibrate and test a CFD model using the data collected in (2)
4. To run the CFD-based model on COMSOL Multiphysics software under different flow scenarios- the effects of different pond foot print size, incorporation of different baffles configurations and length on the treatment performance of waste stabilization ponds.
5. To use the developed model for the optimization of the design of the treatment system by adopti**ng** mode FRONTIER tool**.**

1.5 Scope of study

The scope of this study is limited to laboratory-scale modeling of the treatment system and effluent recirculatory methods. Variation of influent and effluent parameters (physical, chemical, biological and physico-chemical characteristics) were determined and CFD has been used as a reactor model to simulate faecal coliform removal, the velocity field and the residence time distribution in the baffled WSPs.

1.6 Justification of study

Based on the situation already presented, the current study is to investigate the performance of waste stabilization pond and address the literature gaps:

1. Though some institutions have WSP facilities in place for treating their wastewater, an evaluation of these WSPs have shown that they hardly meet the standard for effluent discharge and reuse purposes.
2. The efficiency of existing ponds system is often compromised by hydraulic problems and it is necessary to reliably predict how various modifications of pond design, such as placement and number of inlets, use of baffles, etc, might affect pond performance.
3. Simulation models are now been increasingly used to gain better insight into water resources and environmental processes. Therefore, the development of an appropriate simulation model will facilitate the design an efficient WSP.

1.7 Limitations of the work

1. The simulation performed in this study did not include potential physics that may occur in field WSPs such as surface wind shear, solar radiation, relative humidity and air temperature that may impact WSP design decisions. In addition, the optimized solution was based only on the disinfection process, the kinetics of fecal coliform, and a specific cost objective function along with the associated constraints.
2. The addition of the full scale physics, other target contaminant removal, microbial disinfection kinetics, as well as different objective functions with constraints may result in alternative optimal design solutions.
3. The CFD model was based on steady flow rate condition, while in the field pond; the flow rate can vary continuously both diurnally and with rainfall. The wastewater density in the CFD model was taken to be uniform throughout the pond which may not be so on the field due to sludge material build up which could increase the inlet velocity higher than that which was allowed for in the CFD model.

CHAPTER TWO

LITERATURE REVIEW

2.1 The pressure on water demand.

The increasing scarcities of water in the world along with rapid population increase in urban areas are reasons for concern on the need for appropriate water management practices. Throughout history, the prosperity of nations has always been known to correlate very closely with the management of water resources and the well-being of future generations depends largely on its wise management (Olukanni and Alatise, 2008). According to the World Bank, "The greatest challenge in the water and sanitation sector over the next two decades will be the implementation of low cost sewage treatment that will at the same time permit selective reuse of treated effluents for agricultural and industrial purposes"(Navaraj, 2005).

Population growth will continually increase the demand for water thereby forcing water agencies to look for alternative ways to manage the available resources. It is well known that most of the projected global population increases will take place in the third world countries that already suffer from land, water, food and health problems. However, liberation of water for the environment through substitution with wastewater has now been promoted as a means of reducing anthropogenic impact (Hamilton et al. 2005 a, b). The reuse of such treated wastewater as irrigation water which returns vital nutrients to the soil that would be expensive to add in other forms, therefore, can be used as strategy to release freshwater for domestic use, and to improve the quality of river waters used for abstraction of drinking water (by reducing disposal of effluent into rivers) (Oron, 2002; Adekalu and Okunade, 2002). Therefore, the key challenge facing many countries is to develop strategies to meet the increasing water demands of society but which do not further degrade the integrity of the environment.

2.2 Wastewater Treatment Systems in Use

There are currently a wide variety of systems that can be successfully applied to wastewater treatment. They should, however, be selected on the basis of the specific local context. In developing countries, the number of choices may be higher as a result of the more diverse discharge standards encountered. In any case, effective standards vary from the very conservative to the very relaxed (Pena, 2002). Some of different wastewater treatment processes which are in use globally are; activated sludge plant, package plants, external aeration activated sludge, biological filter, oxidation ditch, aerated lagoon and waste stabilization Pond (WSP). Of all these treatment processes currently available, WSPs are the most preferred wastewater treatment process in developing countries. WSPs are robust and operationally simple wastewater technology that provides a considerable degree of economical treatment especially where land is often available at reasonable opportunity costs and skilled labor is in short supply (Wood et al., 1995; Agunwamba, 2001; Mara, 2004; Abbas et al., 2006; Kaya et al., 2007; Naddafi et al., 2009).

Table 2.1 below shows the various factors being considered in the selection of an appropriate treatment system both in developed and developing countries.

Table 2.1 Important factors in the selection of wastewater treatment systems in developed and developing countries.

Factors	Developed Countries		Developing Countries	
	Critical	Important	Important	Critical
Efficiency				
Reliability				
Sludge disposal				
Land requirements				
Environmental costs				
Operational costs				
Construction costs				
Sustainability				
Simplicity				

Source: Sperling (1995)

The table shows that the important factors in developed countries are: efficiency, reliability, sludge disposal, and land requirements, whereas in developing countries the critical factors are operational costs, construction costs, sustainability and simplicity.

Table 2.2 also provides a comparison of the current treatment systems in use worldwide, identifying the advantages and disadvantages of ponds with those of high-rate and low-rate biological wastewater treatment processes.

Table 2.2 Advantages and disadvantages of various sewage treatment systems (Arthur, 1983)

	Criteria	Package plant	Activated Sludge plant	Extended aeration activated sludge	Biological filter	Oxidation ditch	Aerated lagoon	Waste stabilization pond system
Plant performance	BOD removal	F	F	F	F	G	G	G
	FC removal	P	P	F	P	F	G	G
	SS removal	F	G	G	G	G	F	F
	Helminth removal	P	F	P	P	F	F	G
	Virus removal	P	P	P	P	F	F	G
Economic factors	Simple and cheap construction	P	P	P	P	F	F	G
	Simple operation	P	P	P	F	F	P	G
	Land Requirement	G	G	G	G	G	F	P
	Maintenance costs	P	P	P	F	P	P	G
	Energy Demand	P	P	P	F	P	P	G
	Sludge removal costs	P	F	F	F	P	F	G

FC= Faecal Coliform
SS= Suspended Solids
G= Good
F= Fair
P= Poor

2.3 Waste Stabilization Ponds (WSP).

WSPs above all other methods of treatment technologies are now regarded as the first choice for treatment of wastewater in many parts of the world (Agunwamba, 2001; Mara, 2004; Abbas et al., 2006; Kaya et al., 2007; Naddafi et al., 2009). It is particularly well suited for tropical and subtropical countries because the intensity of the sunlight and temperature are key factors for the efficiency of the removal processes (Mara, 2001, 2004; Mara and Pearson, 1998). Agunwamba et al. (2003), Hamzeh and Ponce, (2007) and Mara (2004) describes WSPs as large shallow basins enclosed by natural embankments in which decomposition of organic matter in wastewater is processed naturally (biologically). Bacteria and algae in the WSP stabilize the organic waste and lower the effluent pathogen levels.

WSPs have also been used widely in cold climate regions (Abis, 2002). A detailed description of the overwhelming advantages of waste stabilization ponds over conventional sewage treatment plants in terms of capital cost, operation and maintenance costs, pollutants removal and the heavy metal removal was stated by Mara (2004). However, waste stabilization ponds release odour when they are overloaded. They work through employing natural influences (wind, sun, gravity and biological processes) that serve to provide primary and secondary treatment over a period of days.

Khowaja (2000) expressed that ponds in this system are classified according to the relative dominance of two processes; anaerobic digestion and aerobic bacterial oxidation by which organic material (BOD), suspended solids and bacteria are removed. They are designed to achieve different forms of treatment up to three stages in series, depending on the organic strength of the input waste and the effluent quality objectives. Well-designed WSP can achieve very high BOD, nitrogen and phosphorous removal rates (Mara and Pearson, 1998). Mara and Pearson (1986) maintained that pond research programs are designed to further knowledge on pond systems by achieving certain specified objectives.

Although waste stabilization ponds are simply constructed, their effectiveness depends upon a complex interaction of physical, chemical and biological processes. Ponds within a system have been classified according to the principal biological process. Middlebrooks et al., (1977); Water Environment Federation, (1998) in Abis (2002) expressed that ponds are also classified by their treatment objective and even by their hydraulic regime. Several different ponds are usually used together in series to provide a complete treatment system. Figure 2.1 below shows different waste stabilization pond configurations.

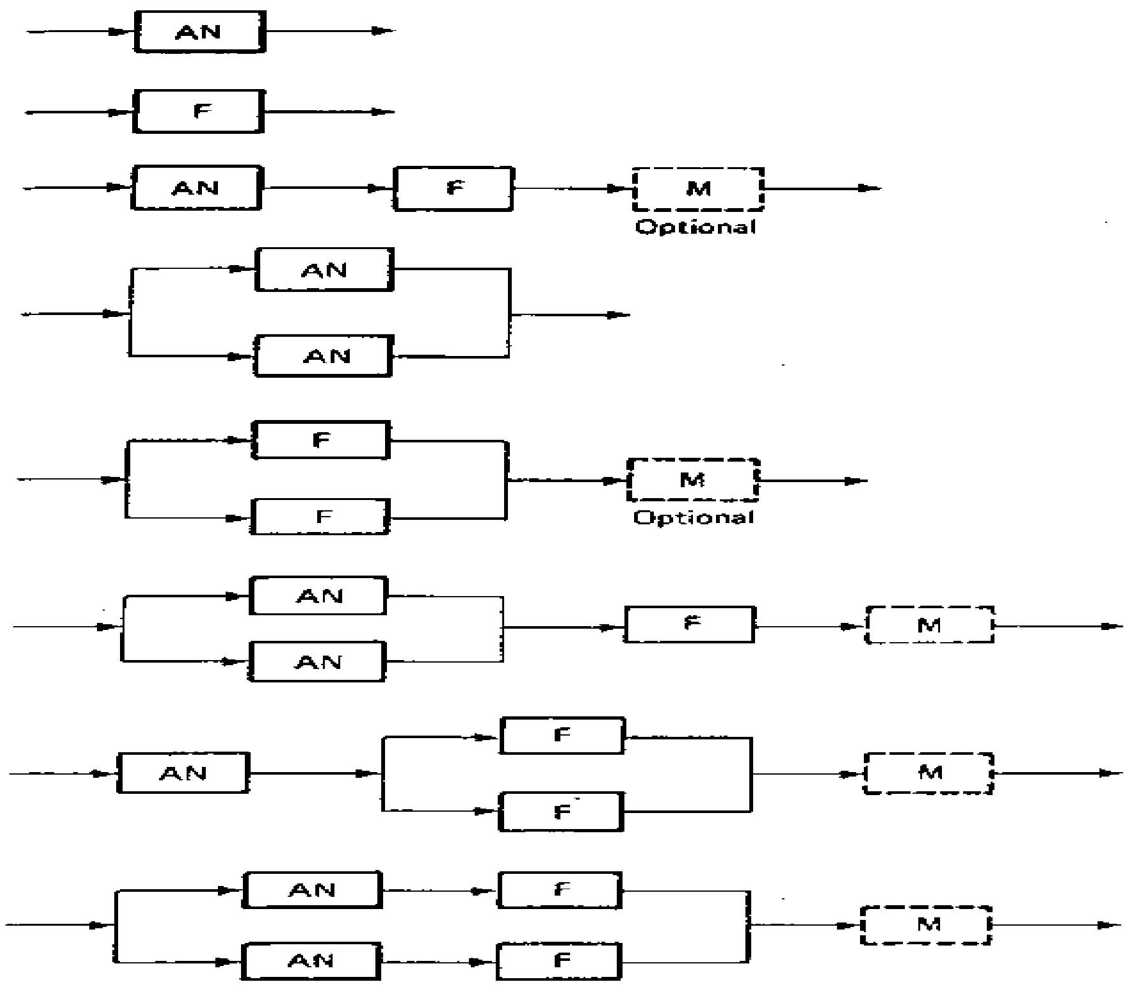

Figure 2.1 Waste stabilization pond configurations: AN = anaerobic pond; F = facultative pond; M = maturation pond (Pescod and Mara, 1988).

2.3.1 Treatment Units in Waste Stabilization Ponds.

Waste stabilization ponds have been described as large shallow basins enclosed by earthen embankments in which wastewater is biologically treated by a natural process. The three principal units consists of anaerobic, facultative and maturation ponds. These pond systems are normally arranged in a single or parallel series depending on the requirements of the final effluent quality (Agunwamba, 1991; Mara 2004, Lloyd et al., 2003; Guganesharajah, 2001; U.S. EPA, 2002). In these units, wastewater is treated by natural process based on the activities of both algae and bacteria. Ponds in these units are classified according to the relative dominance of anaerobic bacteria oxidation by which organic material (BOD), suspended solids and bacteria are removed (Khowaja, 2000; Shilton and Bailey, 2006; Fyfe et al., 2007). The removal efficiency of pathogens in wastewater treatment plants is one of the most important treatment objectives for the public health protection especially when effluent reuse schemes are implemented. In natural system such as WSP, pathogens are progressively removed along the ponds series with the highest removal efficiency taking place in the maturation ponds (Mara et al, 1992).

2.3.2 Anaerobic Ponds

These units are the smallest of the series. Commonly they are 2-5 m deep and receive high organic loads equivalent to 100g BOD_5/m^3d which is equivalent to more than 3000kg BOD/ha-d for depth of 3m. These high organic loads produce strict anaerobic conditions (no dissolved oxygen) throughout the pond. In general terms, anaerobic pond function much like open septic tank and work extremely well in warm climates (Mara et al., 1992). These units are used as the first treatment stage for high-strength wastewaters in systems comprising a series of ponds. In this way, anaerobic ponds produce a reduction in influent organic load of 50 percent or more (Droste, 1997). Figure 2.2 describes the processes that occur in anaerobic pond with organic matter and nutrients as effluent. The wastewater goes through the decomposition process to produce a higher quality 70% less BOD as the effluent.

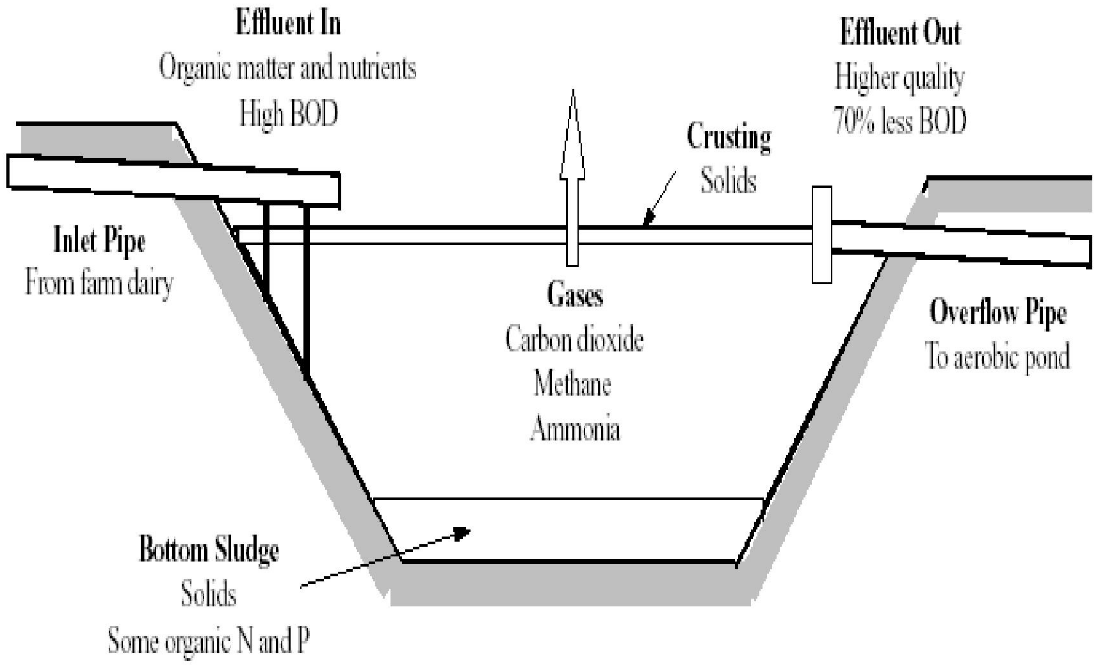

Figure 2.2 Operation of the Anaerobic Pond.

The work done by Mara et al. (1992) and Mara and Mills (1994) has shown that anaerobic pond work extremely well in warm climate. In such conditions and provided that the anaerobic pond has been properly designed and is not significantly overloaded, it will achieve around 60% BOD_5 removal at 20° C and as much as 75% at 25°C. Modern anaerobic ponds operate with a minimum hydraulic retention time of 1 day and their inclusion in a pond system provides a land area saving of over 75 percent at temperature above 16°C (Mara and Mills, 1994; Pescode 1996). One-day hydraulic retention time is sufficient for wastewater with a BOD_5 up to 300 mg/l and temperatures higher than 20° C.

Research findings by Silva (1982) reported that anaerobic ponds operated extremely well at short retention times and it was not good practice to design anaerobic ponds at long retention times as this did not offer any advantage. The liquid retention time, pond temperature, nutrient availability and pond geometry are the main design operating parameters that affect the organic load removal in the anaerobic ponds. Designers have always been preoccupied by the possible odour they might cause. Mara (2001, 2004) strongly advised that well-designed anaerobic ponds with pH of 7.5 do not produce odour.

In addition, limiting sulphate compounds (SO_4) concentration in raw wastewater to less than 300 mg/l is one way of eliminating odour release in anaerobic ponds. This could be achieved if the source of potable drinking water complies with the WHO (2003) guideline, which limits the sulphate concentration in raw water to 250 mg/l. The removal of organic matter in anaerobic ponds follows the same mechanisms that take place in any anaerobic reactor (Mara et al., 1992; Pena, 2002).

2.3.2.1 Design approach for anaerobic pond

In order to keep anaerobic conditions within the pond, these treatment units are designed based on the volumetric BOD_5 loading. Meiring et al. (1968) proposed that the permissible volumetric organic BOD loading rate should be within a range of 100 - 400 g/m^3/day to ensure the satisfactory treatment performance of anaerobic ponds. The upper limit of the volumetric organic BOD loading rate (400 g/m^3/day) was established to avoid the risk of odour release due to the hydrogen sulphide gas (H_2S).

Mara (2004) revised the design BOD loading rate from 300 g/m^3/day to 350 g/m^3/day following the WHO (2003) guideline for the drinking water, which limits the sulphate concentration in raw water to 250 mg/l. It was suggested that the design BOD loading rate (350 g/m^3/day) could not produce odour at the recommended sulphate concentration (250 mg/l). In addition, the design of anaerobic ponds would use less land compared with the earlier BOD loading rate (300 g/m^3/day). Due to scarcity of reliable data, the development of a suitable design equation for anaerobic ponds has not yet been achieved. Nevertheless, the results of the Kenyan Pond Study (Mara et al, 1990) indicated that the general recommendations given by Mara and Pearson (1986) can be used safely for design purposes in tropical regions.

The design approach discussed by the researchers above assumed that the completely mixed hydraulic flow pattern was realized in anaerobic ponds due to the use of the low ratio of length to width of the pond dimensions. This procedure was not based on the actual hydraulic flow patterns that existed in anaerobic ponds. Agunwamba (1991) shows how optimal design could be achieved by a graphical approach. This illustrates better with unique clarity the relationship between the various variables and allows for the imposition of constraints whose state equations are lacking. Research findings by Pena et al (2000, 2002) and Tchobanoglous et al (2003) have shown that the completely mixed hydraulic flow pattern is not achieved in operational anaerobic ponds due to the changes in wind velocity, temperature, influent momentum and the density variation of the wastewater despite the use of square shape dimensions in the anaerobic pond geometry.

Although the process design of anaerobic pond is based on volumetric organic BOD loading rate (Mara, 2004), this classic method currently used to design and evaluate the treatment efficiency of WSP do not account for the hydraulic short-circuiting and stagnations that are inherent in many WSP (Buchauer, 2007; Pena, et al., 2000). One can indeed raise a question about the effectiveness of classic pond design methods in using the nominal hydraulic retention times when designing and assessing the treatment efficiency of waste stabilization ponds. In addition, the classic methods treat the design parameters (population, influent flow, BOD, temperature, faecal coliform numbers etc) with high certainty. However, it was noted by Sperling (1996) that design parameters are not known with high certainty in developing countries because of the limited research resources and he introduced modern methods of designing waste stabilization ponds based on uncertainty principles.

Wood et al (1998) emphasized that the traditional WSP models encounter problems in predicting pond performance because they cannot account for the influence of pond features, such as inlet structure or pond geometry, on fluid hydrodynamics. Shilton and Mara (2005) also maintained that the modern design procedures for waste stabilization ponds can manage the uncertainty of the design variables by considering the range of every design parameter depending on the level of its uncertainty. The method does not account for the improved treatment efficiency that is initiated when baffles of various configurations are installed in waste stabilization ponds.

Several researchers have observed improved performance when baffles of various configurations were fitted in WSP (Shilton and Mara, 2005; Pearson et al., 1995; Mangelson and Watters, 1972; Muttamara and Puetpaiboon 1996 & 1997; Thackston et al.

1987). However, this benefit of the improved pond performance is not utilized in the current design procedures when sizing waste stabilization ponds. This suggests that an innovative design approach is required to overcome the limitations of classic and modern design procedures for waste stabilization ponds. One such 'innovative design approach' is computational fluid dynamics (CFD)

2.3.3 Facultative Pond

The term facultative is used because a mixture of aerobic and anaerobic conditions is found in the pond (Mara, 2004). Aerobic conditions are maintained in upper layer while anaerobic conditions exist towards the bottom of the pond. These ponds are of two types: primary facultative ponds which receive raw wastewater and secondary facultative ponds that receive the settled wastewater from the first stage (usually the effluent from anaerobic ponds). They are usually 1-2 m deep with 1.5m being most common and are geometrically designed to have a high ratio of length to width (up to 10:1) to simulate the hydraulic plug flow in these ponds (Pena et al., 2003; Mara, 2004; Mara *et al.* 2001). Mara (2004) compared the design of waste stabilization ponds using modern and classic methods. It was found that modern design methods are appropriate when upgrading existing waste stabilization ponds and new waste stabilization ponds should be designed using classic methods.

2.3.3.1 Design criteria for facultative pond

Facultative ponds are designed for BOD_5 removal on the basis of a low organic surface load achieved by anaerobic pond to permit the development of an active algal population. This way, algae generate the oxygen needed to remove soluble BOD_5 (Pena and Mara, 2004). Healthy algae populations give water a dark green colour but occasionally they can turn red or pink when slightly overloaded due to the presence of anaerobic purple sulphide-oxidizing photosynthetic bacteria and due to large number of micro-algae (Mara and Pearson 1986; Mara, 2004).

Thus, the change of colouring in facultative ponds is a qualitative indicator of an optimally performing removal process. The concentration of algae in an optimally performing facultative pond depends on organic load and temperature, but is usually in the range 300 to 2000 µg chlorophyll per litre (Mara, 2004). The photosynthetic activity of the algae results in a diurnal variation in the concentration of dissolved oxygen and pH values. Variables such as wind velocity have an important effect on the behaviour of facultative ponds, as they generate the mixing of the pond liquid. As (Mara et al., 1992) indicated, a good degree of mixing ensures a uniform distribution of BOD_5, dissolved oxygen, bacteria and algae, and hence better wastewater stabilization.

Mara (2004) maintained that when designing for facultative ponds, emphasis must be given to the surface area. Increasing the surface area of the facultative pond will improve the performance of the system. A minimum value of t of 5 days should be adopted for temperatures below 20^oC, and 4 days for temperatures above 20^oC. This is to minimize hydraulic short-circuiting and to give the algae sufficient time to multiply (i.e. to prevent algal washout). However, some researchers have observed that the theoretical retention time is not achieved in facultative WSP due to the existence of the hydraulic short-circuiting and formation of stagnations (Mangelson and Watters, 1972; Lloyd et al., 2003). These hydraulic factors are inherent in many operational waste stabilization ponds. Effects of thermo-stratification and wind velocity are thought to cause the hydraulic short-circuiting that diminishes the treatment efficiency of facultative ponds (Banda, 2007).

The BOD removal in primary facultative ponds is usually in the range 70-80 percent based on unfiltered samples (that is, including the BOD exerted by the algae), and usually above 90 percent based on filtered samples. In secondary facultative ponds the removal is less, but the combined performance of anaerobic and secondary facultative ponds generally approximates (or is slightly better than) that achieved by primary facultative ponds. According to Marecos do Monte and Mara (1987), the process design of facultative ponds is based on rational and empirical approaches. The empirical design approach has been developed using performance data of existing waste stabilization ponds.

As reported by Banda (2007), Sperling (1996) proposed an uncertainty principle to design facultative ponds based on random design values selected in a range of each design parameter depending on the degree of its certainty. This is because it was noticed that the input design parameters used in deriving surface loading BOD equations are not known with high certainty in developing countries due to the limitation of research resources. Therefore, in order to design facultative pond area, mean hydraulic retention time and the effluent BOD concentration, Monte Carlo design simulations was employed. Although, the Monte Carlo design simulations give confidence to the resultant designs so produced (area and effluent quality), the design approach assumes that the theoretical retention time is achieved during the operational period of the facultative pond.

Mara (2004) and Marais (1974) observed a weakness of both the Monte Carlo design simulations and the surface BOD loading approach because an assumption was made that the complete-mix hydraulic flow pattern is initiated by effects of wind and thermo-stratification. Banda (2007) noted that the classic design approach and Monte Carlo design simulations do not account for the effects of baffles or various types of inlet and outlet structures on the treatment efficiency of facultative ponds design or the operational stages. As a result of this, there is no optimization of the resultant design. This implies that there is a risk that the design of the facultative pond can require substantially more land than is actually necessary.

2.3.3.2 Surface BOD loading in facultative ponds

These ponds are usually designed by considering the maximum BOD load per unit area at which the pond will still have a substantial aerobic zone, because, biological activities dependent on the temperature. McGarry and Pescod (1970) correlated performance data of primary facultative ponds from 143 different climatic conditions and reported that BOD removal was between 70 - 90%. The statistical model found that pond performance was related to the surface BOD loading with high correlation coefficient of 0.995. It was observed that the maximum surface BOD loading rate, at which a primary facultative pond could become anaerobic (pond failure), was related to the ambient air temperature. McGarry and Pescod (1970) gave the following equation for the calculation of surface BOD loading (λs):

$$\lambda s\ (max) = 60.3(1.099)^{T-20} \qquad 2.1$$

where:

λs (max) is maximum surface BOD loading in kg/ha/day and

T = Minimum monthly mean temperature in ^{0}C.

Equation 2.1 was modified by Mara and Silva (1979) as,

$$\lambda s = 20T - 120 \qquad 2.2$$

Equation 2.2 was further changed by Arthur (1983) especially for hot climates as,

$$\lambda s = 20T - 60 \qquad 2.3$$

Mara (1987) adopted the McGarry and Pescod's failure model by incorporating a factor of safety to ensure the safe design of facultative ponds. This model ensures that healthy concentration of algae is maintained in facultative ponds to avoid development of anaerobic condition.

The tentative global equation that was developed by Mara (1987) which seems to be the most recent is

$$\lambda s = 350\,(1.107 - 0.002T)^{T-25} \qquad 2.4$$

where:

λs = surface BOD loading (kg/ha/day)

T = temperature (^{0}C)

An appreciable number of researchers have used the Arthur (1983) equation to determine the surface organic loading because it is applicable to the tropics where hot climates is persistent (Khowaja, 2000; Mohammed, 2006).

2.3.4 Model approaches for faecal coliform prediction in facultative pond.

Many studies have been conducted to identify the principal factors involved in bacterial reduction. Several researches of these studies have shown unacceptably high pathogen indicator concentrations in treatment plants effluent (Mara and Pearson 1998; Salter et al. 1999; Lloyd et al., 2003; Bracho et al., 2006). Consequently, pathogen removal is an increasingly important objective for municipal waste water treatment plants and it has been shown that bacteria removal efficiency is associated with flow patterns (Lloyd et al., 2003; Muttamara and Puetpaiboon 1997).

2.3.4.1 Continuous stirred reactor (CSTR) model approach

Marais (1974) and Mara et al. (2001) proposed a design model of predicting the effluent reduction of faecal coliform bacteria in any stabilization pond (anaerobic, facultative and maturation) to generally follow first-order kinetics and this was combined with the complete mix hydraulic flow regime. Buchauer (2007) remarked that the design of WSPs for FC removal is frequently undertaken according to the recommendations developed by Marais (1974), who assumed that FC removal can be modeled by first-order kinetics in a completely mixed reactor. This approach was later integrated into the widely used design manuals by Mara et al. (1992), Mara (1997) and Mara & Pearson (1998) and slightly optimized only recently (Mara et al. 2001). It is the typical design approach for WSPs in the Middle East, Africa and Asia. This design model is expressed as:

$$B_e = \frac{B_i}{\left(1 + k_B(T)t^*\right)} \qquad 2.5$$

where:

B_e = Number of faecal coliform/100 ml of effluent

B_i = Number of faecal coliform/100 ml of influent

$k_B(T) = 2.6(1.15)^{T-20}$ is the First-order FC removal rate constant in T^0C per day

t^* = Retention time in any pond, d

For a number of ponds in series, equation 2.5 becomes:

$$B_e = \frac{B_i}{\left(1 + k_B(T)t^*a\right)\left(1 + k_B(T)t^*f\right)\left(1 + k_B(T)t^*m\right)\ldots\ldots\left(1 + k_B(T)t^*_{mn}\right)} \qquad 2.6$$

where:

$t_{m\,n}$ = Retention time in the nth maturation pond.

t^*_a , t^*_f, and t^*_m = Retention times in the anaerobic, facultative and maturation ponds respectively.

The value of K_B given by Marais (1974) and Mara (2001) is extremely sensitive to temperature. The difference between the approaches of Marais and Mara concerns the calculation of $K_B(T)$. Both use a modified Arrhenius equation, but with different coefficients

Marais (1974) $k_B(T) = 2.6(1.19)^{T-20}$ 2.7

Mara et al. (2001) $k_B(T) = 2.6(1.15)^{T-20}$ 2.8

Banda (2007) $k_B(T) = 4.55(1.19)^{T-20}$ 2.9

where T is wastewater temperature (^{0}C).

Although, the first two approaches have the same theoretical background, i.e. both assume that FC removal can be satisfactorily modeled by a first-order kinetic model in a Complete Stirred Treatment Reactor (CSTR). Mara et al. (2001) and Buchauer (2007) recommended equation 2.8 for shallow, short retention time facultative and maturation ponds at temperatures above 20^{0}C which serve to predict FC removal for chosen WSP dimensions.

However, Banda (2007) established that the Marais` (1974) first-order rate constant removal of faecal coliform [$k = 2.6(1.19)^{T-20}$)] is not precise when predicting the faecal coliform in baffled waste stabilization ponds. The equation was only found to be accurate when predicting the faecal coliform removal in unbaffled waste stabilization ponds. Therefore a first-order rate constant removal of faecal coliform was developed for baffled waste stabilization ponds using the predicted-CFD faecal coliform counts and observed faecal coliform counts. It was observed that the effluent faecal coliform counts were estimated closely when the first-order rate constant removal of faecal coliform in the source term function was $4.55(1.19)^{(T-20)}$ day^{-1}. This equation of the first-order rate constant removal (k) of faecal coliform was used for 2 baffles and 4 baffles. It was recognized that the correlation data of the predicted-CFD faecal coliform counts and the observed effluent faecal coliform counts from the baffled pilot-scale ponds has a correlation coefficient of (R^2 = 0.8267).

A suitable design value of B_i in the case of municipal sewage treatment is 1 x 10^8 faecal coliform/100ml, which is slightly higher than average contaminant level found in the field. Equations 2.5 and 2.6 assumes that the hydraulic flow pattern in a facultative pond is completely mixed. However, it is recognized that this hydraulic flow pattern is never achieved in operational facultative ponds. As a result, the design engineer does not have confidence of the predicted BOD effluent quality. Nevertheless, the first-order rate constant for faecal coliform removal proposed by Banda (2007) can be confidently used in CFD to simulate the faecal coliform removal in baffled waste stabilization ponds. Figure 2.3 below describes the processes taking place in a facultative pond.

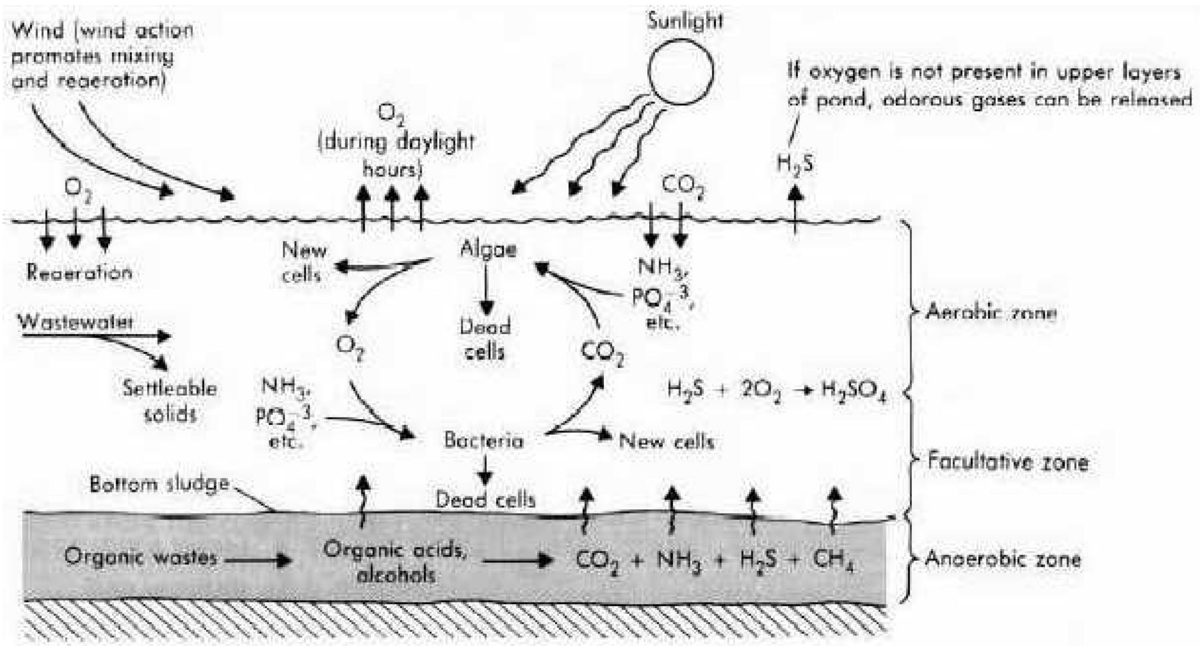

Figure 2.3 Operation of the facultative pond (Tchobanoglous and Schroeder 1985).

2.3.4.2 Dispersed flow (DF) model approach

Oakley's DF model (Buchauer, 2007) is based on the most widespread parameter assumptions. To calculate FC removal, a first-order equation for DF is used as follows:

$$B_e = \frac{4B_i a e^{\frac{1}{2}d}}{\left[(1+a)^2 e^{\frac{a}{2d}} - (1-a)^2 e^{-\frac{a}{2d}}\right]} \qquad 2.10$$

where:

$$a = (1 + 4k_{B(T)} t^* d)^{0.5} \qquad 2.11$$

a = coefficient accounting for the effects of d

$$d = (L/W)/[-0.261 + 0.254L/W + 1.014(L/W)^2] \qquad 2.12$$

d = dispersion number

$$k_B(T) = 1.1(1.07)^{T-20} \qquad 2.13$$

B_e, B_i, t^* are the same for the first model approach as expressed in section 2.5.3.1

Equation 2.6 is recommended for all types of ponds, single or in series which exhibit first-order kinetics and non-ideal mixing conditions under any sort of inlet and outlet arrangements. The equation is valid for ponds in which reactions are occurring uniformly throughout the pond depth at an overall rate coefficient k (Pena et al, 2000). The empirical formulae for the dispersion number d and the rate constant for FC removal k_T were developed in a tracer study of 24 separate pond systems in Peru (Buchauer, 2007).

2.3.5 Maturation Pond

Maturation ponds are shallow (1.0-1.5 m) and show less vertical stratification, and their entire volume is well oxygenated throughout the day. Their algal population is much more diverse than that of facultative ponds. Thus, the algal diversity increases from pond to pond along the series. The main removal mechanisms especially of pathogens and faecal coliform are ruled by algal activity in synergy with photo-oxidation. Maturation ponds are classically designed for excreted pathogen removal if the practice of unrestricted crop irrigation is required (Banda, 2007). In maturation ponds, faecal indicator bacteria and pathogenic bacteria are removed mainly due to starvation and hostile environment. Although models of pathogen removal are based on faecal coliform, which is not itself a pathogen, it has been established that faecal coliform is a suitable indicator of bacterial and viral pathogens in wastewater (Feachem et al., 1983). The mechanisms of faecal coliform removal in waste stabilization ponds are very close to that of faecal viral and bacterial pathogens (Curtis, 1990).

Detention time is the key factor in these ponds. Some other factors such as temperature, UV irradiation and oxidation also play their part (Arthur, 1983). These ponds receive the effluent from a facultative pond and its size and number depend on the required bacteriological quality of the final effluent. As indicated in Figure 2.2 above, the addition of maturation pond may be optional depending on the treatment objectives desired. On the other hand, maturation ponds only achieve a small removal of BOD_5, but their contribution to nitrogen and phosphorus removal is more significant. Researchers have suggested that the dispersed hydraulic flow pattern is the most practical flow pattern that can be achieved in operational maturation ponds. It is proposed that the first-order rate constant of the faecal coliform removal should be combined with the dispersed hydraulic flow pattern model (Banda, 2007). This is due to the fact that the Marais' equation has received strong criticisms from various researchers as being unrealistic and unsafe because the equation is based on the ideal complete mix hydraulic flow regime that is not realized in operational maturation ponds (Thirumurthi, 1974, 1969; Arceivala, 1983; Polprasert and Bhattaria, 1985; Nameche and Vasel, 1998; Sperling, 1999). However, Pearson *et al.* (1995, 1996) and Mara (2004) argued that Marais' (1974) model could be safely used to design and evaluate the treatment performance of waste stabilization ponds that are optimally loaded.

Buchauer (2007) revealed that faecal removal in WSP is a combination of different effects, which depend on the type of WSP, i.e. anaerobic, facultative or maturation and additional factors such as wastewater temperature, pond depth, percentage of water surface covered by algae, duckweed or similar organisms, positioning of inlets and outlets, wind direction, etc. - all of which prevail and interact to different degrees under different circumstances. The study compared the continuous stirred reactor (CSTR) model approach of Mara et al. (2001) and the dispersed flow model of Oakley (1997) for prediction of faecal coliform removal and it was concluded that on the basis of theoretical and practical analysis, under most conditions usually found in practice, the DF model can be expected to predict unrealistically low FC removal. The more the flow conditions in the pond(s) approach plug flow and the higher the retention time per pond, the more likely the DF model will predict too optimistic effluent FC removal, which does not match reality.

Buchauer (2007) asserts that if the DF model should become a more reliable tool, calculation of its underlying parameters, particularly of the rate constant k and the dispersion number d, will require more research because it appears that the present state of the art to calculate these parameters is not yet satisfactory. Consequently, based on usually applied model parameters, for the time being it appears safer to use the CSTR model than the DF model for FC removal prediction in WSP systems.

Lloyd et al. (2003) examined the impact of four sequential maturation pond interventions on the removal of thermo tolerant "faecal" coliform bacteria at a full scale WSP system in tropical Colombia. Each intervention was designed to increase hydraulic retention time and was followed by continuous physico-chemical logging and meteorological monitoring, and simultaneous tracer studies to define hydraulic retention time, flow paths and dispersion. Inlet and outlet monitoring showed that, primarily due to hydraulic short-circuiting; the open maturation pond only achieved a 90% reduction in thermo tolerant “faecal” coliform. By contrast, an in-pond batch decay rate study for thermo tolerant faecal coliform showed that a 1 log (90%) reduction was achieved every 24 hours for 4 days at 26°C, so that the maximum theoretical efficiency would be a 2.6 log reduction (99.7%) if hydraulic efficiency was perfect for plug flow. The results have fundamental implications for improving WSP efficiency, for meeting re-use guidelines, for savings in land area and improvement of design of WSPs. They also highlight short-comings in the indiscriminate use of the Marais design equation for faecal coliform removal.

As reported by Mara et al (1992), a total nitrogen removal of 80% in all waste stabilization pond systems, which corresponds to 95% ammonia removal, takes place in the maturation pond. It should be emphasized that most ammonia and nitrogen are removed in maturation ponds. However, the total phosphorus removal in WSP systems is low, usually less than 50% (Mara and Pearson, 1986; Mara et al 1992). The minimum acceptable value of t_m is 3days, below which the danger of hydraulic short-circuiting becomes too great. t_m is the detention time in maturation pond. The value for t_m should not be higher than that of t_f (the detention time in facultative pond) and the surface BOD loading on the first maturation pond should not exceed the surface BOD loading on the facultative pond (Mara et al., 1992, 2001).

2.4 Waste Stabilization Ponds in Some Selected Institutions in Nigeria

Waste stabilization ponds (WSP) are utilized in all climatic zones of the world to treat domestic and industrial wastewater and their use seems to be increasing in all continents. In developing countries like Nigeria, and especially in the tropical and equatorial regions, wastewater treatment by WSP has been considered an ideal way of using natural processes to improve wastewater effluents. WSP have been installed in some of the institutions in Nigeria, namely: Ahmadu Bello University, Zaria; Obafemi Awolowo University, Ile-Ife and University of Nigeria, Nsukka to mention a few. WSP have also been installed in some of the industrial and housing estates for domestic wastewater treatment while septic tanks are found in almost every household and offices. This section details a review of the WSP in some selected institutions. A World Bank Report (Shuval et al. 1986) endorsed the concept of stabilization pond as the most suitable wastewater treatment system.

Table 2.3 shows the typical dimensions and configurations of some waste stabilization pond in Nigeria. From preliminary investigation, the design population for most of the ponds has long been exceeded which results in serious overloading problems (Agunwamba, 1994 and 2001; Oke and Akindahunsi, 2005). Biological treatments of domestic and industrial wastewaters have increased tremendously, because they have been found to be appropriate for most developing nations, where land and labour are still relatively cheap and the climate favour natural degradation of organic matters (Oke, et al, 2009). Consequently, biological treatment of these wastewaters requires a combined process of carbon and nitrogen removal (Oke, et al, 2006, 2009). The high occurrence of sunlight in Nigeria and lack of adequate facilities for maintenance of complex mechanical systems, make WSP very suitable for the Nigerian environment.

Agunwamba (1993) expressed that, unlike in the developed countries where properly structured maintenance and monitoring programs for WSP system are practiced; ponds in Nigeria are neither monitored nor properly maintained. Many of the WSP systems in use in Nigeria are in poor state and some virtually dried up and almost covered by bush. The sides of the ponds are eroded in many places, blockages of wastewater pipes, large fluctuations in the quantity and quality of the effluent and reduction in the strength of influent sewage - all due to inadequate and irregular water supply- are frequent. Decreased depth of pond due to deposition of sediments has encouraged the growth of nuisance vegetations. No facilities are provided for regular maintenance and the operational and maintenance staff strength is inadequate compared with what is standard (Agunwamba, 1993, 1994).

Table 2.3 Typical Dimensions and configurations of some Waste Stabilization Pond in Nigeria

Location	Type & No. of Ponds	Unit	Liquid depth (m)	Mid-depth Area (m^2)	L/B ratio Approx	Shape/ sequence	Pretreatment /Population served	Place of disposal /Reuse
Ahmadu Bello University, Zaria	2 Facultative ponds (F)	FA	1.372	10,496	3:1	Rectangular series	None / 25,000-30,000	Kubanni river / uncontrolled irrigation
		FB	1.372	10,496	3:1			
	4 Maturation ponds (M)	MA 1	0.915	1,996	2:1			
		MA 2	0.915	1,910	2:1			
		MB1	0.915	1,880	3:2			
		MB2	0.915	1,806	3:2			
University of Nigeria, Nsukka	2 Facultative ponds (F)	FA	1.200	3600	4:1	Series	Imhoff tank	Uncontrolled irrigation
		FB	1.200	2500	4:1			
Obafemi Awolowo University, Ile-Ife	2 Facultative ponds	FA	1.05	Surface area = 4500 m^2	Surface width = 30m	Trapezoidal	Design population = 7320	Uncontrolled irrigation
		FB	1.05	Surface area = 4500 m^2	Surface width = 30m			

Source: (Agunwamba, 2001)

Some researchers in Nigeria have called for WSP effluent reuse in irrigation. However, such reuse is burdened with many problems which will militate against its successful and safe use in Nigeria. Although the problems of public acceptance of effluent irrigation, public health risks, and environmental pollution have been discussed (Agunwamba, 1991; Agunwamba, 1992), little was said about the problems that stem from WSP systems. Three institutions were selected based on geographical location in Nigeria for the purpose of this study. They are: University of Nigeria, Nsukka, in the South-East, Obafemi Awolowo University, Ile-Ife, South-West and Ahmadu Bello University, Zaria, in the

North. Although the effluents from these ponds do not meet the WHO standard, they are still used for uncontrolled irrigation (Agunwamba, 2001; Ibrahim et al., 2005).

2.4.1 Waste stabilization pond in University of Nsukka, Nigeria.

The University of Nigeria, Nsukka is located at 06^0 52`N and $07^0$ 24`E and lies about 70 km north of Enugu, the capital of Enugu State. The mean annual values of rainfall and humidity are 1678 mm and 75.5% respectively. The climate is characterized by high temperature (an average of about 27^0C) and also intense sunshine most part of the year. The topography is located on a plateau and escarpment region with ground elevations ranging from approximately 280 m to 530 m above mean sea level (Agunwamba 2001).

The wastewater treatment facility is situated at the northeastern end of the university campus, 800 m from the junior staff quarters. The treatment plant was constructed and commissioned in April and November 1961 respectively while it started operation in January, 1962 for a projected population of 12,000. As unveiled by Agunwamba (2001), the construction was planned to take place in two phases. The first phase was to treat wastewater for a population of 6,000 to 8,000 over a period of ten years.

The plant consists of a screen (6 mm diameter steel bar racks set at 12 mm centre) which removes the coarse particles followed by two Imhoff tanks, each measuring about 6.667 m × 4.667 m × 10 m, where the organic matter undergoes sedimentation and digestion; four drying beds in which sludge from the Imhoff tank is dried; and two facultative waste stabilization ponds in series. The digestion compartment storage volume is 0.057m^3/capita while the period of digestion is 30 days at an average temperature of 27^0C (Agunwamba, 1994).

Agunwamba (1994) considered the need for expansion of the WSP. A capacity expansion model for the determination of the capacity of additional WSPs and when it should be constructed has been presented. An application of the model to wastewater treatment works at the university revealed that optimal period for expansion has been exceeded several years (19 years) back. Since 1970, the two facultative ponds at Nsukka have not been desludged. Banks have caved in several places. Uncontrolled scum and grasses intercept wastewater particles. Dead zones are developed which reduce the ponds effective area, causing short-circuiting, and consequently resulting in poorer effluent quality (Agunwamba, 1993).

Despite the poor maintenance of the ponds, the influent into the first pond and the effluents from the first and second ponds are used for uncontrolled irrigation. Though the effluent quality is very poor, nutrients required for plant growth are met by wastewater. The high nutrient content is advantageous because the cost of fertilizer is unaffordable to the urban poor. Wastewater irrigation has been a means of livelihood for the urban poor from communities close to the University of Nigeria, Nsukka and reuse also helps the farmers to conserve scarce water resources. The reuse of the university WSP effluent in irrigation of crops, especially vegetables, has often raised public outcry. The disposal is aggravated by creation of odor and mosquito nuisance, destruction of livestock and the endemic nature of typhoid fever and diarrhea in Nsukka, while the ponds have degenerated over the years due to poor maintenance (Agunwamba, 2001). There has been fear that the poorly maintained WSPs and the reuse practices are contributing to environmental degradation and health hazards.

2.4.2 Waste stabilization pond in Obafemi Awolowo University, Ile-Ife, Nigeria.

The Obafemi Awolowo University (OAU) primary oxidation pond has served as an experimental unit upon which extensive BOD, suspended solids, and bacteriological analyses have been made. Both the temperature and pH ranges for the pond liquid are comparable with those of other institutions (Adewumi, 1989). Thus, the OAU primary oxidation pond wastewater samples can be used to determine rational design parameters that may be useful for residential institutions in the tropics, especially in developing countries.

The pond is located on the southern boundary of the university along old Ede road. The pond receives all the wastewater by gravity flow from the whole university community except the staff quarters. The contributing population includes students, staff and visitors to the university community. The receiving stream for the primary oxidation pond effluent is a 5th order tributary of the Opa River at the southern boundary of the university property. The basin for this stream is enclosed within the divide made of ridge tops. There are two ponds designed to operate in parallel either of which can be used at any time. The site is about forty minutes walking distance from the environmental health laboratory where all analyses are performed.

Four months, two each to represent the dry and the rainy season and the temperature range between 25^0C and 27^0C was analyzed. The hot dry season is from November to March, and the more humid rainy season is from April to October. The average effluent FC density showed that there was poor FC bacterial removal in the ponds. The average effluent bacterial density is more than seven times the maximum permissible FC density (i.e. not more than 5000 cells/100ml) stipulated in standard methods (Adewumi, 1989).

Having found that the oxidation ponds system cannot meet the wastewater treatment needs of the institution due to short retention period of few hours instead of thirty days, overpopulation and under-sized pond system, the design of an efficient waste stabilization ponds system became necessary. A preliminary design from which the final treatment system may evolve was proposed (Adewumi, 1989).

The design parameters proposed by Adewumi (1989) for residential educational institutions in tropical regions were as follows:

The average influent $BOD_{5,\,20}$ = 450 mg/l

The de-oxygenation rate, $k_{20} = 0.12\ d^{-1}$

The influent FC density, $N_{i,\,20} = 2.4 \times 10^7$ cells / 100ml

The first order FC removal rate, $k_{b,\,20} = 2.2\ d^{-1}$

The modal water temperature for the coldest month = 25^0C

Based on the specification presented by Adewumi (1989), a waste stabilization pond system was designed for OAU community using the parameters listed above. As much as it is important for the university to consider the preliminary design for approval and construction, there is the need for further work on the design of a full scale treatment plant from pilot plant. The methods of faecal coliform removal rate determination and Design of experiment (DOE) as an aid in analysis requires further work for wider acceptance.

2.4.3 Waste stabilization pond in Ahmadu Bello University, Zaria, Nigeria.

The WSP is located within latitudes 11° 7', 11° 12' N and longitudes 07° 41' E. The entire area is highly influenced by the presence of the University. The population of the university has increased from 20,000 in 1975 to about 100,000 in 1991 (1991 census) and even higher now. It is characterized by a tropical climate with two main reasons; a rainy season of about 210 days (May to October) and a dry/harmattan season (November to April). The monthly mean temperature records show a range from 13.8°C to 36.7°C and an annual rainfall of 1092.8 mm (Agbogu, et al., 2006).

Oke, et al (2006) assessed the physical and engineering properties of the ponds in Ahmadu Bello University (ABU), Zaria as shown in Tables 2.4. Table 2.5 summarizes the assessment of waste stabilization pond at ABU, Zaria while Table 2.6 shows the classification standard for wastewater based on composition (Metcalf and Eddy, 1991). The authors aim was to evaluate the efficiency of ponds at the institution. The pond consists of facultative and maturation ponds in series. Influent and effluent wastewater qualities were monitored from the ponds for a period of a year. Results of the study revealed that the flow rates of both influent and final sewage effluents are $620m^3$/day. The strong raw sewage has a BOD of greater than 400 mg/l and the ponds have BOD removal efficiency in the range of 93.2-95%, suspended solids removal of 55.0-76.1% and faecal coliform reduction of 99.1- 99.6% with average BOD reduction of 93.6%.

The average faecal coliform removal efficiency is 99.3% and average reduction in suspended solids by the ponds is 66.1%. The ammonia and phosphate concentrations of the raw influent were reduced on an average of 87.58 and 80.8% respectively by the ponds and overall average COD reduction was 96.4%. It was concluded that under tropical conditions the waste stabilization ponds are more suitable and appropriate compared to conventional treatment systems such as trickling filters and activated sludge, because of the ease of operation and maintenance and level of treatment efficiencies the ponds are able to achieve.

The treated effluent from the WSP at ABU meets some of the environmental and health criteria set by FEPA (1991). The raw sewage has BOD concentrations of greater than 400mg/l which can be classified as strong domestic wastewater. The mean overall BOD, COD, organic-nitrogen, and faecal coliform of the final effluent is low and satisfactory while the mean overall SS and ammonia of the final effluent is higher and unsatisfactory compared to the guideline value specified in Nigeria Federal Ministry of Environment guideline as stated in FEPA and many other standard environmental engineering literature (Viessman and Hammer, 1993; Hodgson, 2000).

Hodgson (2000) made similar results of a biological treatment plant at Akuse (Ghana) in which ammonia concentration of the effluent from facultative ponds ranged from 26 to 35 mg/l with a mean of 32.50 mg/l, 65% BOD reduction, 99.99% removal faecal count, 46% reduction of suspended solids, 92% and 94% of ammonia and phosphate removal respectively.

Table 2.4 Summary of engineering properties and physical assessment at ABU, Zaria.

Ponds	Liquid depth (m)	Volume of the pond (m^3)	Surface area (m^2)	Hydraulic loading ($m^3/m^2.d$)	BOD loading (kg/ha.d)	Retention time (d)	Length: breadth ratio
Pond A	1.37	14400	10500	15.34	0.75	23	3:1
Pond B	0.92	1700	1850	10.2	4.59	6	1.5:1
Pond C	0.92	1700	1850	10.2	4.59	6	1.5:1

Source: (Oke, et al. 2006)

Table 2.5 Summary of the assessment at ABU, Zaria.

Parameters	Raw influent		Facultative pond		Maturation pond	
	Concentration (mg/l)		Concentration (mg/l)		Concentration (mg/l)	
	Range	Mean	Range	Mean	Range	Mean
BOD(mg/l	450 - 650	609	120 -150	139.5	25 - 45	37.5
COD	880 - 1850	1120.6	235 – 430	256.8	60 – 110	75.8
Organic-Nitrogen	12 – 21	16.5	10 – 16	12.5	7 – 9.0	8.1
Phosphorus	12 – 28	22.6	6 – 11	7.8	2 – 8.2	4.6
Ammonia	38 – 65	62.4	26 – 35	32.5	5 – 10	6.2
Faecal count	1.0×10^5 – 4.0×10^5	3.0×10^5	1.0×10^4 – 7.0×10^4	3.0×10^4	2.0×10^2 – 4.0×10^2	3.0×10^2
Suspended Solid	300 – 1600	800	250 – 600	378.6	100 – 180	122.2

Source: (Oke, et al. 2006)

Table 2.6 Classification of wastewater based on composition

Parameter	Strong	Medium	Weak
BOD (mg/l)	400	220	110
COD (mg/l)	1000	500	250
SS (mg/l)	350	220	100
Phosphate as phosphorus (mg/l)	15	8	4
Organic-nitrogen (mg/l)	35	15	8
Ammonia (mg/l)	50	25	12

Source: (Metcalf and Eddy, 1991)

2.5. Residence time-models in waste stabilization ponds

The inadequacy and inconsistency of pollutant removal is threatening the continued use WSP. Many studies have identified hydrodynamic problems as the main cause of poor performance (Polprasert and Bhattarai, 1985). Hydrodynamics is one of the most important features to be considered at the design stage and subsequent operation of a reactor (Pena et al, 2003). However, very often these crucial factors tend to be overlooked in current design procedures. The performance of wastewater stabilization ponds is strongly dependent on the time the wastewater spends in the pond environment, which is a direct function of the hydraulic regime of the pond (Shilton and Bailey, 2006). The residence-time of wastewater in waste stabilization ponds is characterized by (i) plug flow, (ii) completely mixed flow, (iii) dispersed (arbitrary) flow, (iv) Tanks-In-Series and (v) Gamma-Function Extension to Tanks-In-Series.

These all depend on the assumptions proposed by the designer. These hydraulic flow patterns are used for designing the residence time in waste stabilization ponds and are integrated with biochemical process of the first-order kinetic reaction to simulate the decay of faecal coliform and BOD_5 in waste stabilization ponds. It is interesting to note that the completely mixed flow pattern is mostly used to design and assess the treatment performance of waste stabilization ponds (Mara, 2004).

2.5.1 Plug flow pattern

The concept of plug flow assumes that mixing occurs in the flow direction caused by velocity gradients and that lateral or radial mixing is negligible. The flow pattern is characterized by orderly flow of fluid elements with no elements of fluid overtaking or mixing with other elements ahead or behind along the flow paths. There is no mixing or diffusion as the wastewater moves through the pond. Environmental systems where plug flow models work best include long narrow channels, packed beds, fluidized beds, and any system where a majority of the dispersion occurs in one dimension (Levenspiel, 1999; Clark, 1996; Ducoste et al., 2001; Shilton and Harrison, 2003a). It was suggested by Levenspiel (1972, 1999) that the necessary condition for plug flow development was for the residence time in the reactor to be same for all fluid elements. Plug flow hydraulic regime was considered as the rational basis of designing efficient waste stabilization ponds (Thirumurthi, 1969). This design approach is the most efficient as it ensures that wastewater pollutants attain the theoretical hydraulic retention time.

Plug flow model tries to eliminate the hydraulic short-circuiting and stagnation regions formation that are inherent in many waste stabilization ponds. Thirumurthi (1969, 1974) strongly recommended that waste stabilization ponds be designed based on a plug flow model in order to maximize their hydraulic performance and treatment efficiency. Plug flow pattern can be achieved by fitting a large number of conventional baffles in a waste stabilization pond such that the width of the baffle openings and baffle compartments are the same. However, there is a performance risk that BOD overloading may be initiated in the first baffle compartment because the installation of baffles could concentrate the influent BOD_5 in the baffle compartments (Banda, 2007). Reed et al. (1988) used the plug hydraulic flow model to design primary facultative ponds and their model is presented in equation 2.14 as

$$\frac{L_e}{L_i} = e^{-k_{BOD\ P}\theta_f} \qquad 2.14$$

where:

L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

$k_{BOD\ P}$ = plug flow first-order rate constant for BOD removal (day^{-1})

$k_{BOD\ P}$ is related to temperature as follows

$$k_{BOD\ P} = k_{BOD\ P20}(1.06)^{T-20} \qquad 2.15$$

$K_{BOD\ P\ 20}$ = first-order rate constant of BOD_5 removal at 20^0C (day^{-1})

T = mean temperature of the coldest month (^0C)

Reed et al. (1988) suggested that the $BOD_{P\ 20}$ value depended on the surface BOD loading rate and advised that, if the value of $BOD_{P\ 20}$ was not known, a value of $0.1day^{-1}$ could be confidently adopted. This model could be inappropriate in warm climate regions where higher organic BOD surface loading rate has been reported to be appropriate (Mara, 2002 and 2004). The plug hydraulic flow regime is considered as unrealistic because zero longitudinal mixing of wastewater flow in waste stabilization ponds is difficult to achieve (Thirumurthi, 1969; Thirumurthi, 1974; Marecos do Monte and Mara, 1987). Wehner and Wilhelm (1956) in (Banda, 2007) argued that plug flow conditions could only be achieved if the length of the wastewater travel in the pond was infinity. However, all facultative ponds have finite limited lengths. Infinite length of liquid travel cannot be attained in practice and the proposed plug flow model was indeed difficult to achieve in practice (Banda, 2007).

Research has been carried out with the use of baffles to increase the distance of the wastewater flow in facultative and maturation ponds (Pearson et al. 1995, 1996; Reed et al. 1988; Mangelson and Watters, 1972). These researchers suggested that baffles could initiate plug flow in waste stabilization ponds; surprisingly there is no design procedure that recommends the number, position and length of baffles that could form plug flow in waste stabilization ponds (Banda, 2007). Although the approximate plug flow model could be achieved by fitting a large number of conventional baffles in facultative ponds, some researchers (Shilton and Harrison, 2003a; Banda et al. 2006) have expressed concern of the possibility of BOD overloading in the first baffle compartment. This might reduce the treatment efficiency of the plug flow pond as the algae concentration could decrease with increased loading of ammonia and sulphur concentration (Pearson et al. 1987). This potential risk of BOD overloading in the first baffle compartment has not been researched in the operational facultative ponds. It is not surprising to note that researchers recommend the use of conventional baffles in maturation ponds where BOD removal is not the primary objective.

2.5.2 Completely mixed flow pattern

The completely mixed flow assumes the wastewater is instantaneously fully mixed upon entering the pond. Both the Plug flow and the completely mixed flow are the two theoretical extremes flow behavior and are referred to as an ideal flow (Shilton and Harrison, 2003a). By assuming these types of flow behavior when integrating the rate equation for first order kinetics, equations can be derived that allow calculation of the treatment efficiency achieved after a certain period of time (Shilton and Harrison, 2003a). Levenspiel (1972) describes complete mixing as a reactor in which the contents are well stirred and uniform throughout. The completely mixed reactor produces effluent quality that has similar composition as fluid elements within the reactor. Researchers have used the completely mixed flow pattern in simulating hydraulic flow patterns in waste stabilization ponds (Marais and Shaw, 1961; Marais, 1974; Mara, 1976, 2001, 2004). Marais (1974) used the completely mixed hydraulic flow pattern when deriving the model of faecal coliform removal in waste stabilization ponds and this model is currently used to design maturation ponds. The completely mixed model proposed by Marais and Shaw (1961) is given by the basic relationship shown in equation 2.16;

$$\frac{L_e}{L_i} = \left[\frac{1}{1 + k_{BOD_C}\theta_f}\right]^n \qquad 2.16$$

$$k_{BODc} = 0.3(1.05)^{T-20} \qquad 2.17$$

where:

L_e = effluent BOD_5 (mg/l)

L_i = Influent BOD_5 (mg/l)

k_{BOD_c} = completely mixed first-order rate constant for BOD removal (day^{-1})

Θ_f = mean hydraulic retention time in facultative pond (day)

n = number of ponds in series

T = mean temperature of the coldest month (^0C)

Equations 2.16 and 2.17 enable the mean hydraulic retention time to be calculated.

For pond design, some researchers have proposed the use of the plug flow equation while others have argued for the application of the completely mixed flow equation; partly because it is less efficient and therefore gives a more conservative design equation for completely mixed ponds in series (Shilton and Harrison, 2003a)

Levenspiel (1972) believed that completely mixed reactors could be achieved when the influent wastewater flow was in ideal steady state. In reality, waste stabilization ponds receive quasi-steady state influent flow due to the daily variation of water usage and ground water infiltration and these factors prevent the development of steady state flow in waste stabilization ponds. This convinced Thirumurthi (1974) to propose the dispersed hydraulic flow pattern in simulating the hydraulic flow pattern in facultative ponds for the realistic determination of the mean hydraulic retention time.

Marais and Shaw (1961) suggested that wind mixing and temperature difference were principal factors that initiated the completely mixed flow in waste stabilization ponds. In contrast, Shilton and Harrison (2003a) and Tchobanoglous et al. (2003) argued that wind mixing and temperature difference without mechanical mixers could not develop the completely mixed flow pattern in waste stabilization ponds. However, Mara (2004) argued that the first-order kinetic removal of pollutants (faecal coliform and BOD) in waste stabilization ponds was well represented by the completely mixed model. This argument was supported by research findings of Pearson et al. (1995, 1996) who compared the accuracy of Marais' (1974) equation with the observed first-order rate constant removal of faecal coliform in facultative and maturation ponds. They observed that Marais' (1974) equation was sufficiently accurate to predict the observed effluent faecal coliform numbers in a series of waste stabilization ponds that were optimally loaded.

2.5.3 Dispersed hydraulic flow regime

An alternative to using the ideal flow equation is to use the Wehner-Wilhelm equation. This equation is called the non-ideal which is somewhere between the two extremes of plug flow and completely mixed flow and it incorporates the dispersion number (Shilton and Harrison, 2003a). The dispersion number is a function of all the physical influences that affects fluid flow within the pond. Mara (2004) argues that the ideal steady state of complete mix and plug flow pattern are difficult to achieve in practice. Levenspiel (1972) observed that real reactors never fully follow an ideal steady state flow regime. It was noted that deviations from the ideal flow regime are quite considerable. These deviations from the ideal steady-state are caused by channeling of wastewater from the inlet to outlet, recycling of wastewater and creation of stagnation regions that are inherent in all reactors including waste stabilization ponds.

Thirumurthi (1969) recommended that waste stabilization ponds be designed as dispersed flow reactors because waste stabilization ponds are neither plug flow nor completely mix reactors. He proposed the use of the first-order equation of Wehner and Wilhelm (1956) when designing facultative ponds. The proposed dispersed hydraulic flow model is presented in equations 2.18 – 2.20 as follows:

$$\frac{L_e}{L_i} = \frac{4ae^{\frac{1}{2d_f}}}{(1+a)^2 e^{\frac{a}{2d_f}} - (1-a)^2 e^{\frac{-a}{2d_f}}} \qquad 2.18$$

$$a = \sqrt{(1 + 4k_{BOD_D}\theta_f d_f)} \qquad 2.19$$

$$k_{BOD_D} = k_{BOD_{D20}}(1.09)^{T-20} \qquad 2.20$$

The degree of inter-packet mixing that takes place is expressed in terms of a dimensionless “dispersion number” defined as

$$d_f = \frac{D}{Vl} \qquad 2.21$$

where:

L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

k_{BOD_D} = dispersed flow first-order rate constant for BOD removal at T temperature (day^{-1})

$k_{BOD_{D20}}$ = dispersed flow first-order rate constant for BOD removal at 20 ^{0}C temperature (day^{-1})

θ_f = mean hydraulic retention time in facultative pond (days)

d_f = dispersion number

D = coefficient of longitudinal dispersion (m^2/h)

V = mean velocity (m/h)

l = mean path length of a typical particle in the pond (m)

T = minimum pond temperature in the coldest month (^{0}C)

Thirumurthi developed a chart to facilitate the use of the complicated equation 2.17 where $_{BOD_D}$ ($k \times \theta f$) is plotted against the percentage of the BOD remaining in the effluent for various dispersion numbers, *df* varying from zero for a plug flow to infinity for a complete mixed reactor. The observed dispersion numbers in waste stabilization ponds range from 0.1 to 4 with most values not exceeding 1.0 (Banda, 2007). The difficulty which is encountered in designing facultative ponds using the dispersed hydraulic flow model lies in the fact that at design stage, the value of dispersion number (d_f) and the first-order rate constant for BOD removal ($k_{BOD\ D}$) are not known.

Banda (2007) recognized the work done by Thirumurthi who proposed that values of k BOD_D should be developed based on various environmental conditions that are known to be toxic to the pond ecology. This cumbersome and expensive laboratory work is such that the resulting value of k BOD_D could not be determined with high level of accuracy. Dispersion number has been suggested to be obtained from tracer experiments in existing waste stabilization ponds. Interestingly, at the design stage of new waste stabilization ponds, dispersion number may not be determined since there could be no existing waste stabilization ponds with similar BOD loading conditions and geographical location.

Recently, Sperling (2002) compared four dispersion number models proposed by Polprasert and Bhattarai (1985) and Agunwamba et al. (1992). Sperling used Monte Carlo design simulations to predict the variation of dispersion number of four models. It was found that the dispersion number models proposed by Polprasert and Bhattarai (1985) and Agunwamba et al. (1992) were not accurate in predicting the dispersion number. Dispersion number model developed by Sperling (2002) predicted accurately the dispersion number since it produced a narrow variation of dispersion number. Dispersion number models developed by Polprasert and Bhattarai (1985) and Agunwamba (1992) not only show a weakness by predicting a wide range of dispersion numbers, but they also require the designer to assume some design variables such as kinematic viscosity, shear velocity and flow velocity when using the equation. It is also considered that measurements of these design variables in existing waste stabilization ponds cannot be determined accurately. In addition, these design variables depend on other factors such as temperature and the influent momentum that vary significantly on a daily basis and this increases the inaccuracy of the prediction (Banda, 2007).

2.6 Wind effect and thermo-stratification on hydraulic flow regime

It is worth mentioning as Wood (1997) observed that wind effect is another physical factor that is not taken into account in the dispersed hydraulic flow regime. It is known that wind speed and its prevailing direction induces shear stress at the top surface of the pond and this affects the hydraulic flow pattern in waste stabilization ponds. The extent of the wastewater mixing, which is initiated by the wind velocity can significantly change dispersion numbers and this could diminish the treatment efficiency of waste stabilization ponds. Wind velocity has also been noticed to cause hydraulic short-circuiting in waste stabilization ponds with a large surface area and a large inlet pipe (Shilton and Harrison, 2003a).

Kilani and Ogunrombi (1984) and Muttamara and Puetpaiboon (1996, 1997) found that the effects of thermo-stratification on shallow laboratory-scale ponds were not significant enough to influence the hydraulic and the treatment performance of waste stabilization ponds. The facultative pond was assumed to have isothermal conditions, so there was no short-circuiting associated with thermal-stratification. It was argued that the effect of wind on the flow pattern of the wastewater flow was so small that the resulting flow pattern could be deemed to be sustained by the inlet momentum. With this significant inlet momentum, the wind effect was considered to be negligible in influencing the treatment performance of a laboratory-scale waste stabilization pond.

Although, it has been recommended to include wind speed and its direction in CFD model when designing and evaluating the hydraulic flow pattern in waste stabilization ponds (Shilton, 2001; Sweeney, 2004; Wood, 1997), wind effects are negligible when considering the laboratory scale ponds. This is due to the confined environment and also based on the depth of the laboratory scale pond, the effect of thermo-stratification is not pronounced. One should also realize that the developed model cannot design accurately the temperature in waste stabilization ponds because it requires various input design parameters such as longitude, latitude, daily values of cloud cover fraction and relative humidity of a particular site that are difficult to obtain locally especially in developing countries where research resources and equipments are severely limited.

2.7 Tracer experiment

Tracer experiments are of great concern to wastewater treatment engineers and researchers because of the importance of flow analysis. The determination of hydraulic regimes and retention times in wastewater treatment units also depends largely on tracer studies. The studies have been used extensively to determine the transport, mixing and diffusion of harmful substances discharged to a water system or to a water body (Shilton and Harrison, 2003a; Bracho et al., 2006; Valero and Mara, 2009). The performance of a wastewater treatment unit depends mostly on adherence to hydraulic design and a phenomenon such as short circuiting deeply affects the facility's` overall effectiveness and efficiency.

Tracer experiments have been the most common method for undertaking research into pond hydraulics reported in literature and this has contributed significantly to the understanding of hydraulic flow patterns that exist in waste stabilization ponds (Shilton, 2001; Marecos do Monte and Mara, 1987; Mangelson and Watters, 1972; Wachniew and Rozanski, 2002; Wachniew et al. 2002). The experiment involves the addition of the tracer chemical at the pond inlet and its concentration is measured over time at the pond outlet.
Levenspiel (1972) in Banda (2007) expressed that the normalized residence time distribution curve obtained from tracer experiment in WSP is used in determining the mean hydraulic retention time.

Despite the advantage of tracer experiments in calculating the mean hydraulic retention time in field ponds, the technique cannot predict the retention time of wastewater in stagnation regions and the experiments cannot provide reliable experimental data of the effluent quality as the daily climatic conditions including wind play significant role in controlling the hydraulic flow pattern in the pond. As a result, it is difficult to assess the extent of the effective volume that is useful in the treatment of waste water in the waste stabilization pond. However, CFD can calculate the residence time at all points in the pond and this can help designers to investigate physical design interventions that can minimize the extent of the hydraulic short-circuiting and stagnation regions in waste stabilization ponds.

Using tracer experiment as a hydraulic design tool, one should realize that this approach cannot be used at the design stage of new waste stabilization ponds because the experiment is carried out in existing pond systems (Banda, 2007). Interestingly, CFD can be used at the design stage of waste stabilization ponds to simulate tracer experiments using the time-dependent equation of the scalar transport equation or the species transport equation (Langemyr, 2005). Baléo et al. (1991); Wood et al (1998); Salter (1999); Shilton (2001); Sweeney (2004); Banda (2007) all used different CFD tools to simulate the residence time distribution in waste stabilization pond models. It is admitted that this design approach allows the designer to make informed decisions regarding ways of improving the hydraulic performance of waste stabilizations ponds rather than relying on tracer experiments that are difficult to achieve successfully in a full-scale waste stabilization pond.

2.8 Effects of baffles on the performance of waste stabilization ponds

The performance of baffles in waste stabilization ponds (WSPs) has been evaluated and several researchers have found that the addition of baffles to WSPs could improve treatment efficiency (Muttamara and Puetpaiboon, 1996, 1997; Shilton and Harrison 2003a; Shilton and Mara 2005; Abbas et al. 2006). Baffles are walls used to channel or direct the flow of wastewater through the ponds. These baffles would provide additional submerged surface area to which microorganisms could attach themselves, thus increasing the concentration of microorganisms in ponds and, theoretically, the rate of organic stabilization as well.

Watters et al (1973) undertook an in-depth study on horizontal, vertical and longitudinal baffling. Three different lengths of baffles were tested: 50%, 70% and 90% pond width. Each of the lengths was tested in the ponds by using two, four, six and eight evenly spaced baffles. Short circuiting problems were found to occur in the 50% pond width when more baffles were used. Baffles of 70% width gave superior performance compared to the 50%

and 90% baffle width. Increasing the baffle width to 90% was found to give a lower hydraulic efficiency than was seen with the 70% width baffles. Watters et al (1973) believed that this was due to the narrow channel created at the end of the baffles that increased the velocity of the fluid in this area. Further investigation on vertical baffles with four experiments was performed: two with four baffles and the other two with six baffles. It was discovered that the four baffle cases proved to be more efficient than the six baffle cases. This was attributed to channeling effects. However, when the results were compared against the horizontal baffle experiments, it was found that the horizontal configuration was more efficient. The comparison of the horizontal baffling and the longitudinal baffling gave the same result.

The effect of baffles on the treatment and hydraulic efficiency of a laboratory-scale waste stabilization pond was investigated by Kilani and Ogunrombi (1984). The dimensions of the laboratory scale pond were 1m long, 0.5m wide and 0.1m deep. Baffles were fitted along the longitudinal axis of the pond. The unbaffled laboratory scale pond was used as a control of the experiment. Although the publication did not specify the length of baffles that were used in the pond, the study investigated three, six and nine-baffle ponds respectively. The treatment performance of the pond was assessed by observing the dispersion number, the effluent BOD and COD concentration in the pond effluent. It was noted that BOD removal increased with increasing number of baffles. However, there was no significant improvement in the COD removal when baffles were installed in the pond. The BOD removals in the three, six and nine baffle ponds were 81%, 86% and 89% respectively while that of COD was 84%, 84.2% and 84.2% respectively.

The results of the dispersion number in the three, six and nine baffle ponds were 0.126, 0.112 and 0.096 respectively indicating the initiation of plug flow pattern with increasing number of baffles. The baffled-laboratory ponds developed isothermal condition due to the shallow depth (0.1 m) that was used and this eliminated the effects of thermo-stratifications on the performance of the baffled ponds. The results indicated that ponds can be fitted with baffles to enhance the hydraulic performance without the risk of BOD overloading being initiated in the first baffle compartment.

Investigation into the treatment performance of complex ponds arrangement was carried out by Pearson et al. (1995, 1996). The ponds comprise of five-series waste stabilization ponds that employed different geometry, depth and the hydraulic retention times. Two anaerobic ponds were operated at volumetric loading of 187 g BOD per m^3 per day while the secondary facultative pond was operated at the surface organic loading of 217 kg BOD per ha per day. One of the tertiary maturation ponds was fitted with baffles such that the ratio of the effective length: breadth was greater than 100:1. It was observed that this baffled maturation pond was more efficient at faecal coliform removal than other tertiary maturation ponds.

Although the results of the baffled maturation pond were encouraging, conclusions could not be drawn to suggest that the treatment performance of baffled primary facultative ponds could be similar to that of baffled maturation ponds. It is known that the hostile environmental conditions that remove faecal coliform in maturation ponds are different from those found in facultative ponds. The investigation shows that baffles can improve significantly the treatment efficiency and hydraulic performance of facultative ponds. This can be one area of optimizing classic design methods in reducing the land area requirements for the construction of waste stabilization ponds.

Muttamara and Puetpaiboon (1996) evaluated the performance of baffles in waste stabilization pond comprising three laboratory-scale ponds with different number of baffles and one control units without baffle. The study was aimed at promoting WSP practice for wastewater treatment in tropical countries by reducing the land area requirement through the use of baffles. The dimensions of the laboratory scale pond were 1.5 m long, 0.5m wide and 0.15 m deep and neglected thermo-stratification effects due to its shallow depth. It was revealed that the dispersion number decreased with increasing length and number of baffles, which indicates more plug flow conditions. The laboratory-scale pond was investigated with zero, two, four, and six baffle configurations corresponding to the biofilm surface area of 1.35, 2.15, 3.03 and 3.92 m^2 respectively.

The deviation of actual HRT from theoretical HRT was computed and the flow pattern suggested the existence of an optimum spacing of baffles in baffled waste stabilization pond units. The hydraulic efficiency and physicochemical parameters were used to assess the treatment performance of the baffled ponds. It was observed that the hydraulic efficiency in the pond increased with increasing number of baffles. TN, NH_3-N and COD removal was increasing with number of baffles in the BWSP units with its maximum removal efficiency at six baffles. Faecal coliform die-off was also increased with increasing number of baffles in the order of 2-4 log removal. It was concluded that the treatment performance of waste stabilization ponds can be increased significantly by installing baffles.

Shilton and Harrison (2003a, 2003b) and Shilton and Mara (2005) adopted the findings of Watters et al (1973) in respect of the use of 70% pond width baffles. A 2D-CFD model was used to assess the treatment performance of the baffled facultative pond. It was found that faecal coliform removal in the model increased with the number of baffles installed along the longitudinal axis of the pond. Faecal coliform removals of 4.22 - 5.92 log units were obtained in a primary facultative pond with 2 – 4 baffles. For these cases, the width of flow channel in baffle compartments was greater than the width of flow channel at the baffle openings. Simulation of ponds with a large number (ten or more) of conventional baffles was not undertaken because the anticipated high effluent quality cannot be economically justified.

However, research into baffled facultative ponds with a large number of 70% pond width baffles is very significant because performance assessments of the initiated plug flow pond can be evaluated against possible pond failure due to BOD overloading in the first baffle compartment. In addition, this configuration of baffled pond may form a width of flow channel in baffle compartments that is less than the width of flow channel at the baffle openings. In this situation stagnations can develop in the baffled pond due to a reduction of velocity magnitude at the baffle opening. This may reduce the effective pond volume and may reduce the expected pond performance.

Other researchers (Zanotelli et al., 2002; Sperling et al., 2002, 2003) have also observed that baffles improve the hydraulic and treatment efficiency of waste stabilization ponds. The hydraulic efficiency and physicochemical parameters were used to assess the treatment performance of the baffled ponds. It was observed that the hydraulic efficiency in the pond increased with increasing number of baffles. Banda (2007) identified that current design procedure for waste stabilization ponds are not modified to include the improvement in the treatment efficiency and hydraulic performance that is initiated when baffles of various configurations are fitted in the pond and this will be part of the investigations in this research. Limited studies have been performed using numerical models to help quantify and elucidate the WSP performance.

2.9 Computational Fluid Dynamics Approach to Waste Stabilization Ponds.

In order to understand the internal processes and interaction in waste stabilization ponds, the simulation of the hydrodynamics has become a tool worth studying (Abbas et al., 2006). Pond design involves several physical, hydrological, geometrical and dynamic variables to provide high hydrodynamic efficiency and maximum substrate utilization rates. Computational fluid dynamic modeling (CFD) allows the combination of these factors to predict the behavior of ponds by using different configurations. The simulation of hydrodynamic in bioreactors supported by modern computing technology is an important tool to gain an improved understanding of the process functioning and performance (Abbas et al., 2006, Shilton et al., 2008).

CFD modeling analyzes flow problems and makes quantitative predictions to simulate the performance of systems. Simulations of different flow conditions are run, providing data and visualizations of what will actually happen when these conditions occur. Ducoste et al. (2001) identified that CFD simulated tracer tests would have a distinct cost advantage over experimental tracer tests when quantifying the reactor hydraulic efficiency for multiple flow conditions. Knartz (2009) and Langemyr (2005) also expressed that traditional testing methods require modeling and physical testing, which are time consuming and have physical limitations. In addition, CFD can be used to assist in conceptualizing new designs, maximizing existing systems, and troubleshooting.

Banda (2007) expressed that CFD is a generic flow model that calculates velocity, temperature, pressure and scalar variables at all points in a reactor. The CFD equations are developed using conservation equations of mass, momentum and energy (Versteeg, 2007; Langemyr, 2005; Shilton, 2001; Versteeg and Malalasekera, 1995). The use of Computational Fluid Dynamics (CFD) software has three compelling benefits: insight which enables to virtually crawl inside the design, foresight which helps to predict what will happen under a given set of circumstance and efficiency which helps to design better and faster based on the foresight gained (Knartz, 2009).

The argument about whether 2D or 3D model can simulate appropriately the hydrodynamics of flow in ponds has been a contention by some researchers. The derivation of a 2-D depth integrated CFD equations is based on partial derivatives techniques (COMSOL, 2005). The finite volume method is used to integrate the CFD differential equations to facilitate the computation of non-linear equations.

The technique of simulating tracer experiment in CFD model of waste stabilization pond was initiated by Wood (1997). He used the time dependent scalar transport equation to calculate the tracer concentration at the pond outlet. A 2D model was used to replicate the tracer experiment that was carried out in the laboratory pond by (Mangelson and Watters, 1972). Wood *et al.* (1995, 1998) also demonstrated that CFD-based design of waste stabilization pond could assess precisely the improvement in the hydraulic and treatment performance of waste stabilization ponds that are fitted with baffles of various configurations.

FIDAP software was used in the CFD model and it was noted that the simulated residence time distribution curves did not replicate satisfactorily the residence time distribution curves observed by Mangelson and Watters (1972). The author decided to use a 3D model to improve the accuracy of the time dependent scalar transport equation by including the depth of the pond, inlet and outlet structure in the model. It was found that the 3D model replicated more satisfactorily the Mangelson and Watter`s residence time distribution curves than the 2D model. However, it must be stated here that the idea of integrating depth into the 2D model was not realized by the author. And this runs contrary to the advise that 2D model should not be used to model the hydraulic flow patterns in waste stabilization ponds because it could not represent precisely the pond depth, inlet and outlets pipes in the CFD model.

It is worth noting that in another experiment, Wood et al (1998) compared 2-D CFD models to experimental RTD from literature and found out that in one of the three geometries simulated, the 2D CFD model successfully predicted the experimental RTD. He submitted that the other two geometries were not well described due to the difficulty of representing the three dimensional experimental inlet in the 2D CFD model. The author noted that the use of the time-dependent scalar transport equation with a source term was the innovative way of modelling the pollutant removal in waste stabilization ponds. However, this was not carried out. It was suggested that the development of a sub-model of the source term function that represented the pollutant removal was more complex and difficult to validate. However, the use of the scalar transport equation with a source term function that represented the pollutant removal was a significant step in assessing realistically the performance of waste stabilization ponds. This could have been the best approach for comparing the treatment performance of baffled waste stabilization pond models rather than relying on residence time distribution curves. It could then be said that the authors advice that 2D model should not be used to model hydraulic flow patterns in WSP is not justified.

Vega et al (2003) used two-dimensional depth-integrated model MIKE 21 in a research to simulate hydrodynamic and advection-dispersion processes in a full-scale anaerobic pond (AP) located in southwest Colombia. A set of 12 configurations including sludge contents, inlet-outlet positioning, baffling and pond geometry were modeled. The authors results showed that a crosswise (diagonally opposite) inlet-outlet layout, a length-to-breadth ratio of 2:1, plus provision of two cross baffles at 1/3 L and 2/3 L were the most effective measures to improve overall AP hydrodynamics and dispersion patterns.

Abbas et al (2006) provided detailed governing dynamic equations to solving the 2D-depth integrated equations of fluid mass and momentum conservation of an incompressible fluid in two horizontal directions. The forms of the solved equations are:

$$h\frac{\partial u}{\partial t}+hu\frac{\partial u}{\partial x}+hv\frac{\partial u}{dy}-\frac{h}{\rho}\left(E_{xx}\frac{\partial^2 u}{\partial x^2}+E_{xy}\frac{\partial^2 u}{\partial y^2}\right)+gh\left(\frac{\partial a}{\partial x}+\frac{\partial h}{\partial x}\right)+\frac{gvn^2}{\left(1.486h^{1/6}\right)^2}+\left(u^2+v^2\right)^{1/2}-\xi va^2\cos\psi-2h\omega v\sin\phi=0 \quad 2.30$$

$$h\frac{\partial v}{\partial t}+hu\frac{\partial v}{\partial x}+hv\frac{\partial v}{dy}-\frac{h}{\rho}\left(E_{yx}\frac{\partial^2 v}{\partial x^2}+E_{yy}\frac{\partial^2 v}{\partial y^2}\right)+gh\left(\frac{\partial a}{\partial x}+\frac{\partial h}{\partial x}\right)+\frac{gvn^2}{\left(1.486h^{1/6}\right)^2}+\left(u^2+v^2\right)^{1/2}-\xi va^2\sin\psi-2h\omega v\sin\phi=0 \quad 2.31$$

$$\frac{\partial h}{\partial t}+h\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+u\frac{\partial h}{\partial x}+v\frac{\partial h}{dy}=0 \quad 2.32$$

where:

h is the water depth,

u and v are the velocities in the Cartesian directions, *x*, *y* and *t* are the Cartesian coordinates and time,

ρ = density of fluid,

E = the eddy viscosity coefficient,

xx = normal direction on x-axis surface;

yy = the normal direction on *y*-axis surface;

xy and *yx* are the shear direction on each surface),

g = acceleration due to gravity,

a = elevation of bottom,

n = Manning’s roughness n-value, 1.486 the conversion from SI (metric) to non-SI units,

ξ = empirical wind shear coefficient,

Va = wind speed,

ψ = wind direction,

ω = rate of earth’s angular rotation and

φ = local latitude.

Equations (2.30) – (2.32) are solved by the finite element method using the Galerkin method of weighted residuals. The solution is fully implicit and the set of simultaneous equations are solved by Newton–Raphson non-linear iteration. The computer code executes the solution by means of a front-type solver, which assembles a portion of the matrix and solves it before assembling the next portion of the matrix.

For determination of water quality parameters, the following governing equation solves the depth-integrated equations of the transport and mixing process. The form of the depth averaged transport equation is

$$h\left(\frac{\partial c}{\partial t}+u\frac{\partial c}{\partial x}+v\frac{\partial c}{\partial y}-\frac{\partial}{\partial x}Dx\frac{\partial c}{\partial x}-\frac{\partial}{\partial y}Dy\frac{\partial c}{\partial y}-\sigma+kc+\frac{R(c)}{h}\right)=0 \qquad 2.33$$

where:

h = water depth,

c = concentration of pollutant for a given constituent,

t = time,

u and v are the velocity in x direction and y-direction,

Dx and Dy are the turbulent mixing (dispersion) coefficient,

k = first order decay of pollutant, the source/sink of constituent

σ = source/ sink of constituent

R(c) = the rainfall/evaporation rate.

The equation is solved by the finite element method using Galerkin weighted residuals. The Spatial integration of the equations is performed by Gaussian techniques.

Some other researchers (Patankar, 1980; Versteeg and Malalasekera, 1995) in Banda (2007) also provided detailed derivations of CFD equations in three dimensions for the time dependent flow with compressible fluids. Equation 2.34 presents the mass conservation for fluid that exhibits unsteady state flow characteristics.

$$\frac{\partial \rho}{\partial t}+\frac{\partial(\rho u)}{\partial x}+\frac{\partial(\rho v)}{\partial y}+\frac{\partial(\rho w)}{\partial z}=0 \qquad 2.34$$

where:

ρ = density of fluid (kg/m3)

u = velocity in x direction (m/s)

v = velocity in y direction (m/s)

w = velocity in z direction (m/s)

δx, δy, δz = differential change in distance(m)

δt = differential change in time (s)

For an incompressible fluid in steady state conditions, the density ρ in equation above is constant and the equation becomes:

$$\frac{\partial (u)}{\partial x} + \frac{\partial (v)}{\partial y} + \frac{\partial (w)}{\partial z} = 0 \qquad 2.35$$

The second conservation equation that is used in CFD is the momentum equation. The momentum equation is developed based on the Newton's Second law of motion. Simplification of the momentum equation involves the use of the Navier-Stokes equation and is very useful for the application of the finite volume method. It was shown by Banda (2007) that three momentum equations are used to calculate the fluid velocity in the CFD model. In the X-direction, the momentum equation is written as:

$$\frac{\partial (\rho u)}{\partial t} + div\left(\rho u U\right) = -\frac{\partial p}{\partial x} + div(\mu\, grad\, u) + S_{mx} \qquad 2.36$$

where:

$$div(\rho u U) = \frac{\partial (\rho u u)}{\partial x} + \frac{\partial (\rho u v)}{\partial y} + \frac{\partial (\rho u w)}{\partial z} \qquad 2.36a$$

$$div(\mu\, grad\, u) = \frac{\partial}{\partial x}\left(\mu \frac{\partial u}{\partial x}\right) + \frac{\partial}{\partial y}\left(\mu \frac{\partial u}{\partial y}\right) + \frac{\partial}{\partial z}\left(\mu \frac{\partial u}{\partial z}\right) \qquad 2.36b$$

where:

S_{mx} = momentum source term (N/m^3)

μ = dynamic viscosity (kg/m-s)

ρ = density (kg/m^3)

p = pressure (N/m^2)

U = vector velocity (m/s) = [u,v,w]

Additional momentum equations that are similar to equation (2.36) are used to calculate the fluid velocity in the Y and Z directions in the CFD model.

Banda (2007) developed the source term functions that represent faecal coliform and BOD_5 removal in the scalar transport equation which he expressed as the most challenging task that the CFD modeler will encounter when using CFD as the design code. The scalar transport equation that was presented by Patankar (1980) and FLUENT manual (2003) were modified to enable simulation of the transport of pollutants in waste stabilization ponds. The scalar transport equation of the pollutant removal in 3D model is presented as

$$\frac{\partial(\rho\phi)}{\partial t} + div(\rho\phi U) = div(\Gamma\, grad\, \phi) + S_\phi \tag{2.37}$$

where:

$$div(\rho\phi U) = \frac{\partial(\rho\phi u)}{\partial x} + \frac{\partial(\rho\phi v)}{\partial y} + \frac{\partial(\rho\phi w)}{\partial z}$$

$$grad\, \phi = \frac{\partial\phi}{\partial x} + \frac{\partial\phi}{\partial y} + \frac{\partial\phi}{\partial z}$$

$$div(\Gamma\, grad\, \phi) = \frac{\partial}{\partial x}\left(\frac{\Gamma\partial\phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(\frac{\Gamma\partial\phi}{\partial y}\right) + \frac{\partial}{\partial z}\left(\frac{\Gamma\partial\phi}{\partial z}\right)$$

S_ϕ = A+Bϕ = Source term of ϕ (kg/m^3/s)

ϕ = pollutant concentration (Faecal coliform per 100ml or BOD_5)

Γ = coefficient of diffusivity (kg/m/s)

ρ = density (kg/m^3)

U = velocity vector (m/s) = [u, v, w]

To simplify the derivation of the source term function to represent faecal coliform or BOD_5 removal, the scalar transport equation presented in equation 2.37 was expressed in one-dimensional form as:

$$\frac{\partial(\rho\phi)}{\partial t} + \frac{\partial(\rho\phi U)}{\partial x} = \frac{\partial}{\partial x}\left(\frac{\Gamma\partial\phi}{\partial x}\right) + S_\phi \tag{2.38}$$

where:

U = velocity in x direction (m/s)

Using the concept of a steady state plug flow pond, the steady state flow that represents the faecal coliform removal can easily be developed (Banda, 2007).

In the plug flow model, fluid elements do not mix with each other. The wastewater flow is carried by convection only, therefore the diffusivity coefficient of the scalar transport equation is zero ($\Gamma = 0$).

Thus equation 2.38 was further simplified to.

$$\frac{\partial \left(\rho \phi \ u\right)}{\partial x} = S_{\phi} \qquad 2.39$$

Patankar (1980) and FLUENT (2003) defined the source term (S_{ϕ}) as a function that depends on a constant term (A) and a coefficient (B) of the dependent variable such that source term function was expressed as:

$$S_{\phi} = A + B\phi \qquad 2.40$$

This was changed to a decay term by changing the sign of the coefficient of the variable to a negative. The source term function became a decay term:

$$S_{\phi} = A - B\phi \qquad 2.41$$

Equation 2.41 then becomes

$$\frac{\partial \left(\rho \phi \ u\right)}{\partial x} = A - B\phi \qquad 2.42$$

The first order reactor model of the rate of decay of faecal coliform in waste stabilization pond is normally expressed as equation 2.43 below.

$$\frac{d\phi}{dt} = -k\phi \qquad 2.43$$

where:

ϕ = faecal coliform numbers per 100ml

k = first-order rate constant of faecal coliform removal (day^{-1}).

Further derivation of equation 2.43 gives

$$\phi = \phi_0 e^{-kt} \qquad 2.44$$

Equation 2.44 is the fundamental equation of the plug flow pond model and this suggests that the CFD will predict faecal coliform counts that are close to the plug flow pond solution. This gives an understanding that the use of the scalar transport equation with a source term function that represented the pollutant removal is a significant step in assessing the performance of waste stabilization ponds (Banda, 2007).

2.10 Laboratory scale ponds

The majority of hydraulic studies on waste stabilization ponds have been undertaken on full-scale, field ponds which have transient inflow-rates and large surface areas that are exposed to constantly changing wind and temperature conditions (Marcos and Mara, 1987; Moreno, 1990; Agunwamba, 1992; Fredrick and Lloyd, 1996; Pena, 2002; Adewumi et al., 2005). Because of climatic variation, the low velocity and long residence time systems are difficult to systemically study in the field. Field studies will, therefore, be only indicative of the hydraulic behavior resulting from the conditions that existed during the study period (Shilton, 2001, Shilton and Bailey 2006). Antonini et al., (1983) noted that given the numerous changes in operating and weather conditions that inevitably occur during an experimental run, the study of retention time distribution could only be effectively done on scale models studied under controlled conditions. The best approach that has been suggested is to undertake research on scale-model ponds operated under controlled conditions in a laboratory (Shilton and Bailey 2006).

There have been a number of researchers who have attempted to study pond hydraulics using laboratory models. However, there appears to have been a generally limited understanding and some confusion as to how these model ponds should be built. In the worst cases, some models have lacked even geometric similarity (Shilton, 2001). Several researchers such as Thirumurthi (1969), Antonini et al., (1983) and Agunwamba (1992) suggested the use of dimensional analysis in the design of these laboratory models in order to improve their ability to reliably represent full-scale systems. Two promising dimensional analysis approaches to the scaling of ponds are based on Froude number and the Reynolds number.

Shilton and Bailey (2006) piloted the research on applying image processing for drogue tracking in a laboratory-scale waste stabilization pond. The prototype (full-scale) pond that the model represented was 32.58 m in length, 21 m in width and 1.5 in depth. The model was placed within a confined, constant temperature room to minimize temperature changes and exposure to air currents. A length scale ratio *(SL)* of 1:12 was used which set the internal dimensions of the model at:

Length of model = 2.715 m

Width of model = 1.750 m

Depth of model = 0.125 m

It should be noted that the dimension of the depth does not represent the scale ratio of the prototype pond. This should not be so if truly the dimensional analysis strictly has to be followed. This has not given the exact scale factor for which the model was designed and it is not a true geometrical representation of the prototype pond.

The model's flow rates were set to maintain Froude number similarity with the prototype and had the following scaling factors: Flow in prototype is 498.8 m^3/d while that of the model is 3.463m^3/d. Different flow-rates, effect of baffles, different inlet and outlet positioning and different inlet designs were examined: 0.060 m diameter pipe directed along the horizontal axis of pond positioned at mid-depth; 0.120 m diameter pipe directed along the horizontal axis of pond positioned at mid-depth and 0.120 m diameter pipe directed vertically discharging towards the base of the pond, positioned 0.025 m below water surface.

To track the flow, small drogues were placed in the pond and swept around with the flow. To ensure that these drogues were representative of the fluid movement, testing of this technique was undertaken using a tracer dye. The application of the image processing technique allows continuous tracking of multiple drogues over a lengthy period of time and also allows for very clear observation of such flow behavior. This revealed that the use of scale-model ponds tested under controlled conditions in the laboratory is an important experimental method for such work. Shilton and Bailey (2006) stated that the technique of using image processing for drogue tracking in these ponds is an effective way of quantifying their hydraulic regime and can be considered as a valuable complement to traditional tracer studies. It is relatively cheap and effective compared to other techniques that are available for measuring flow velocities.

Abbass et al. (2006) used a two-dimensional CFD surface-water modelling system (SMS) on scaled WSP with various rectangular shape configurations to simulate the hydrodynamics and water quality. The model was run at steady state to examine the effect of the assumed rectangular shapes and dimensions with a constant area for values of various water depth, flow rate and HRT of wastewater. The result demonstrated an increased BOD removal efficiency from 16% at length to width ratio of 1:1 without baffles to 93% and 96% at length to width ratio of 4:1 with the inclusion of two and four baffles respectively. The authors' results also displayed a decreased DO effluent concentration from 0.5mg/l at length to width ratio 1:1 without baffles to 6mg/l with two transverse baffles and increased to 10mg/l when the length to width ratio was 4:1 with the inclusion of four transverse baffles at one-third and one-fifth of the baffle length respectively. The result showed that the length to width ratio 4:1 with two and four transverse baffles is most efficient in improving the overall water quality in the WSP hydrodynamics and BOD removal efficiency.

Banda (2007) used experimental data of the effluent fecal coliform concentration to validate the computational fluid dynamics (CFD) model in three pilot-scale ponds. Differences were noted between the predicted effluent fecal coliform concentration and the pilot-scale pond experimental results. For the effluent fecal coliform numbers, the difference between the CFD model and the experimental log removal in the pilot-scale pond increased with decreasing number of baffles used in the WSP (i.e., 18% for unbaffled pilot-scale pond, 19% for two-baffle pilot-scale pond, and 7 % for four-baffle pilot-scale pond) (Banda, 2007).

Shilton et al. (2008) maintained that the use of computation fluid dynamics (CFD) for waste stabilization pond design is becoming increasingly common but there is a large gap in the literature with regard to validating CFD pond models against experimental flow data. He assessed a CFD model against tracer studies undertaken on a full-sized field pond and then on a 1:5 scale model of the same pond operated under controlled conditions in the laboratory. While the CFD tracer simulation had some discrepancies with the field data, comparison to the laboratory model data was excellent. The issue is, therefore, not in the way the model solves the problem, but rather with how accurately the physical conditions are defined in the field.

2.11 Optimization of waste stabilization pond design

Generally, real world problems require the concurrent optimization of several and frequently challenging principles. The solution to such problems is usually computed by combining them into a single criterion to be optimized, according to some utility function. Fonseca and Fleming (1993 a, b) established that in many cases, however, the utility function is not well known prior to the optimization process. A number of previous studies have discussed the idea of optimizing the cost of treatment plant construction and operation of waste stabilization pond and concluded that using more baffles gives better hydraulic efficiency but with cost in mind it is necessary to better understand the effect of baffle number on treatment efficiency (Oke and Otun, 2001; Shilton and Harrison, 2003a; Bracho et al, 2006). It is important to estimate the economic costs and benefits of a range of selected configurations to improve effluent quality.

The range of options available for improving effluent quality is wide, especially in developing countries where large proportions of the population have access to limited basic facilities. It is expedient to favor intervention options that are at low cost and feasible which do not require heavy construction and maintenance cost because land availability and price are to be considered as a key factor for final decision on any configuration chosen.

The utilization of WSP system has been discussed in previous section to be limited by its large area requirements especially in the urban areas where land is scarce and costly. Studies show that pond depth could be increased beyond the generally acceptable value particularly in tropical countries where enough light is available (Agunwamba, 1991). Therefore, Ponds need better design in order to achieve an efficient land usage. With this understanding, it is pertinent to design for an economic size which is the least cost size of pond that gives the best treatment and at the lowest operational cost.

Capital cost function as a linear function have been used to consider expansion problems in the design of WSP and modeling technique has been identified for the analysis of wastewater treatment system capacity expansion problems (Agunwamba, 1994). The capital cost includes land cost, cost of inter-ponds connection works and the number of ponds. The objective function that comprises the net present value of the capital investment cost associated with construction and operation costs of a WSP was minimized. The model also determines the capacity of additional WSPs and when they should be built. Also, a graphical approach was applied by Agunwamba (1991) to cost minimization in WSP subject to area, cost, depth and efficiency constraints. Optimal solutions for both plug-flow and completely-mixed flow models were compared. The solution has the values of area, cost and depth at optimality.

Oke and Otun (2001) highlighted two different approaches to the economic sizing of WSP: The graphical and the parallel tangent solutions. The authors formulated the mathematical solution on the development of economic size formula for waste stabilization ponds. The mathematical derivation revealed that there is a linear relationship between economic size, depth, influent BOD, effluent BOD, discharge rate and the rate constant. The results of the authors' research show that economic size is inversely proportional to the square of velocity of flow for both plug and completely mixed flow. In addition, the relationship between the depth of the pond and the economic size is directly proportional to the square of the depth for both plug flow and completely mixed flow. Having derived these relationships, the authors suggested that an attempt of practically integrating the concepts is important. The result of such attempt will be useful in the design and operation of a treatment system.

However, with the unveiling of the authors approach to solving optimization problem in terms of cost, limited research have been carried out by using computer simulated tools in the developing countries. The use of such software and other robust computer optimization tool will help to ascertain not only good results but also saves time. The optimization technique searches for the best solution among the various different possibilities to achieve maximum pollutant reduction with minimum cost of construction. One primary focus of this research has been to identify a tool to solve the optimization problem that will incorporate a search technique in order to arrive at the objective function.

2.12 Summary of literature review

A review of pressure on water demand and the existing WSP in three higher institutions has been presented. The chapter has revealed that the classic and modern design procedures for waste stabilization ponds which are based on the completely mixed flow and plug flow cannot evaluate the improvement in the hydraulic and treatment performance of WSPs that are fitted with baffles of different configurations. It has also been demonstrated that CFD overcomes the limitations of current design procedures for waste stabilization ponds. Therefore, CFD can be used as a reactor model to simulate precisely the pollutant removal and investigate the potential hydraulic performance and improvement due to different baffle configurations, outlet positions, depth, and velocity distribution in waste stabilization ponds. This can be carried out by developing source term functions that modify the default scalar transport equation.

The source term functions that represent the pollutant removal, velocity distribution and residence time distribution can be developed into a form consistent with the source term function for the default transport equations in a CFD tool. This would help in understanding the influence of reactor design on the treatment performance of WSP. It has also been expounded that baffles can improve significantly the hydraulic and the treatment performance of waste stabilization ponds. This understanding can help designers to identify the physical design interventions that can provide detailed information regarding flow patterns and the treatment performance of waste stabilization ponds.

The area also worth researching is the investigation of the optimal baffle configuration that will give optimal performance of a WSP resulting in cost reduction without jeopardizing the treatment efficiency. These approaches use mathematical models that give a reliable image of the existing and optimized systems respectively. The available literature does not give guidance on the number and length of baffles that provide economic baffle configuration. Numerical experiments using CFD optimization model can be used to investigate the suggested problems and proffer useful solutions..

CHAPTER THREE

METHODOLOGY

3.1 Description of the study area.

This research work has been conducted to establish proper design guidelines for the construction of waste stabilization pond systems using base data from a residential tertiary institution in Nigeria. Operational performance of wastewater treatment in three universities in three regions of the country namely: North, South-East and South-West, were obtained. However, due to lack of secondary experimental data, a typical representative university was selected for establishing design parameters by considering the climatic conditions, population growth rate and socio-economic conditions. After collection of all the required data, the study led to a design procedure for the design and construction of a laboratory scale waste stabilization pond system.

The research consists of four main parts:

1. Take inventory of water supply and the usage in Covenant University,
2. Review and analysis of the current wastewater treatment system and proffer a suitable technological option. This translates to the design of a treatment system in order to propose a prototype treatment unit,
3. Design and construction of a laboratory-scale treatment unit to model the performance of the treatment plant,
4. Evaluate the performance of the model, simulate and optimize the treatment system to reduce cost without jeopardizing the treatment efficiency.

A typical representative community was selected for establishing WSP design parameters that consider the community's population growth and climatic conditions. Covenant University community, within Canaan Land in Ota town, is in close proximity to the city of Lagos, Nigeria and was selected in this study. Temperatures are high throughout the year, averaging from 25°C to 28°C (77° to 82°F). The institution has been witnessing an increasing population since its inception in 2002 with a current population above 9000 people and a daily water requirement that was estimated at 136L/C/day. Table 3.1 shows the population trend for staff and students since the inception of the university. Figure 3.1 shows the bar chart of the trend.

Table 3.1 Population Trend of staff and students in Covenant University since Inception.

Session	2002/2003	2003/2004	2004/2005	2005/2006	2006/2007	2007/2008
Students	1064	2908	4272	5727	6375	6819
Staff	128	266	409	522	574	722
Total	1192	3174	4681	6249	6949	7541

Source: Center for Systems and Information Services, Covenant University, Ota.

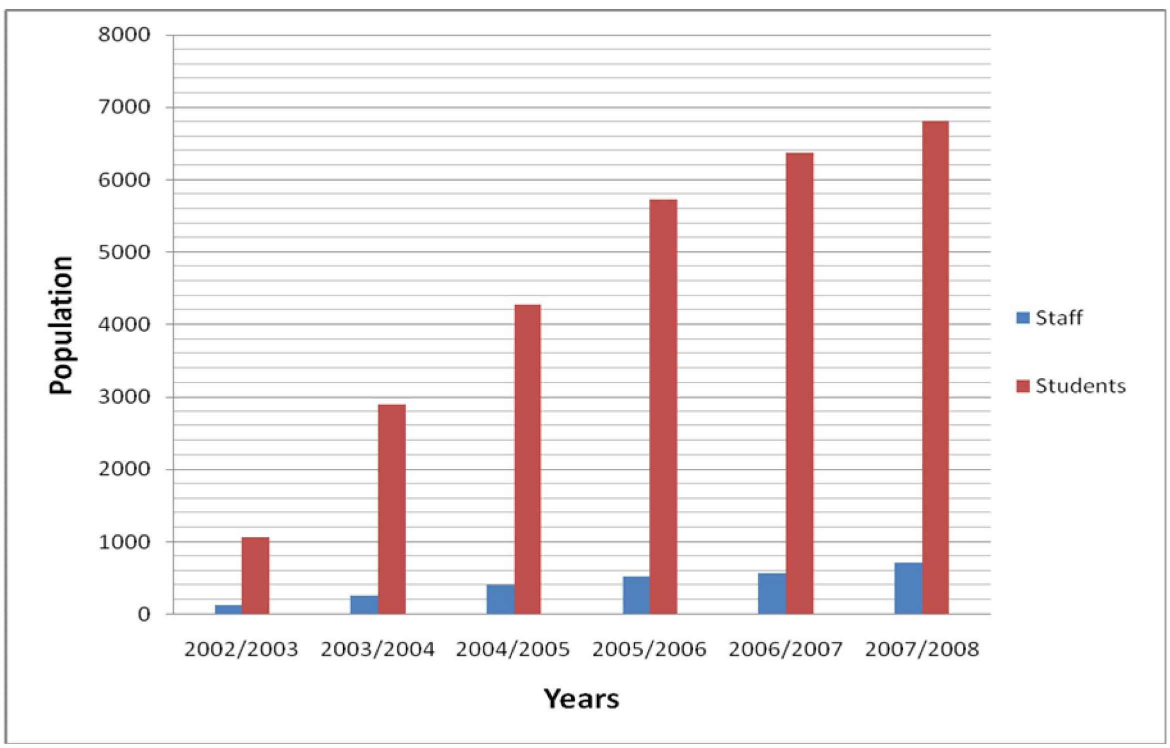

Figure 3.1 Barchart of staff and student population trend

Canaan Land has an expanse of 524 acres of land with an array of architectural masterpiece which consist of the Centre for Learning Resources (university library), college buildings, a 3,000 seat capacity student chapel, 22 duplexes with 48 chalets in the Professors' Village, 64 suites at the senior staff guest house, 64 three-bedroom flats in the senior staff quarters, 100 rooms in the university guest house, two Cafeterias, 96 two-Bedroom apartments and 24 one-Bedroom apartments in the Postgraduate residence halls. In addition to these, there are 10 blocks of student hostels, administrative offices, lecture halls, a gymnasium and four New Engineering workshops. The Institution covers one third of the total land mass, with substantial amounts of the treated wastewater from the existing treatment plant being discharged into water course without harnessing its usefulness. Though effluent from the existing treatment plant appears clean, its characteristics and composition suggest that better treatment approach should be explored.

Covenant University, with a restricted natural water source, is being supplied with freshwater from six groundwater boreholes that pump water into two surface reservoirs and two overhead reservoirs both at the staff quarters and the student hostels. The overhead and surface tanks at the staff quarters have a capacity of 340,000 and 500,000 liters respectively while those at the students' hostels have a capacity of 500,000 and 1.2 million liters, respectively. There has been high water demand for the growing population, especially for the irrigation of large expanse of lawns and horticultural plants, increasing construction work on sites and washing of cars by the staffs coupled with domestic uses. This has led to a significant constant refill of the water reservoirs (surface and overhead tanks) and this may put pressure on the aquifers in a short amount of time. In order to avoid abandonment of the boreholes and also to conserve the freshwater, quick measures have to be taken to prevent the boreholes from being burdened.

Moreover, Canaan land wastewater is being transferred to the Covenant University treatment plant, which eventually gives a large volume of wastewater discharged into the stream every day. Canaan land comprises Green Pastures Restaurant, The Mission lodge, Secretariat complex, The Faith Tabernacle, Kingdom Heritage School, Faith Academy School, Canaan Land Mass Transit Workshop (CLMT), Dominion Publishing House and

Hebron Waters. The university owes the communities around it the responsibility to enhance a clean and neat environment by discharging non-toxic effluents from the wastewater treatment plant which must meet the minimum regulatory standards and also harness the effluents for useful purposes.

3.2 Collection of data on water demand.

Questionnaires were administered to the entire university community in order to establish the approximate population of the institution. This involves the collection of data on house allocation to staff in various quarters and students in hostels. The existing reservoir sizes, daily pumping and supply rate, were used as estimates of water quantity. Time of usage for each loading was also collected from the School's Physical Planning and Development unit (PPDU). The data were useful in estimating the daily per capita usage of the available water resource.

Figure 3.2 shows a template used in calculating the water demand per person per day.

An estimate of how much water used, on a per capita basis was calculated by filling the form about home water-use activities.

Baths Taken: 2 baths	**Showers:** 2 showers	**Average shower length:** 10 minutes
Teeth Brushings: 2 brushings	**Hand/Face Washings:** 2 times	**Face/Leg shavings:** 1 time
Dishwasher Loads: No loads run	**Dishwashing by Hand:** 3 times	**Clothes Washing Loads:** 1 time
Toilet Flushes: 2 times	**Water drunk (8 oz.):** 8 glasses	

Submit your answers or Start over

Fig. 3.2 Template for calculating the per-capita water use. Source USGS (2005)

The total population of the community was estimated to be 9,114 people. The daily water requirement was found to be 136 L/C/day. Thus, the total quantity of water used by the university community was estimated as;

Qp = population x per capita used = 9114 X 136 = 1239,504 liters / day = 1239.504 m^3 / day.

3.3 Estimation of wastewater generated

The main type of wastewater identified in the institution is municipal wastewater

The waste is itemized as follows;

a. Foul waste which is from items such as basins, baths, showers and toilets from the student hostels, college buildings and the library.
b. Raw influent (sewage) which is the liquid waste from toilets, baths, showers, kitchens, sinks etc. from the staff residential quarters.
c. Sullage water from basins, baths and kitchens from the cafeterias and those brought to the treatment plant by tanker trucks from other parts of the Canaan land.

80% of the household water consumption has been reported as a suitable wastewater design flow value and is dependent on the per capita consumption (Mohammed, 2006). This consumption amount produces a wastewater flow rate of 991.603 m^3 / day.

3.4 Study of the existing wastewater treatment system

The wastewater (WW) from the university community and the faith tabernacle are carried in sewers to a central treatment plant behind the undergraduate halls of residence. The treatment plant outfall is situated on a cliff over a perennial stream that drains the campus and forms a tributary that discharges into River Atuara, a few kilometers from the Campus. Wastewater from septic tanks in isolated locations within the canaanland is also taken by tankers for discharge into either of two sedimentation tanks each with a surface area of 15.5m by 17.1m and 5m deep (Plate 3.1). These tanks function like anaerobic ponds within which the biochemical oxygen demand (BOD) and total solids are substantially reduced by sedimentation and anaerobic digestion before the partially treated effluent enters a diversion chamber. It is from this point that the wastes are fed into the hyacinth beds (Plate 3.2).

The reed beds consist of six units of concrete facultative aerobic tanks 1.2m deep and each partitioned into four cells with an internal surface area 5.70 m by 4.80 m with open sluice gates protecting the influx of wastewater into other cells at alternate ends of the partition walls (Plate 3.3). This is to ensure proper mixing of the wastewater as they flow through the tanks. The effective depth of each cell is about 0.9 m and has a volume of 23.16 m^3 with a free board of 0.30m. The final effluent discharges into an outfall that is about 8m long and empties into space at the edge of the cliff on which the whole treatment unit is constructed (Plate 3.4-3.5).

Plate 3.1 Shows tanker dislodging wastewater into the treatment chamber

Plate 3.2 The water hyacinth reed beds showing baffle arrangement at opposing edges.

Plate 3.3 The inlet compartment showing gate valve

Plate 3.4 The Outfall waterway leading into the valley below the cliff

Plate 3.5 Effluent discharging through the outfall into the thick vegetation valley.

3.5 Analysis of wastewater samples

Samples of the raw influent and treated effluent from the existing water hyacinth reed bed were collected and analyzed in the laboratory for its BOD_5, Faecal coliform, pH, temperature, COD, Suspended Solids, Total Solids, Nutrients and Heavy Metals. Variation of influent and effluent parameters (physical, chemical, bacteriological and physico-chemical characteristics) was determined. The critical pollutants (BOD_5, Faecal coliform) in the sample sources were selected and used in the design of the model treatment system. Grab samples from raw and settled wastewater were collected from influent and effluent points of the treatment plant and taking to the Central Research Laboratory at the Obafemi Awolowo University, Ile-Ife for analyses of BOD_5 and Faecal Coliform present. A BOD_5, fecal coliform, and pH analysis was performed on the raw influent and treated effluent from the existing water hyacinth reed bed at the university. The analysis showed that the average values of 64×10^6 FC/100 ml and 25×10^4 FC/100 ml for Faecal Coliform, 197 mg/l and 118 mg/l for BOD_5, and a pH of 7.58 and 7.41 were measured at the inlet and outlet, respectively. This prompted the need for the design of an adequate WSP system for the university that will take care of the future population.

3.6 Design of the laboratory-scale plant layout:

The wastewater from the existing plant was used as the raw wastewater for the laboratory experiment. Based on the results obtained from the measured samples and design parameters, a laboratory-scale pond was designed and constructed. A field scale prototype of a WSP was designed and scaled down to a laboratory-scale model using Froude number and dimensional analysis. The full scale and consequently laboratory-scale design utilize literature data in the development of the foot print size, baffle configuration and length (Mara 2001, 2004; Shilton and Harrison, 2003a; Hamzeh and Ponce, 2007). There are three ponds in series, namely: anaerobic, facultative and maturation ponds. The WSP system design was based on an expected population growth rate of 4.5% (Khowaja, 2000) over the next twenty years, which will amount to 22,000 people. The total water consumption for the design period was: 136L/C/day × 22,000persons = 2992 m^3/day. The wastewater flow rate based on the per capita demand is 2393.6 m^3/day.

The pond was designed and constructed by using galvanised sheet (Fig. 3.3). An elevated tank serves as a reservoir for the supply of the wastewater to the ponds in series at a constant rate. A timer was used for periodical feeding of wastewater to avoid hose blockage into the three treatment units constructed. Samples of the effluent from succesive pond were taken at intervals for laboratoy analysis. The inner walls of the galvanised material were painted with suitable aluminum paint in order to avoid rusting. Each pond has varying width (w), depth (d) and length (I) based on design.

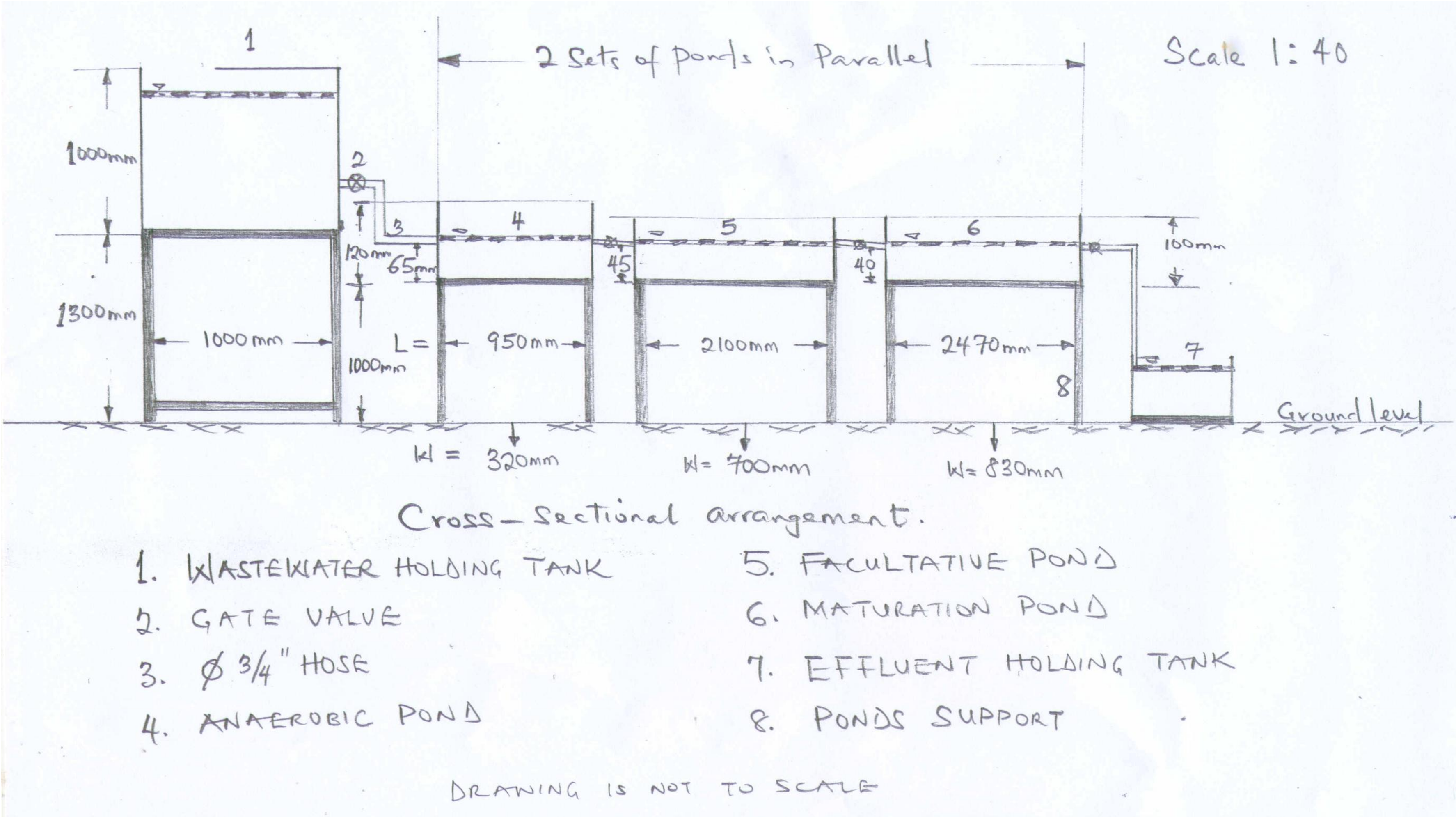

Fig. 3.3 A sketch of the laboratory-scale WSP and operating conditions.

3.6.1. Design Guidelines for the Covenant University, Ota.

In order to design for waste stabilization pond for the university, consideration was given to the population and its climatic condition. Factors considered include; temperature, population, wastewater generation and other factors that will be discussed in later sections.

3.6.1.1 Temperature (T)

The average monthly mean temperature of the institution selected for this study varies from 25^0C minimum and 28^0C maximum. Since, for the larger period, hot climate persists, a conservative value of 27^0C was chosen.

3.6.1.2 Population (P)

According to the questionnaire and data obtained from the CSIS of the University, the population of the university was estimated to be 9114 people. Since, waste stabilization pond system is usually designed for 20 years period (Khowaja, 2000); the expected population for the next twenty years with a growth rate of 4.5% will be 22,000 people. This is to cover for all the developments that have been proposed to take place in the future.

3.6.1.3 Wastewater generation (Q) and Design for 20 years period

From the analysis of questionnaire administered, the daily water requirement was found to be 136L/C/day. Therefore, the total water consumption for the design period was:

136L/C/day × 22,000persons = 2992000L/day = 2992 m^3/day

Since 80% of the water consumed is given as the wastewater flow, therefore, Q, which is the daily wastewater flow = 2393.6 m^3 / day.

3.6.1.4 BOD Contribution per capita per day (BOD)

The values of BOD, usually, vary between 30 and 70gm per person per day (Khowaja, 2000). For this particular study it is taken as: BOD = 50gm/capital/day based on the standard of living.

3.6.1.5 Total Organic Load (B)

This was calculated as B = Projected population × BOD

22,000 × 50 = 1100 kg/day

3.6.1.6 Total Influent BOD Concentration (Li)

The total influent BOD concentration (Li) was calculated from the equation below (Mara, 1987, 2001and 2004).

$$L_i = \frac{B}{Q} = \frac{1100}{2393.6} = 459.55 mg/l \quad \ldots\ldots 3.1$$

3.6.1.7 Volumetric organic loading (λv)

The design of first pond in the series, anaerobic pond is based on the volumetric organic loading, which normally ranges between 100 (at temperatures below 12^0C) and 400gm/m^3/day (at temperature above 25^0C). But for this particular study the following relationship was obtained with the help of the graph drawn by Mara (1987),

$$\lambda v = [\, 300(T-12)/18] + 100 \quad \ldots\ldots 3.2$$

where:

T = temperature at 27^0C in above equation.

Therefore, λv = 350 gm/m^3/day.

3.6.1.8 Influent Bacteria Concentration (Bi)

Bacteria concentration in an influent ranges between 10^7 to 10^9 faecal coliform per 100 ml. Geldreich (1970) proposed a conservative value of Bi = 1×10^8 faecal coliform per 100ml. This is higher than average value normally found in practice. However, from the analysis and test done, Bi value was given as 64×10^6 faecal coliform per 100 ml, which is within the above range.

3.6.1.9 Required effluent standards

It is assumed that the effluent will be used for unrestricted irrigation. Therefore, the following effluent standards are required

i. Faecal coliform in effluent; Be ≤ 100FC/100 ml
ii. Effluent BOD; Le ≤ 25 mg/litre

3.7 Waste stabilization pond design

3.7.1 Design of Anaerobic Ponds

An anaerobic pond was designed on the basis of the permissible volumetric organic loading λ_v which is related to flow (Q), influent BOD_5 (L_1) and pond volume (V). The volume of the anaerobic ponds (V_a) in m^3 was computed by using the formula of Mara and Pearson (1986) and Hamzeh and Ponce (2007)

$$\lambda_v = \frac{L_1 Q}{V_a} \quad \ldots\ldots 3.3$$

where: V_a = Volume of Anaerobic pond

L_i = influent BOD

Q = wastewater flow rate

λ_v = volumetric loading

Substituting all the obtained values in equation (3.3),

$$V_a = \frac{459.55 \times 2393.6}{350} = 3142.86\ m^3$$

The retention time was estimated as two days and depth was taken as 2.5m. Desludging may be needed in 2 to 5 years; therefore, two ponds are often arranged in parallel to allow one pond to be taken out of service for desludging.

Detention time t_a^* = V_a / Q ≈ 2days

No. of ponds, n_a = 2

Volume of each pond = V_a/2 = 3142.86 / 2 = 1571.43m^3

Assuming depth of each pond = 2.5m

Therefore, the area of each pond will be:

1571.43/2.5 = 628.57 m^2

Also providing one extra pond to be used during repair / desludging. Thus,

Total no. of anaerobic ponds = 3

Length = 43.42 m, width = 14.47 m, and Depth = 2.5 m [(3:1) ratio (Mohammed, 2006)]

Length = 35.46 m, width = 17.73 m, and Depth = 2.5 m [(2:1) l: w ratio (Mara, 2004)]

BOD removal in the anaerobic ponds was calculated from the following relation:

% BOD removal = 2T + 20 = 2(27) + 20= 74%

To be conservative, a value of 60% was taken.

3.7.2 Design of Facultative pond

This pond was designed by considering the maximum BOD load per unit area at which the pond will still have a substantial aerobic zone. This is because; biological activities are dependent on the temperature. Arthur (1983) gave the expression for hot climate which has been adopted in this design as

$\lambda s = 20T - 60$ ……………………………………………………………………….....3.4

Putting the value of $T = 27^0C$

$\lambda s = 20(27) - 60 = 480$ kg/ha/day

Assuming 60% BOD removal in anaerobic ponds, then the influent BOD to facultative ponds will be 40%. $Li = 0.40 \times 459.55 = 183.82$ mg/litre.

The mid-depth area of the facultative ponds (A_f) in m^2 has been calculated by using the formula (Mara and Pearson, 1987 and Mara, 2004).

$A_f = (10 \times Li \times Q) / \lambda s$……………………………………………………………….3.5

where:

A_f = Area of facultative pond

Li = Influent BOD to facultative pond

Q = wastewater flow rate

λs = surface BOD loading

Substituting the values in equation (3.5), $A_f = (10 \times 183.82 \times 2393.6) / 480 = 9166.49\ m^2$

Assuming mid-depth, $d_f = 1.75$ m, the volume of facultative ponds

$V_f = 9166.49 \times 1.75 = 16041.36\ m^3$

Detention time $t_f^* = V_f / Q = 16041 / 2393.6 = 6.7 \approx 7$days

Since a minimum value of t of 5 days should be adopted for temperatures below 20^oC, and 4 days for temperatures above 20^oC to minimize hydraulic short-circuiting and to give the algae sufficient time to multiply (i.e. to prevent algal washout). A 7 day retention time has been adopted (Hamzeh and Ponce, 2007).

The BOD removal in primary facultative ponds is usually in the range 70-80 percent based on unfiltered samples (that is, including the BOD exerted by the algae), and usually above 90 percent based on filtered samples.

Taking an average of 75% for conservative purpose, we have

75% of the incoming 40% i.e., about 30%

Therefore, the cumulative BOD removal is 60+30 = 90%

No. of facultative ponds n_f = 2, therefore,

Area of each pond = 9166.49 / 2 = 4583.25 m^2

Volume of each pond = 16041.36/2 = 8020.68 m^3

The length and width ratio of a facultative pond is usually 3:1 (Mohammed, 2006)

Area of the pond = $3x^2$ = 4583.25

x = 39.08 m

Therefore, 3x = 3×39.09 = 117.26 m

Length = 117.26 m, width = 39.08 m, and Depth = 1.75 m (3:1)

3.7.3 Design of Maturation Ponds

The number and size of maturation ponds in a system depend upon the bacteriological quality required of the effluent. The number of faecal coliform bacteria per 100 ml of the effluent (Be) can be estimated by the following equation (Mara and Pearson, 1987):

$B_e = B_i / (1 + K_{B(T)}\, t^*)$……………………………………………………………3.6

where:

Bi = Bacterial concentration in no. of FC/100ml of the effluent,

t* = Detention time

$K_{B(T)}$ = First order FC removal rate constant in T^0C per day and is computed as,

$K_{B(T)} = 2.6\ (1.19)^{T-20}$……………………………………………………………..3.7

By putting the value of T, in equation (3.7), the first order FC removal rate constant T^0C per day is given as: $K_{B(T)} = 2.6(1.19)^{27-20} = 8.78\ d^{-1}$.

The number of faecal coliform per 100ml can be calculated for the effluent from each pond in the series with the help of equation 3.8

Also the total number of faecal coliform in the effluent from the last pond of the series can be found from the equation (Mara and Pearson, 1987):

$$B_e = \frac{B_i}{\left(1 + K_{B(T)} t_a^*\right)\left(1 + K_{B(T)} t_f^*\right)\left(1 + K_{B(T)} t_m^*\right)^n} \quad \text{……...3.8}$$

where:

t_a^* , t_f^* and t_m^* are the detention times of the anaerobic, facultative and maturation ponds respectively and n is the number of maturation units in the series.

Assuming two 3-day-retention maturation ponds,

t_m^* = 4 days and n = 2 ponds, then the bacterial concentration in number of FC/100ml of effluent can be calculated from equation (3.8).

$$B_e = \frac{1 \times 10^8}{(1 + 8.78 \times 2)(1 + 8.78 \times 7)(1 + 8.78 \times 4)^2} = 66.11\ FC/100\ ml$$

The value of B_e signifies effluent standard that is less than 100FC/100ml. Thus two maturation ponds, two facultative ponds are satisfactory for the treatment of the university wastewater along with two anaerobic ponds.

Volume of each pond V_m = Q× t_m^* = 2393.6 × 4 = 9574.4 m^3

Total volume V_m= 9574.4 × 2 = 19148.8 m^3

Assuming depth of ponds = 1.5m

Area of maturation ponds = V_m/1.5 = 9574.4 / 1.5 = 6382.9 m^2

Length to width ratio = 3:1

Let x = Width of the pond in (m)

Area of the pond = $3x^2$ = 6382.9m^2

x = 46.12 m

Therefore, 3x = 3 × 46.12 = 138.36 m

Length = 138.36 m, width = 46.12 m, and Depth = 1.5m

Values of Length = 138.36 m, width = 46.12 m and Depth = 1.5 m (3:1)

Probable cumulative BOD removal at higher temperatures is 96% after maturation ponds, therefore, effluent BOD i.e.,

L_e = 4% of 459 mg/l.

L_e = 18.36 mg/l which is lower than the required.

Figure 3.4 shows the sketch of the new designed configurations for the university.

Since the pond effluent satisfies the standards for unrestricted irrigation, it can be used to grow crops.

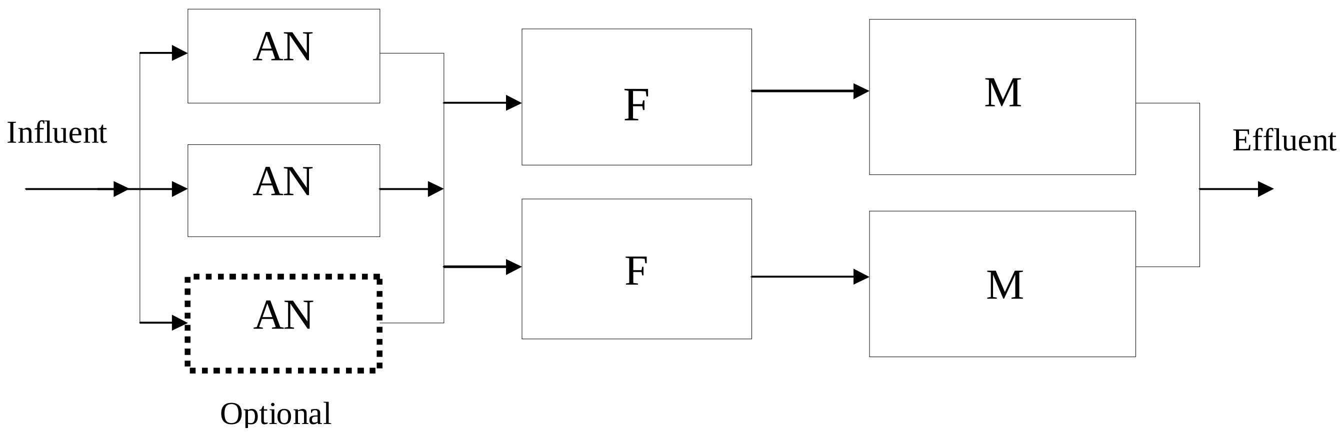

Fig 3.4 Cofiguration of the Designed WSP for Covenant University community

3.8 Design of Laboratory scale model:

The experimental design of scale models requires application of the principles of similarity and dimensional analysis if they are to yield meaningful results that are representative of full-scale systems (Shilton and Bailey 2006). Although it is recognized that the choice of scaling criteria is debatable, it was decided to design the laboratory model of the ponds based on Froude number. The existence of Froude number similarity between the model and prototype ponds for WSPs has been used successfully by Shilton and Bailey (2006). The reactor length to width ratio adopted was understood to be cost effective and also for the purpose of arrangement in the available space in the laboratory. Length to width ratio for all the ponds was taken as 3:1 (Mohammed, 2006).

3.8.1 Modeling of the Anaerobic Laboratory-scale pond

Two scale ratios were adopted for the linear dimensions (horizontal and vertical). This was due to cost reduction for the model construction and was also necessitated by the space available in the laboratory to contain the set-up.

Q_p = Discharge of the designed prototype = 2393.6 m^3/day

There are two ponds in parallel, the discharge into each pond=2393.6 / 2 = 1196.8 m^3/day

Volume of Anaerobic pond = 1571.43 m^3

Depth of each pond = 2.5 m

Area of anaerobic pond = 628.57 m^2

A scale ratio of 52.5 was adopted for the discharge after several attempts,

$$\frac{Q_p}{Q_m} = L_r^{2.5} = (40)^{2.5} \quad \ldots\ldots 3.9$$

$$\frac{1196.8}{Q_m} = (40)^{2.5}$$

$Q_m = 0.12 m^3/day$

The area of Anaerobic prototype = 628.57 m^2

Assuming a length to width ratio of 3:1, as expressed in section 3.7.1, the dimensions of the prototype pond becomes: Length = 43.42 m, width = 14.47 m, and Depth = 2.5 m

As mentioned above, two different scale ratios were adopted after several attempts: scale ratio 45.5 for the horizontal and 38.5 for the vertical dimensions

For the horizontal dimension,

$(L_r)_H$ = scale ratio for horizontal dimension

$$\frac{L_p}{L_m} = \frac{B_p}{B_m} = \frac{Linear..horizontal..\dim ension..of..prototype}{Linear..horizontal..\dim ension..of..\bmod el} \quad \ldots\ldots 3.10$$

$$\frac{43.42}{L_m} = 45.5$$

$L_m = 0.95$ m

also,

$$\frac{14.47}{B_m} = 45.5$$

$B_m = 0.32$ m

In order to get the cross sectional area of flow for the model, equation 3.11 was adopted for the area ratio. The depth of flow is 2.5m while the width was taken as 0.23m.

$$\frac{A_p}{A_m} = (L_r)_H{}^2 \quad \ldots\ldots 3.11$$

where: $(L_r)_H = 45.5$

$$\frac{2.5 \times 0.23}{A_m} = (45.5)^2$$

$A_m = 2.78 \times 10^{-4}\ m^2$

Applying the scale ratio of 38.5 for the vertical dimension

$$\frac{h_p}{h_m} = \frac{Linear.vertical.\dim ension..of.prototype}{Linear.vertical.\dim ension..of.\bmod el} = 38.5 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.12$$

$$\frac{2.5}{h_m} = 38.5$$

$h_m = 0.065$ m.

The flow width for the model

$$\frac{b_p}{b_m} = \frac{Linear.horizontal.\dim ension..of.prototype}{Linear.horizontal.\dim ension..of.\bmod el} = 45.5$$

$$\frac{0.23}{b_m} = 45.5$$

width of flow = 0.005 m

Velocity of model = $\dfrac{Q}{A} = \dfrac{0.12}{0.065 \times 0.005} = 4.27 \times 10^{-3}\ m/s$

where: Q and A are the flow velocity and cross-section area of flow path respectively.

3.8.2 Modeling of the Facultative Laboratory-scale pond

Q_p = Discharge of the designed prototype = 1196.8 m^3/day

Area of each facultative pond = 4583.25 m^2

Depth of each pond = 1.75 m

Volume of each pond = 8020.68 m^3

Assuming a length to width ratio of 3:1, the dimensions of the prototype pond becomes:

Length = 117.26 m, width = 39.09 m, and Depth = 1.75 m

Two different scale ratios were adopted after several attempts: scale ratio 55.8 for the horizontal and 38.9 for the vertical dimensions

For the horizontal dimension,

$(L_r)_H$ = scale ratio for horizontal dimension

$$\frac{117.26}{L_m} = 55.8$$

$$L_m = 2.1 \text{ m}$$

also,

$$\frac{39.09}{B_m} = 55.8$$

$B_m = 0.70$ m

In order to get the cross sectional area of flow for the model, equation 3.11 was adopted for the area ratio. The depth of flow is 1.75m while the width was taken as 0.23m.

$$\frac{A_p}{A_m} = (L_r)_H{}^2$$

where: $(L_r)_H = 45.5$

$$\frac{1.75 \times 0.23}{A_m} = (55.8)^2$$

$A_m = 1.29 \times 10^{-4}$ m^2

Applying the scale ratio of 38.9 for the vertical dimension

$$\frac{1.75}{h_m} = 38.9$$

$h_m = 0.045$ m.

The flow width for the model = 0.005 m

Velocity of model = $\dfrac{Q}{A} = \dfrac{0.12}{0.045 \times 0.005} = 6.17 \times 10^{-3}\, m/s$

where: Q and A are the flow velocity and cross-section area of flow path respectively

3.8.3 Modeling of the Maturation Laboratory-scale pond

Q_p = Discharge of the designed prototype = 1196.8 m^3/day

Area of each facultative pond = 6382.9 m^2

Depth of each pond = 1.50 m

Volume of each pond V_m = 9574.4 m^3

Assuming a length to width ratio of 3:1, the dimensions of the prototype pond becomes:

Length = 138.38 m, width = 46.13 m, and Depth = 1.50 m

Two different scale ratios were adopted after several attempts: scale ratio 56 for the horizontal and 37.5 for the vertical dimensions

For the horizontal dimension,

$(L_r)_H$ = scale ratio for horizontal dimension

$$\frac{138\ .38}{L_m} = 56$$

$$L_m = 2.47 \text{ m}$$

also,

$$\frac{46.13}{B_m} = 56$$

$$B_m = 0.83 \text{ m}$$

In order to get the cross sectional area of flow for the model, equation 3.11 was adopted for the area ratio. The depth of flow is 1.50m while the width was taken as 0.23m.

$$\frac{A_p}{A_m} = (L_r)_H{}^2$$

where: $(L_r)_H = 56$

$$\frac{1.50 \times 0.23}{A_m} = (56)^2$$

$A_m = 1.10 \times 10^{-4}$ m^2

Applying the scale ratio of 38.9 for the vertical dimension

$$\frac{1.50}{h_m} = 37.5$$

$h_m = 0.04$ m.

The flow width for the model = 0.005 m

Velocity of model = $\dfrac{Q}{A} = \dfrac{0.12}{0.040 \times 0.005} = 6.94 \times 10^{-3}\, m/s$

where: Q and A are the flow velocity and cross-section area of flow path respectively.

The model dimensions and flow rates were obtained for the treatment facility by using the above equations. These ponds have been designed maintaining Froude number similarity so as to be representative of a full-scale system. Table 3.2 shows the model dimensions for the scale ratio. Its operation would be at a constant flow rate and with a controlled climate, which would be used to test a wide range of hydraulic design variations. This wide range of experimental data would then be able to compare and contrast against simulated results obtained using a CFD model.

Table 3.2 Dimensions of Laboratory Scale models of Waste Stabilization Ponds

Parameters	Type of pond	Dimensions
Area (m^2)	AP	0.304
	FP	1.47
	MP	2.05
Length (m)	AP	0.95
	FP	2.10
	MP	2.47
Width (m)	AP	0.32
	FP	0.70
	MP	0.83
Depth (m)	AP	0.065
	FP	0.045
	MP	0.040

3.9 Laboratory studies

3.9.1 Construction of the laboratory-scale waste stabilization reactors

The first set of laboratory-scale waste stabilization reactors was constructed at Lola Technical Engineering Services located at Km 9, Idiroko road, Ota while the CFD/optimized design set was constructed within Covenant University. Details of the design for the laboratory-scale reactors are as given in section 3.8.1-3.8.3 (p.75-83) of METHODOLOGY. For the anaerobic pond, the designed full-scale pond that the model represent are 43.42 m in length, 14.47 m in width and 2.5 m in depth. The facultative pond designed prototype dimensions are 117.26 m in length, 39.09 m in width and 1.75 m in depth while the maturation pond dimensions are 138.38 m in length, 46.12 m in width and 1.5 m in depth respectively.

The model was housed within a confined, constant room temperature of 24^0C to minimize temperature changes and exposure to air currents. The experimental design of scale models required application of the principles of similarity and dimensional analysis if they are to yield meaningful results that are representative of full-scale systems. Although it was recognized that the choice of scaling criteria is debatable, it was decided to design the pond laboratory model for Froude number similarity. The reactors were placed side by side as shown in Plate 3.6. The dimensions of the reactors constructed are:

Length of model = 0.95 m, width of model = 0.32 m and depth of model = 120 mm for anaerobic reactors.

Length of model = 2.09 m, width of model = 0.70 m and depth of model = 100 mm for facultative reactors

Length of model = 2.47 m, width of model = 0.825 m and depth of model = 100 mm for maturation reactors

The model's flow rate was set to maintain Froude Number similarity with the prototype and had the scaling factors of 1: 40

$$\frac{Q_m}{Q_p} = L_r^{2.5} = (40)^{2.5}$$

$Q_m = 0.12\ m^3/d$

The primary objective of this exercise was to provide sets of reliable data for input to the computational fluid dynamics (CFD) model. Though the number of experimental runs undertaken was limited due to time constraint and available resources, the data produced still allowed some general evaluations of the hydraulic behavior to be made and these are discussed towards the end of this chapter.

3.9.2 Materials used for the construction of the inlet and outlet structures

1½ inch galvanized plate

2 inch square pipe for the stands

¾ inch G.I Socket

¾ inch G.I Tee

¾ inch G.I Nipple

¾ inch Italy gate valve

½ inch red handle tap

½ inch G.I Socket

¾ inch Plug

1inch Hose

Clips for tightening the hose to the sockets

Aluminum gloss paint to avoid corrosion of the plates

There are two sets of the three reactors in series (anaerobic, facultative and maturation reactors) sited parallel to each other. each pond was costructed using 1.5 inch galvanised plate welded at conners as designed (Plates 3.6 and 3.7). An elevated tank of 1 m^3 serves as a reservoir for the supply of the wastewater to the first pond in series (the anaerobic pond). This tank was feed continuously with wastewater to maintain a constant head. The tank serves the reactors in series at a constant flow rate of 0.12 m^3/day equivalent to 5 liters/hr. The tank is position to supply the wastewater by gravity. This was used for continuous feeding of raw wastewater and is void of pipe blockage into the three treatment units constructed (Plate 3.8).

The inner and outer walls of the galvanised material as shown in Plate 3.9, were painted with alumminium gloss paints to avoid rusting and reaction with the wastewater. Each of the reactors have varying width (w), depth (d) and length (l) based on design. The construction of the pond was thoroughly welded to prevent leakage of the wastewater. It was necessary to do this as any wastewater leakage could have effect on the hydraulic flow pattern, thereby making validation of the data collected for the purpose of sample analysis inaccurate. The reactors were installed and connected to each other at equal distance along the series and also spaced at a distance equal to the length of the hose connecting the two reactors in parallel by the use of inlet-outlet alternation (Plate 3.9).

The laboratory-scale pond (anaerobic reactors) has total top surface dimensions of 0.95 m long, 0.32 m wide and 0.12 m depth, giving an estimated volume of 0.0365 m^3 (Plate 3.10). The position of the inlet and outlet from the pond base is at 65 mm allowing a free board as designed for the anaerobic pond volume detailed in section 3.8.1. This gives a total water surface volume of 0.01976 m^3. All the stands for the three reactors in series and parallel were of 1 m height except for the stand of the wastewater holding tank that is 1.2 m height to allow for flow by gravity. The facultative reactors have a total surface dimensions of 2.09 m long, 0.70 m wide and 0.10 m depth, giving an estimated volume of 0.146 m^3 (Plate 3.11).

The position of the inlet and outlet from the pond base is at 45 mm allowing a free board as designed. This gives a total water surface volume of 0.0658 m^3. The maturation reactors have a total surface dimensions of 2.47 m long, 0.83 m wide and 0.10 m depth, giving an estimated volume of 0.205 m^3 (Plate 3.12). The position of the inlet and outlet from the pond base is at 40 mm allowing a free board as designed. This gives a total water surface volume of 0.082 m^3. The maturation is usually shallower compared to the other reactors.

Plate 3.6 Front view of the laboratory-scale reactors

Plate 3.7 Areal view of the laboratory-scale reactors

Plate 3.8 An elevated tank serving as reservoir.

Plate 3.9 Inlet-outlet alternation of laboratory-scale WSP

Plate 3.10 Laboratory-scaled anaerobic reactors

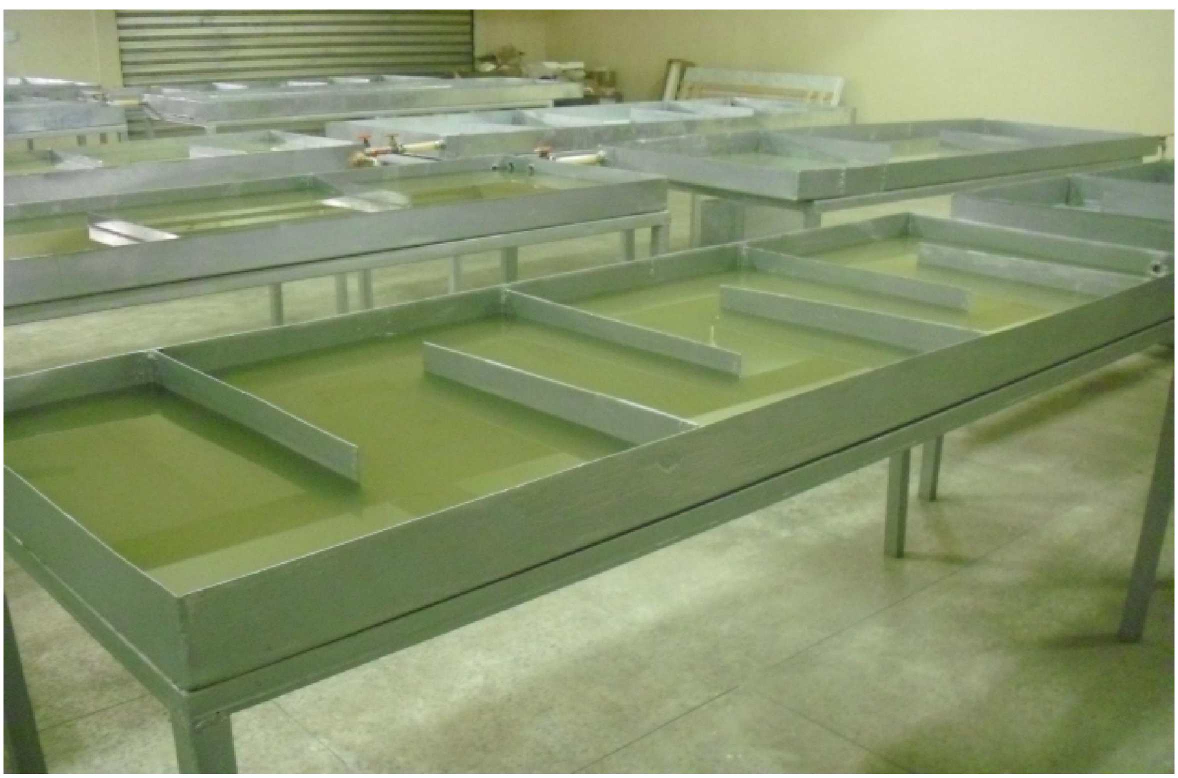

Plate 3.11 Laboratory-scaled facultative reactors

Plate 3.12 Laboratory-scaled maturation reactors

3.9.3 Design of inlet and outlet structures of the WSP

A 25mm PVC hose was used to connect the outlet-inlet of the reactors in series and also in parallel. The inlet and outlet joint was made of ¾ inch socket welded to the designed inlet and outlet position of all the reactors (Plate 3.13). The sockets were coupled with nipples of their size to allow the fixing of clips in order to make it water tight and also to avoid leakages. Two 25-mm PVC hoses were linked with the T-connector (Plate 3.14) that was connected to the raw wastewater holding tank inlet carrying wastewater into the two anaerobic reactors in parallel. Control valves were screwed to position (Plate 3.15) and the inlet and outlet positions were alternated to allow for proper residence time (Shilton and Harrison, 2003a). The outlet structures were connected to two pieces of ½ inch hoses inserted to the mouth of the taps at a vertical height of 1.0 m from the base of the pond to allow effluent to discharge into the effluent tank (Plate 3.16). The effluent tank has a total surface dimension of 0.64 m long, 0.33 m wide and 0.33 m depth, giving an estimated volume of 0.0697 m^3.

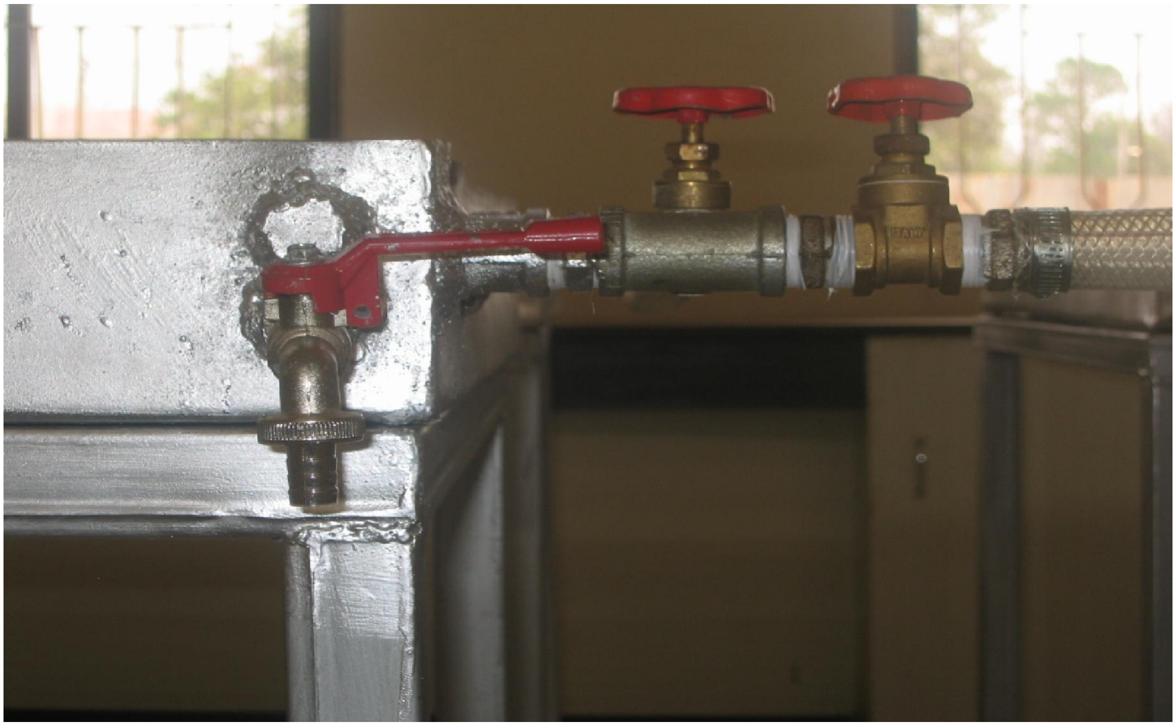

Plate 3.13 Inlet and outlet structure of the laboratory-scale waste stabilization pond

Plate 3.14 Two 25-mm PVC hoses linked with the T-connector

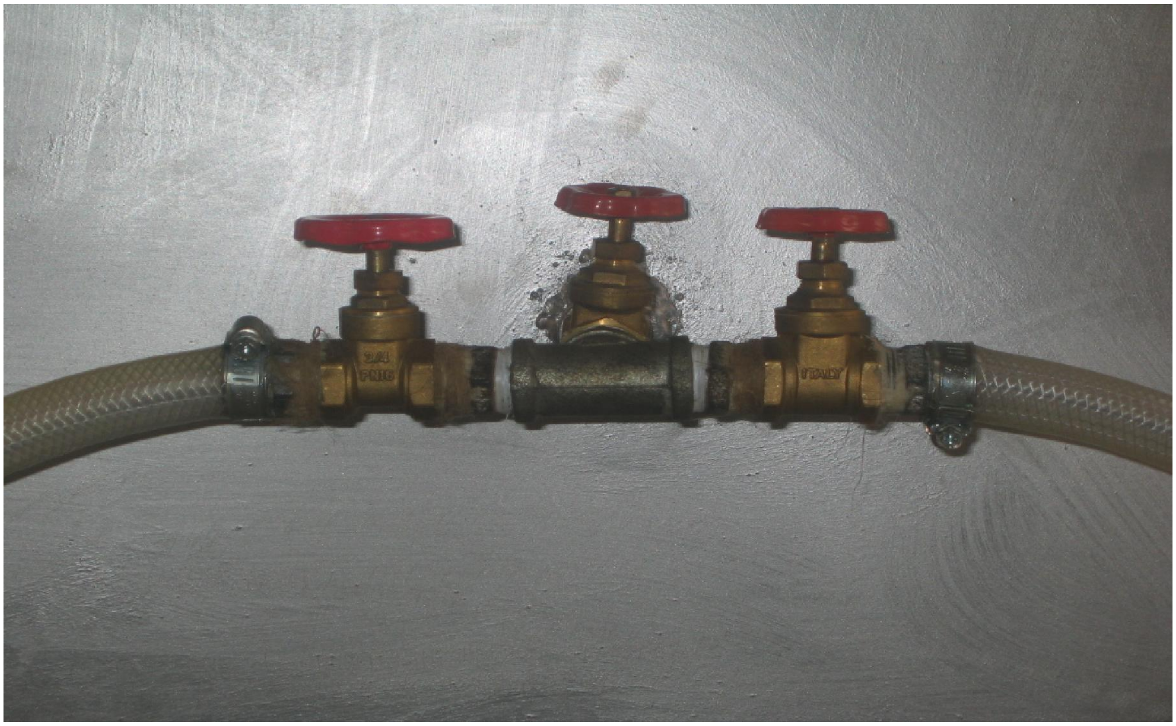

Plate 3.15 Control valves screwed to position for wastewater flow

Plate 3.16 outlet structures connected to two pieces of ½ inch hoses for effluent discharge

3.9.4 Operation of the Laboratory-Scale waste stabilization pond

At the start of operation when the installation was completed, a trial run of the system was conducted. The water holding tank was filled with fresh water from the tap. This was necessary to ascertain the flow rate of 0.24 m^3/day equivalent to 10 liters/hr for wastewater to flow into the reactors in series as designed. A calibrated bucket was used to determine the volume of water that filled it over a given time. The gate valve from the tank was adjusted until the desired flow of 10 liters/hr was accomplished. This was further divided into two to maintain continuity of flow ($Q = Q_1 + Q_2$) through the Tee-pipe and to allow water to flow into the two series of pond in parallel (Plate 3.14). The same procedure was followed to have a flow rate of 0.12 m^3/day equivalent of 5 liters/hr flowing simultaneously into the sets of reactors in series by controlling the gate valves. It was at this stage that the hose was now connected to the anaerobic pond and the fresh water began to flow into it.

It took 3 hrs 55 minutes for the first pond to be filled to the inlet-outlet positions in the reactors. The flow further moved into the facultative reactors and it took 13 hrs 38 minutes to be filled to the inlet-outlet positions in the reactors while the maturation reactors took 16 hrs 40 minutes to be filled before the water finally got into the effluent holding tank. This time range was a valuable resource to verify the flow rate that was initially set through the gate valve connected to the effluent holding tank. After the trial experiment had been performed, the fresh water was ejected from the tank and all the reactors. It was at this stage that the water holding tank was filled with the raw wastewater from the existing treatment facility of the university to carry out the first major experiment without baffles.

3.9.5 Sampling and data collection

3.9.5.1 Water temperature

The laboratory temperature was taken as room temperature (24^0C) since the temperature in the field would be different to what we would have in the laboratory where the experiment was performed. This value was used in the modeling to represent the temperature in the first order kinetics equation.

3.9.5.2 Influent and effluent samples

In order to assess the treatment performance of the laboratory-scale reactors before the modeling exercise, influent and effluent grab samples were collected weekly at about 9:00 am for the analysis of the indicator parameters (Faecal coliform, Nitrate, Chloride, Phosphate, Sulphate, PH, Conductivity and Total Dissolved Solids). Samples were collected at both inlet and outlet position of each pond in series. The samples of the effluent from succesive reactors were taken to the laboratory on the same day of collection. The result of the effluent principal pollutant (Faecal coliform) was to serve as input parameters in the CFD model for the calibration, testing and validation of the model.

3.10 Laboratory methods

The results of samples that were taken to the commercial laboratory for analysis before modeling had some controversies in terms of the method used especially for analyzing the BOD_5. As at that time, the laboratory in the department could not carry out the tests because some of the equipments were not set. Every effort made in informing the commercial laboratory about the errors was not accepted and they stood on the fact that their method was correct. This raised a thorough controversy until it was advised by the co-supervisor that the data were characterized with errors and could not be used based on the low value recorded which does not represent the true state of the wastewater samples.

More so, the anaerobic pond was exempted in the verification process because samples of influent going into the reactor were not collected for analysis during experimentation. This was due to the assumption that measured wastewater going into the holding tank is the same as what goes into the anaerobic reactor. It was later realized that some biodegradation would have taken place in the wastewater reservoir. The laboratory experiments were later performed after the modeling exercise at Covenant University with improved knowledge about the sources of errors in order to verify part of the CFD/optimization results produced. Samples were collected at both inlet and outlet position of each pond in series for the analysis of Feacal coliform, Chloride, Sulphate, Nitrate, Phosphate, Total Dissolved Solids, Conductivity and pH.

3.10.1 Faecal coliform

The fecal coliform bacteria for the influent and effluent samples were determined at the microbiology laboratory at Covenant University by the membrane filter procedure 9222 D which uses an enriched lactose medium and incubation temperature of 44.5 ± 0.2°C for selectivity. The results of the above parameters are presented in the result section.

3.10.2 Chloride

Measurement of chloride concentration on both influent and effluent samples was done by using the Hanna Instruments (H1 3815 chloride test kit) with reference to official methods of analysis, A.O.A.C, 14th edition, 1984, p.625 and the Standard methods for the examination of water and wastewater, 16th edition, 1985, p. 288-290.

3.10.3 Sulphate

Measurement of sulphate concentration on both influent and effluent samples was done by using the Hanna Instruments (H1 38000 Sulfate test kit) with reference to the adaptation of the Barium Sulfate Turbidimetric Method.

3.10.4 Nitrate

Measurement of Nitrate concentration on both influent and effluent samples was done by using the Hanna Instruments (H1 93728 Nitrate test kit) with reference to adaptation of the cadmium reduction method

3.10.5 Phosphate

Measurement of Phosphate concentration on both influent and effluent samples was done by using the Hanna Instruments (H1 93713 Phosphate test kit) with reference to the adaptation of the Ascorbic Acid method.

3.10.6 Total Dissolved Solids (TDS)

The TDS was done using the HANNA C99 Multiparameter Bench photometer

3.10.7 Conductivity

The conductivity was done using the HANNA C99 Multiparameter Bench photometer

3.10.8 pH

The pH was done using the HANNA Instruments pH meter

3.11 Tracer Experiment

An attempt was made to perform a tracer study in the laboratory during the experiment. Sodium Aluminum Sulphosilicate (locally called “blue”) was used to access the hydraulic performance of the three reactors in series. The mean hydraulic retention time is the indicator parameter of the hydraulic efficiency of the laboratory-scale pond. The experimental data were to be used as validation data for the CFD model that was used in the research but the tracer chemical did not perform as expected. The chemical faded out in a short time before it got to the outlet of the reactor and there was no equipment to measure its concentration (Plates 3.17 and 3.18).The wastewater quality was observed to improve as it moved to successive treatment unit (Plate 3.19)

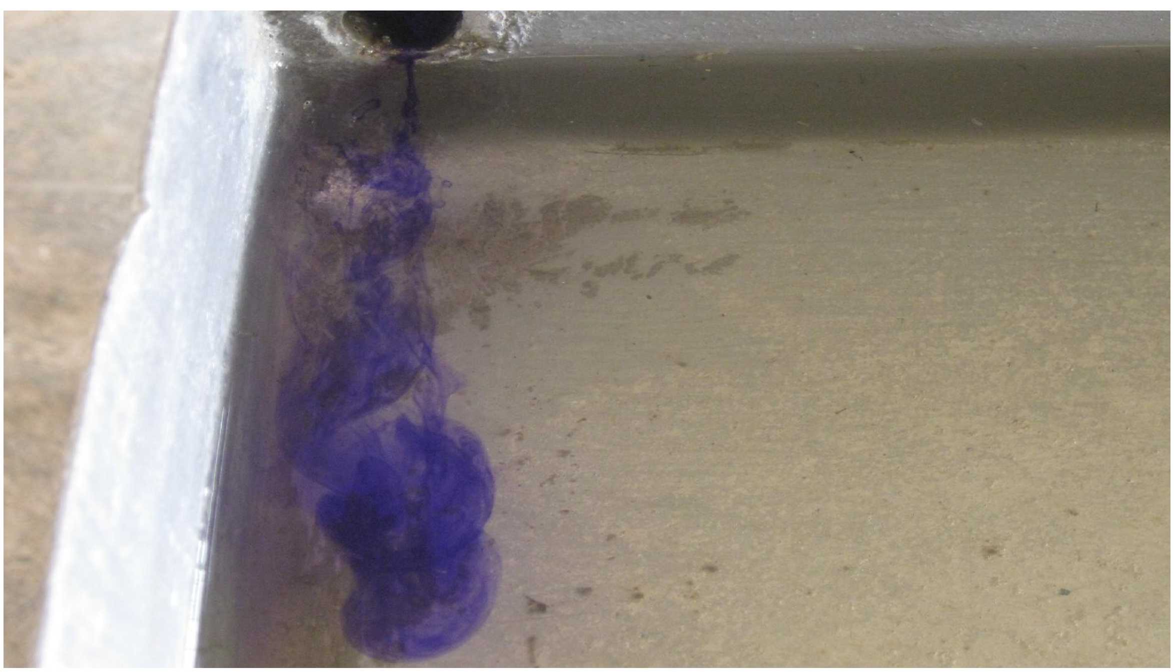

Plate 3.17 Tracer experiment with Sodium Aluminum Sulphosilicate

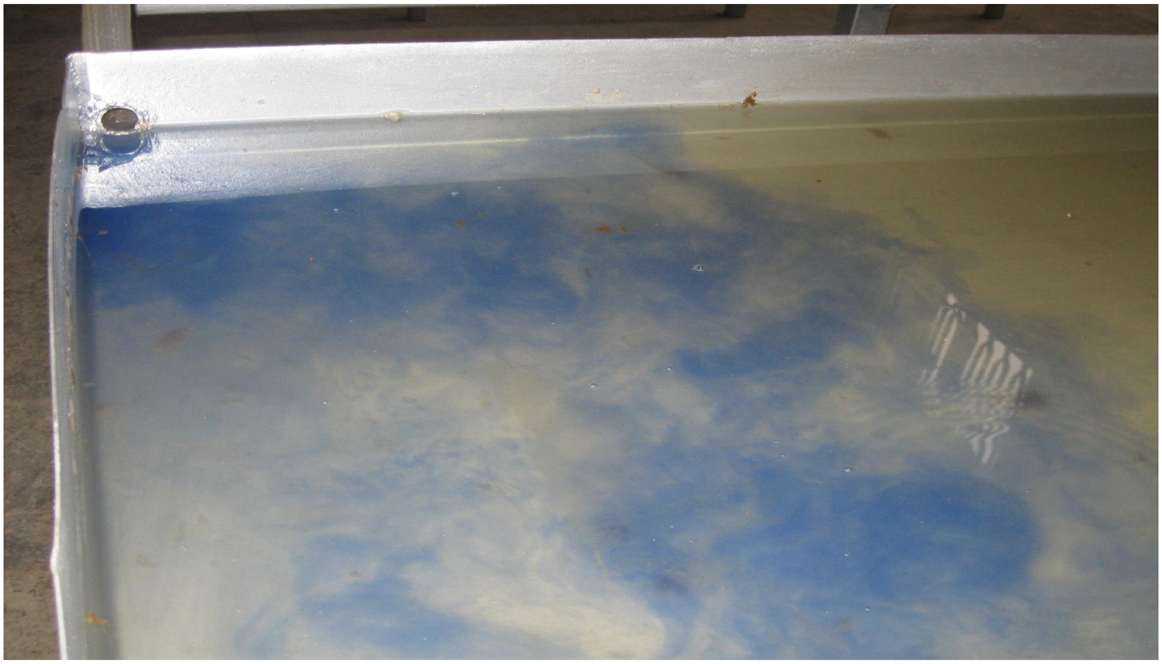

Plate 3.18 Tracer chemical diluting with the wastewater before getting to the outlet.

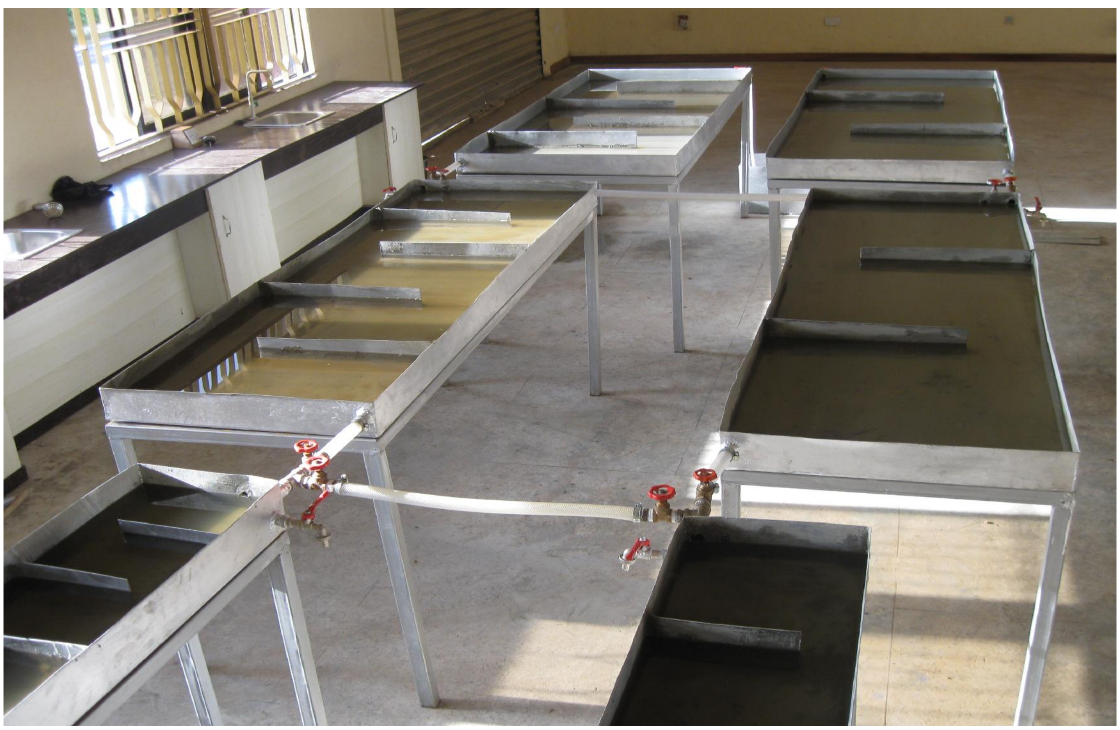

Plate 3.19 Improvement in wastewater quality along the units

3.11.1 Determination of First Order Kinetics Rate constant (k) value for Residence time distribution (RTD) characterization

In this research, prior to the modeling exercise, series of models were tested in the laboratory using evenly spaced baffles of 70% and 80% pond-width in the transverse arrangement. Each baffle length was tested using 2, 4, and 6 evenly spaced baffles. The 70% pond width has been taking as a base for this research because it has been discussed by various researchers to be the most hydraulically efficient option and that it gives superior performance compared to 50% and 90% (Shilton and Harrison, 2003a). It was decided to use the decay of faecal coliform as a parameter. This was chosen to be simulated in this study because it is a reliable and commonly used indicator of effluent quality and the experimental data were reasonable enough to represent the real state of the wastewater quality. It is also convenient from a computational point of view as its decay follows the first-order kinetic theory. Figures 3.5 to 3.7 illustrate the example of 70% pond-width configurations that were tested for wastewater quality parameters.

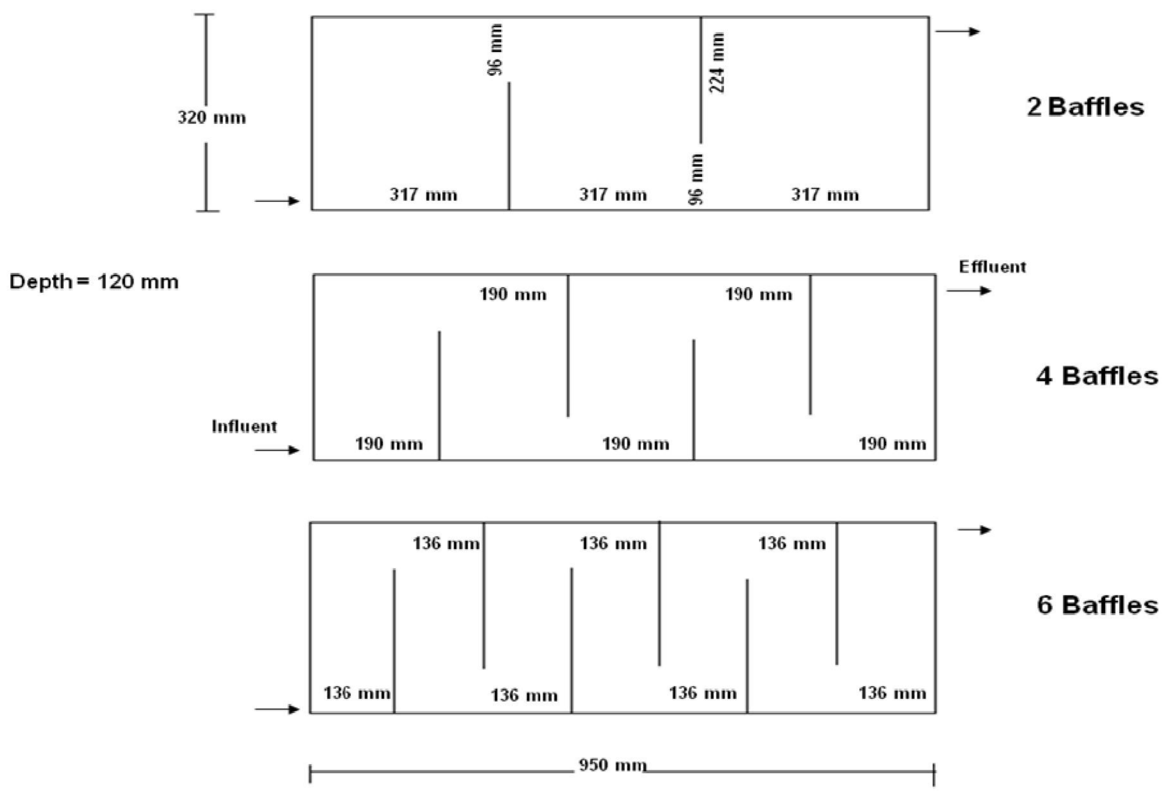

Figure 3.5 Different baffle arrangements with 70% pond width anaerobic reactors

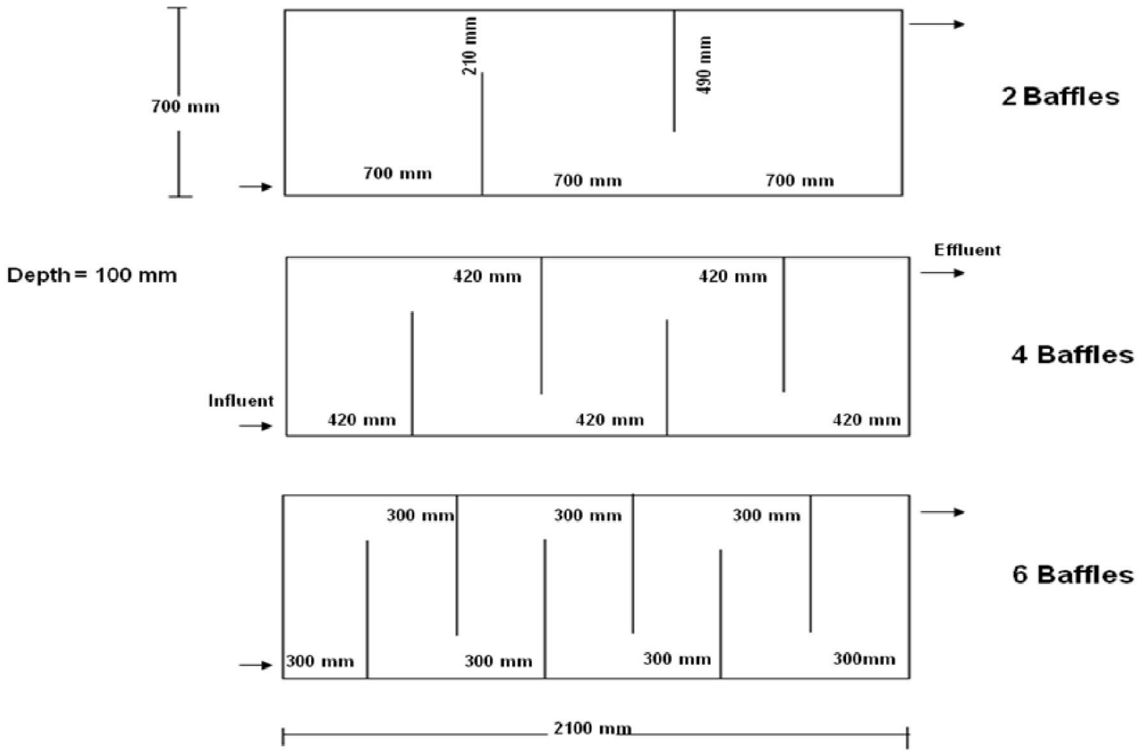

Figure 3.6 Different baffle arrangements with 70% pond width facultative reactors

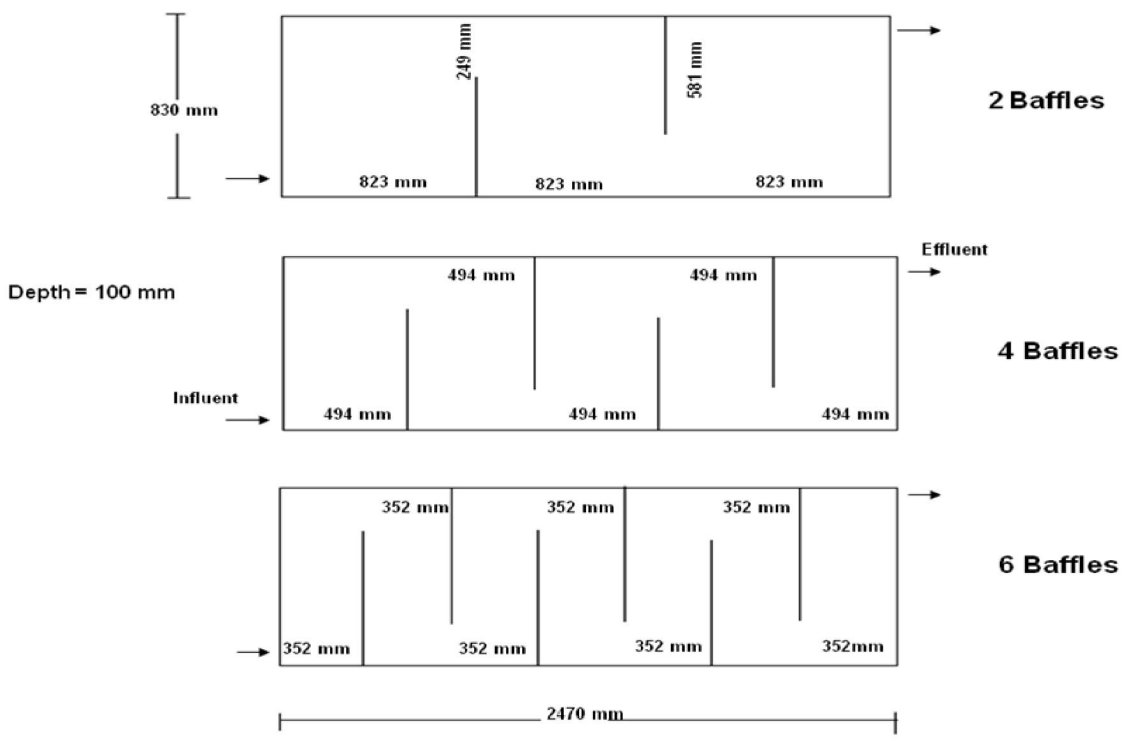

Figure 3.7 Different baffle arrangements with 70% pond width maturation reactors

3.11.2 The gamma extension to the *N*-tanks in series model approach

The tracer experiment to verify the mean hydraulic retention time which is an indicator parameter of the hydraulic efficiency of the laboratory-scale pond was attempted but because the tracer chemical used did not perform as expected, it was therefore expedient to use mathematical models to characterize the RTD. The complete mixed tank (N-tanks) in series model was adopted. This is based on setting up mixing tanks in series and allowing the fluid to flow from the effluent of one tank to the influent of the following tank. The pattern is repeated for *N* number of tanks. The *N*-tanks in series model is also very simple to use and requires little input data to generate the RTD function (Ducoste et al., 2001; Clark, 1996). The basic approach utilizes the gamma extension to the *N*-tanks in series model and is displayed in equation 3.14 as:

$$f(\theta) = \frac{N^{N}\theta^{N-1}}{\Gamma(N)} e^{-N\theta} \qquad 3.14$$

Where Γ(N) is a gamma function:

$$\Gamma(N) = \int_{0}^{\infty} \exp(-x) x^{N-1} dx \qquad 3.15$$

The tanks-in-series model imagines a number of perfectly mixed tanks of equal size arranged in series.

Prior to the modeling exercise, the anaerobic pond was exempted in the verification process because samples of influent going into the reactor were not collected for analysis during experimentation. This was due to the assumption that measured wastewater going into the holding tank is the same as what goes into the anaerobic reactor. It was later realized that some biodegradation would have taken place in the wastewater reservoir. This resulted in limiting the analysis to the facultative and the maturation reactors using the N-tanks in series model. The data collected was fitted with the open complete mixed tanks (N-tanks) in series model and the gamma extension N-tanks in series model (equation 3.14 and 3.15). The data conversion for reactor length to width ratio to N for N-tanks in series by Crozes et al., (1998) and Ducoste et al., (2001) was adopted (Figure 3.8).

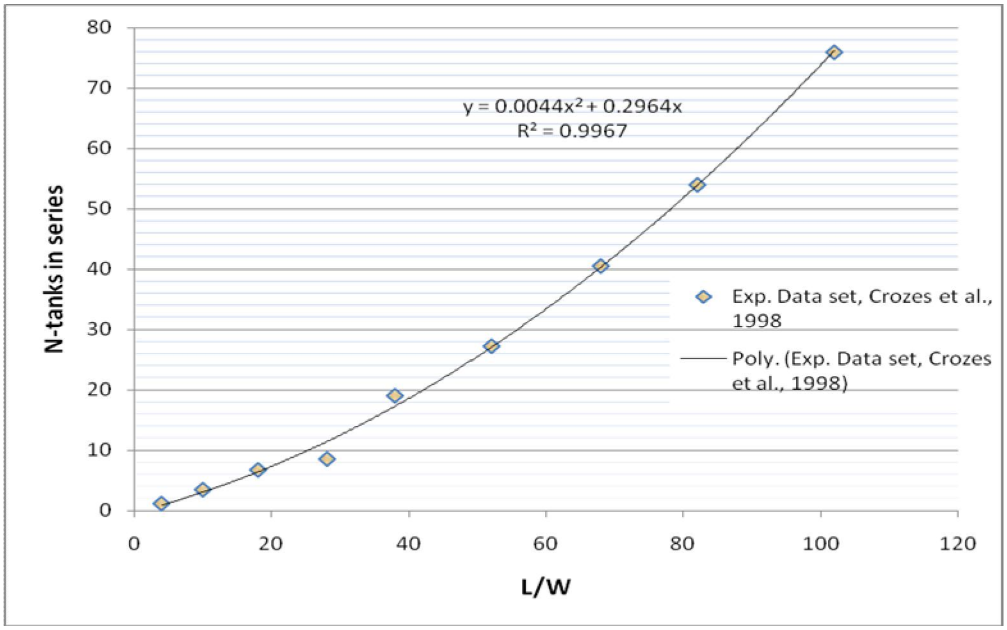

Figure 3.8 Data conversion for reactor length to width ratio to N for N-tanks in series model. Source: Ducoste et al., 2001.

Figure 3.9 gives a description of the length to width ratio for the baffle configurations used in this research. The length cuts across the wastewater flow line in the reactor.

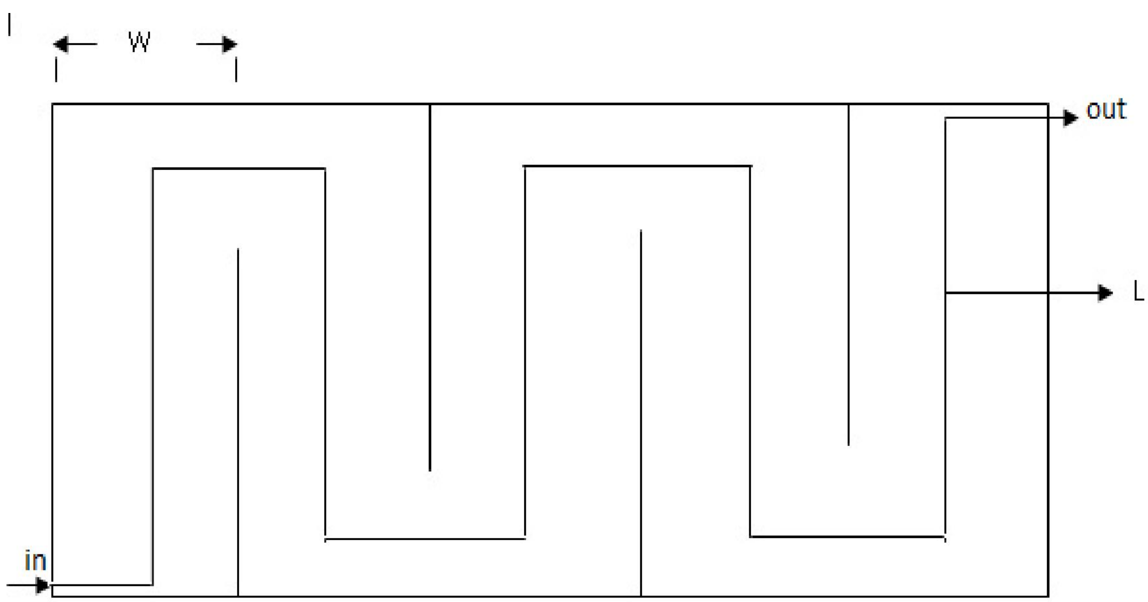

Figure 3.9 Description of length to width ratio for the laboratory-scale model

3.12 Methodology and application of Computational Fluid Dynamics model

3.12.1 Introduction

The use of CFD tool to investigate the potential hydraulic performance and improvement due to different baffle configurations, outlet positions, depth, and velocity has given a great insight into the hydraulics of waste stabilization ponds. CFD model and simulation have been used to produce data on the flow patterns, tracer responses and faecal coliform inactivation within the designed laboratory scale waste stabilization pond. In both instances, these were undertaken for a set of different pond configurations. In addition to predicting the pond hydraulics, the first order kinetics was incorporated into the CFD model to determine the decay of water quality indicator (faecal coliform). This allowed direct evaluation of the treatment performance of various physical arrangements in the ponds.

The knowledge derived from the construction of the Laboratory-scale pond gives the insight into the geometry of a reactor model that is based on a CFD methodology of solving the time dependent flow and transport equations on a 2D depth-integrated model in COMSOL Multiphysics modeling tool. COMSOL Multiphysics is the first engineering tool that performs equation-based multiphysics modeling in an interactive environment (COMSOL Manual. 2005). The outcome is a model that can predict precisely the velocity distribution, residence time distribution and faecal coliform concentration at all points in the reactor of which the effluent is of utmost interest to the researcher.

The decay of faecal coliform as a parameter was chosen to be simulated in this study because it is a reliable and commonly used indicator of effluent quality. It is also convenient from a computational point of view as its decay follows the first-order kinetic theory. The first-order kinetic reaction has been formulated such that it is compatible with the source term function in the convective diffusion equation in COMSOL Multiphysics.

BOD_5 could have been simulated as well but the fact that it follows the same first order kinetics for decay, only faecal coliform was simulated. The velocity distributions and fluid transport were simulated based on the Incompressible Navier-stokes equation that is incorporated into the modeling tool. In addition, the first-order kinetic reaction depends mainly on the temperature and this is simple and accurate to measure when designing and evaluating treatment performance of waste stabilization ponds. Other nutrient parameters such as Nitrate, Chloride, Phosphate, Sulphate, PH, Conductivity and Total Dissolved Solids were not incorporated into this CFD model because the removal of these parameters depends on various processes such as algae uptake, sedimentation, vaporization and denitrification, which are more complex to model as several sub-models and empirical measurement would be required. In addition, it would also be difficult to test and verify the results of such complex models.

The use of COMSOL Multiphysics CFD has allowed the analysis of the residence time distributions curves for waste stabilization pond under study. COMSOL Multiphysics supplies a number of easy-to-use tools and commands to help with modeling and analysis. The model results of these simulated residence time distribution can help designers to identify the physical design interventions that can be used to minimize the extent of hydraulic short-circuiting and stagnation regions that are intrinsic in many waste stabilization ponds discussed in literature. An indication of residence time is a pointer toward the effectiveness of a reactor design which can measure the maximum, minimum and the average time that a tracer is present within the reactor to provide information about the reactor design and the extent of mixing.

The Chemical Engineering Module 3.4 package in COMSOL Multiphysics modeling environment was used to run the simulation. This has a customized user interface and functionality for the analysis of transport phenomena that is coupled to chemical reactions. It consists of a number of modeling interfaces for the modeling of laminar and turbulent fluid flows, multiphase flow, multi-component mass transport, and energy transport in reacting systems. These interfaces contain all the steps in the modeling process: drawing the geometry, specifying the chemical and transport properties, meshing, solving, and post-processing.

The basic structure of COMSOL Multiphysics divides transport phenomena into momentum transport, energy transport, and mass transport, which is also the underlying organization of the Chemical Engineering Module. This consists of a number of modeling interfaces, called application modes, which form the backbone of the module. These are based on the equations for momentum transport, energy transport, and mass transport which are found under the Chemical Engineering Module folder in the Model Navigator.
The modeling process consists of six main steps expressed as follows:

1. Selection of appropriate mode in the model navigator
2. Drawing or importing the model geometry in the draw mode
3. Setting up the sub-domain equations and boundary conditions in the physics mode
4. Meshing in the mesh mode
5. Solving in the solve mode
6. Post-processing in the post-processing mode.

All the steps listed above are available in the graphical user interface. Once the model is defined, changes can always be made in input data, equations, boundary conditions, and mesh. The solver can also be restarted using the existing solution as an initial condition or initial guess. More so, the geometry can also be altered. The equations and boundary conditions are always available through the associative geometry feature. By adding an application mode, a phenomenon not previously described in a model can be accounted for.

The modeling of transport-reaction processes usually involves highly nonlinear terms in the equations, which can have their origin in the reaction kinetics, in the convective term in the transport equations, and in the strong coupling between different transport phenomena. These nonlinearities result in specific requirements for the preparation of a model and its setup. The use of 2D model in this research has several advantages: it takes a shorter time to run, it requires a comparably small amount of computer memory, and it is easier to verify and validate because the results are easier to generate and interpret. Moreover, 2D model can be verified by using mesh convergence tests and other tests that might be very costly in 3D.

3.12.2 CFD Model Application

3.12.2.1 Simulation of fluid mechanics fecal coliform inactivation in the WSP

In this study, a finite element-based commercial CFD code was used. The simulation of fecal coliform and fluid transport within the WSP requires the solution of the conservation of mass (continuity), momentum (Navier-Stokes) and convective-diffusion equations [Equations (3.16)-(3.18)].

Continuity:

$$\sum \frac{\partial u_i}{\partial x_i} = 0 \tag{3.16}$$

Navier-Stokes:

$$\rho \frac{\partial u}{\partial t} - \nabla.\eta\left(\nabla u + (\nabla u)^T\right) + \rho u.\nabla u + \nabla p = F \tag{3.17}$$

Convective-Diffusion

$$\frac{\partial c_i}{\partial t} + \nabla.(-D_i \nabla c_i + c_i u) = R_i \tag{3.18}$$

where:

$\nabla.u = 0$

ρ = Density of fluid (kg/m^3)

η = Dynamic viscosity (kg/(m.s))

F = volume force term in the x and y direction (N/m^3)

u = velocity (m/s)

t = time (s)

p = pressure (N/m^2)

D_i = diffusion coefficient (m^2/s),

R_i = the reaction term (mol / (m^3·s)).

c_i = concentration of fluid

The incompressible Navier-Stokes equation is the general momentum balance and continuity equations for fluids with constant density. The simulation of fluid transport within the WSP requires the solution of the conservation of momentum (Navier-Stokes) equation 3.17.

Equation 3.17 describes the flow of incompressible Newtonian fluids, and this formulation was used for the laminar flow regime with a constant wastewater density. The flow equation was solved together with the Convective-Diffusion equation. As a rule, it is always good to evaluate the Reynolds number related to the specific flow conditions of the model because its magnitude guides in choosing the flow model and the corresponding application mode. The Reynolds number is well within the limits of the laminar flow regime and the Incompressible Navier-Stokes application mode is appropriate to model the flow.

Apart from the domain equations, proper boundary conditions were selected for both the inlet and the outlet. At the inlet a velocity vector normal to the boundary was specified as:

$$u = -u_0 n \qquad 3.19$$

where:

u_0 = u_in normal inflow velocity (m/s) at the inlet

At the outlet boundary, pressure was specified as:

$$\eta (\nabla u + (\nabla u)^T) n = 0, \qquad 3.20$$

where: η = dynamic viscosity

u = velocity (m/s)

T = temperature and n represents the normality of the inflow velocity

p = p_0 (Pa) pressure at the outlet

Finally, at the surfaces of the reactor the velocity was set to zero, that is, a no-slip boundary condition. By selection of the incompressible Navier-Stokes application mode, momentum balance equation 3.17 and boundary conditions equations 3.19 and 3.20 was easily associated with the modeling geometry.

Equation 3.18 expresses the material balance and transport by diffusion and convection for species in dilute solutions. This was used in determining the resident time distribution which is a time-dependent equation and also for the coliform inactivation in the reactor domain. The first-order reaction model of the rate of faecal coliform inactivation is equivalent to the scalar transport equation and when isothermal conditions develop in the pond, the wastewater density is taken to be constant (Perry and Green, 1984).

The first term in equation 3.18 describes the accumulation of species over time while the second term and third term signify transport due to diffusion and convection, respectively. Also on the right-hand side, R_i term represents the creation or ingestion of species *i* due to chemical reactions. In order to implement chemical reactions in the model, the application mode was set up to account for the reacting species and subsequently enter the rate expressions into the R edit field in the sub-domain settings dialog box.

For the boundary conditions, the concentration of tracer at the inlet was specified as:

$$c_2 = c_{2\,0} \qquad 3.21$$

where:

$c_{2\,0} = c\_in$ concentration (mol/m^3)

At the outlet, it was specified that the mass flow through the boundary is convective dominated. This assumes that any mass flux due to diffusion across this boundary is zero. Convective flux was selected at the outlet and finally, at the surface of the reactor, it was assumed that no mass is transported across the boundaries, that is, an insulation boundary condition:

The simulations were performed at steady state. The reaction term in Equation 3.18 was used for characterizing the fecal coliform inactivation kinetics. For the boundary conditions, the concentration of coliform was specified at the inlet. The inlet boundary condition was specified as Coliform = $coliform_0$ (1×10^8) mol/m^3 concentration while at the outlet, convective flux was specified. Using these boundary conditions, the faecal coliform inactivation was well represented in the CFD model of the laboratory-scale WSP. It was this equation that enabled the simulation of the transport of pollutants in the modeled laboratory-scale ponds. By selecting the Convection and Diffusion application mode, the convective diffusion equation 3.18 and boundary conditions were easily associated with the modeling geometry

A central differencing scheme was used for the diffusion terms. Two convergence requirements were used in this study. First, the sum of the absolute residual sources over the whole solution domain must be less than 0.1% of the reference quantities based on the total inflow for a specific variable. Second, the values of the monitored dependent variables at several locations must not change by more than 0.1% between successive iterations.

3.12.2.2 Constants used in the application modes

rho = 997.38 (kg/m^3) Density of the wastewater at room temperature. It was taken that an isothermal condition exists in the ponds and therefore, the constant wastewater density.

eta = $9.11e^{-4}$ (Pa.s) Dynamic viscosity at the same temperature

u = $4.27e^{-3}$ (m/s) velocity of flow into anaerobic reactor

c_0 = 1 (mol/m^3) inlet peak concentration

k = 9.124 (1/d) first order decay rate

The inactivation of faecal coliform depends on values of the wastewater density and the first-order rate constant of which temperature is a function.

c_in = c0*exp (-(t[1/s]-3)^2) inlet concentration

fst = -k*coliform Inactivation for first-order kinetics in the reactors

The inclusion of first order kinetics within COMSOL is achieved by using the user defined functionality (UDF) that is provided by the package. The UDF facility allows the user to develop extensions to the basic CFD functionality.

Boundary integrations are expressed as:

Inlet: c_int_in

Outlet: c_int_out

From all the expression given above, it can be taken that all the application mode equations and functions has been developed and incorporated correctly into the COMSOL Multiphysics CFD model.

3.12.2.3 Mesh generation for the computational fluid dynamics model

A set of 24 configurations including baffling and pond geometry were modeled. The mesh partitions the geometric model into small units of simple shape. The mesh was generated by entering the mesh mode to initialize the mesh. In creating the unstructured mesh, the number of mesh elements was determined from the shape of the geometry and various mesh parameters. For the 2D geometry, an unstructured mesh consisting of triangular elements was chosen. Though finer mesh takes a larger computing time, it always gives a better result. The accuracy of the CFD solutions depends also on the quality of the grid. The grid size was determined through successive refinement in the grid and evaluating the impact of that size on the local fecal coliform concentration and mean velocity at selected points in the reactors. The final maximum grid size was specified as 5 percent of the width of each reactor. The grid spacing was small enough to produce a grid-independent solution without significantly impacting the computational cost. The simulations were performed on a desktop computer (Intel ® Core(TM) 2 Duo CPU E6550 with 2048MB RAM).

It was possible to specify the mesh-element sizes and control their distribution by using the mesh parameters. All mesh parameters aim at prescribing the maximum allowed mesh element size. This principle was followed by the author when using CFD to simulate the hydraulic flow patterns and the treatment efficiency of the model laboratory-scale WSP that was run with different configurations. Table 3.3 describes the mesh statistics and other parameters that were specified based on the specifications recommended in the manual. Figures 3.10-3.12 describe the mesh generation for the unbaffled laboratory-scale model.

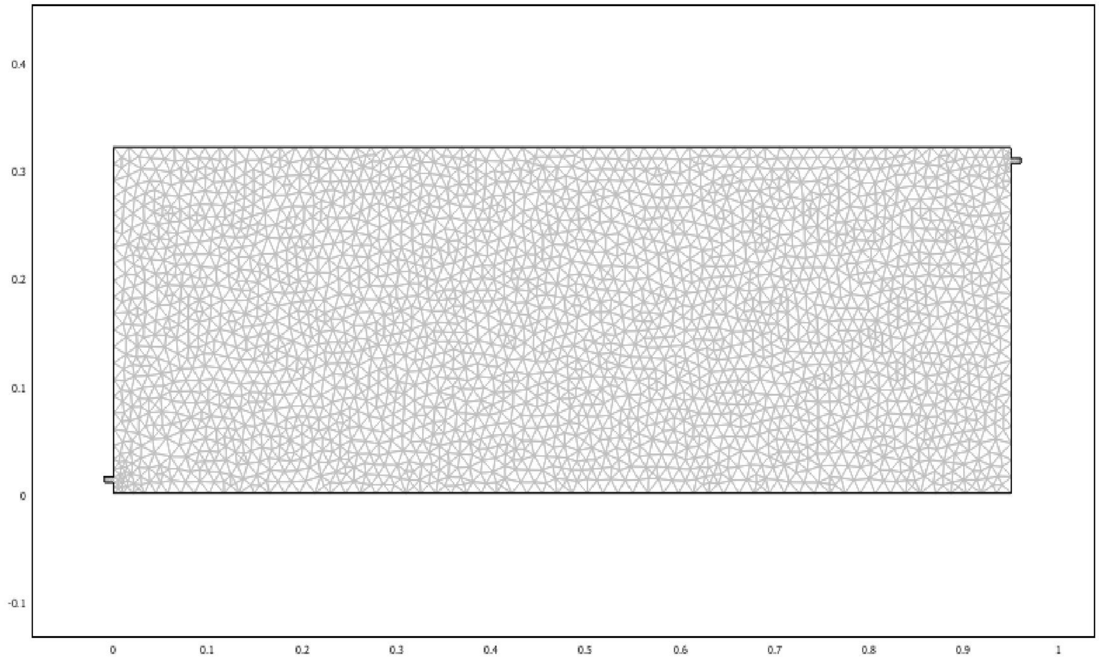

Figure 3.10 Triangular meshes for the model anaerobic reactor

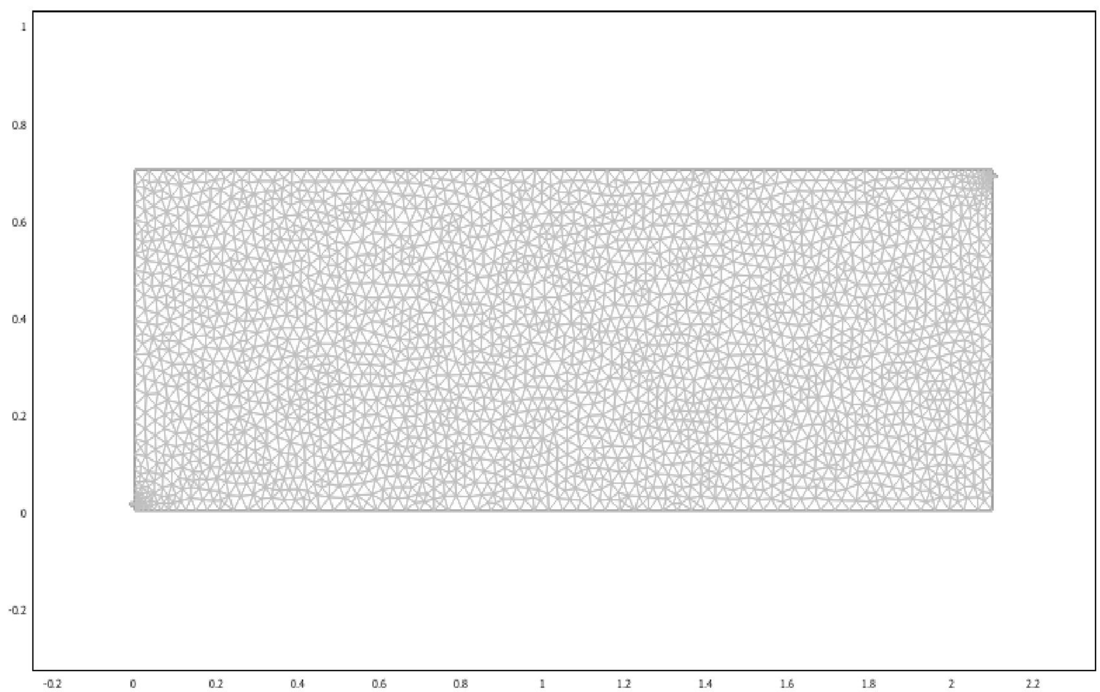

Figure 3.11 Triangular meshes for the model facultative reactor

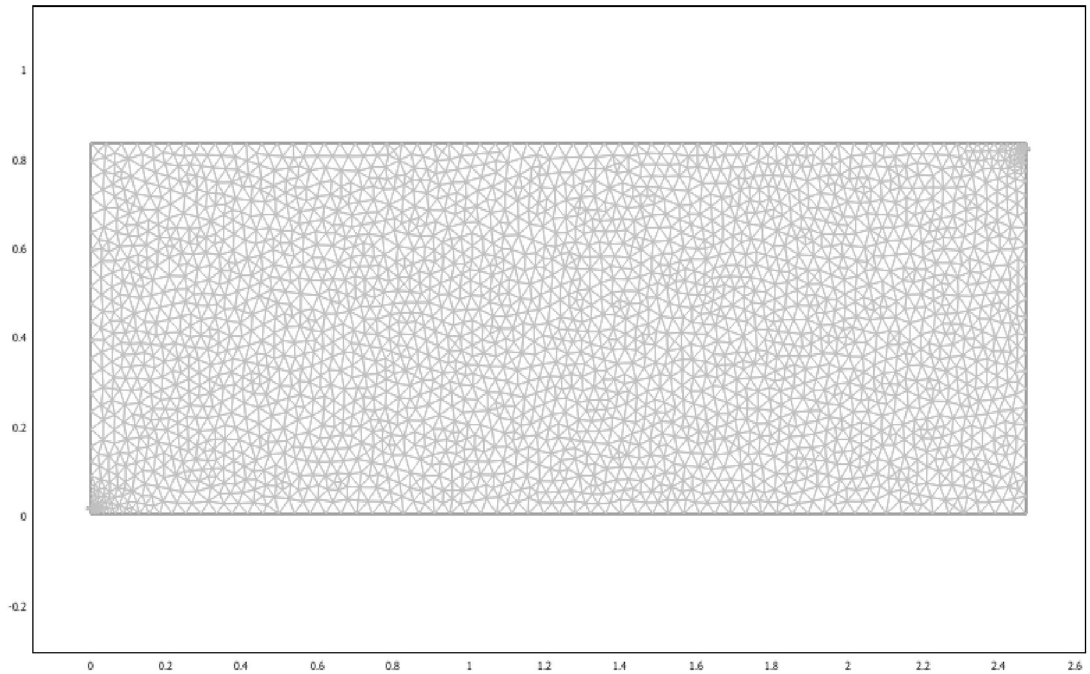

Figure 3.12 Triangular meshes for the model maturation reactor

Table 3.3 below presents the mesh statistics for the three reactors in series: anaerobic, facultative and maturation reactors respectively.

Table 3.3 Mesh statistics for the unbaffled laboratory-scale pond models

	Anaerobic	Facultative	Maturation
Number of degrees of freedom	44471	45511	46005
Number of mesh points	2659	2723	2753
Number of elements	5136	5252	5308
Number of boundary elements	180	192	196
Number of vertex elements	12	12	12
Minimum element quality	0.683	0.683	0.676
Element area ratio	0.011	0.003	0.002

Figure 3.13 describes the model navigator showing all the tools in the graphical user interface (GUI) in COMSOL Multiphysics. A typical representation of a four-baffled reactor has been shown in the geometry mode. Each tool on the GUI does a specific function that is specified by the user. After the mesh parameter has been set up, the model is ready to be run for any solution that has been specified.

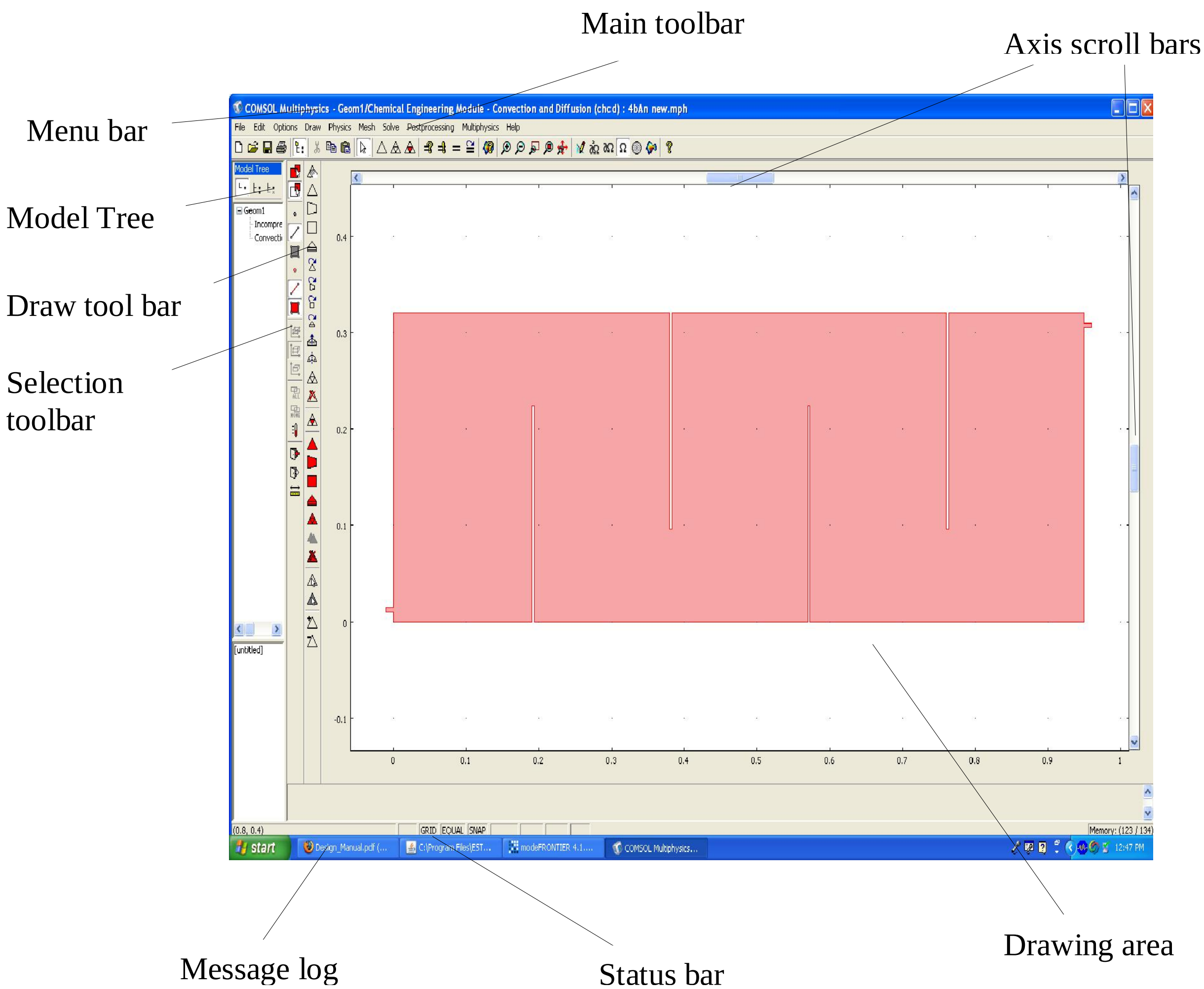

Figure 3.13 Model navigator showing the application modes and the Multiphysics

3.12.2.4 Model test for the simulation of residence time distribution curve in the CFD

The CFD was extended to determine the residence time distributions (RTD) in the laboratory-scale models by incorporating equation 3.18 into the second modified time dependent transient state transport equation in the application mode.

The transient equation was solved by using the converged current solution of the incompressible Navier-Stokes equation for velocity distribution. The solution converged and produced a residence time distribution curve in the CFD simulation as would be presented in results chapter. This demonstrates that the developed equation for the residence time distribution is correct and can be used in further CFD model simulations. Banda (2007) expressed that it could be very disturbing if the CFD simulation solution did not represent the theoretical residence time correctly. However, deviations could happen when there are errors in the model setup or in the solver applied in developing the solution. Besides, deviations can occur from transient turbulent structures that are not captured with two-equation turbulence models performed by many researchers as previously discussed in literature.

3.12.2.5 Model test for the simulation of faecal coliform inactivation in the unbaffled reactors

The three reactors in the experiment with dimensions of $(0.950\times0.320\times0.065)\ m^3$, $(2.1\times0.70\times0.045)\ m^3$ and $(2.470\times0.830\times0.040)\ m^3$ length, width and depth for anaerobic, facultative and maturation pond were simulated using the CFD model. The dimensions of the inlet and the outlet of the ponds were taken to be 0.005 m wide multiplied by the wastewater depth of each reactor. The steady state pattern in the model was achieved by treating the ponds surfaces as free slip to simulate the frictionless boundaries that exist between the free surface and the wastewater.

Isothermal condition was assumed to exist in the pond at a temperature of 24^0C. Marias (1974) first-order rate constant removal of faecal coliform ($2.6\times1.19^{(T-20)}$) was used for the unbaffled reactor while Banda (2007) first-order rate constant removal of faecal coliform ($4.55\times1.15^{(T-20)}$; where T is temperature) was included for the baffled reactor in the source term function together with the wastewater density at that temperature.

Banda’s (2007) equation for first order-rate constant of faecal coliform removal was adopted because the author`s investigation shows that the effluent faecal coliform counts were estimated closely when the first-order rate constant removal of faecal coliform was $4.55(1.19)^{T-20}$ day^{-1}. The correlation coefficient of 0.8267 was realized which predicted the observed effluent faecal coliform counts in baffled ponds accurately when used in the source term function. Figure 3.14 describes the correlation between the two sets of data. It has been found to be satisfactory in predicting the faecal coliform removal in a baffled WSP.

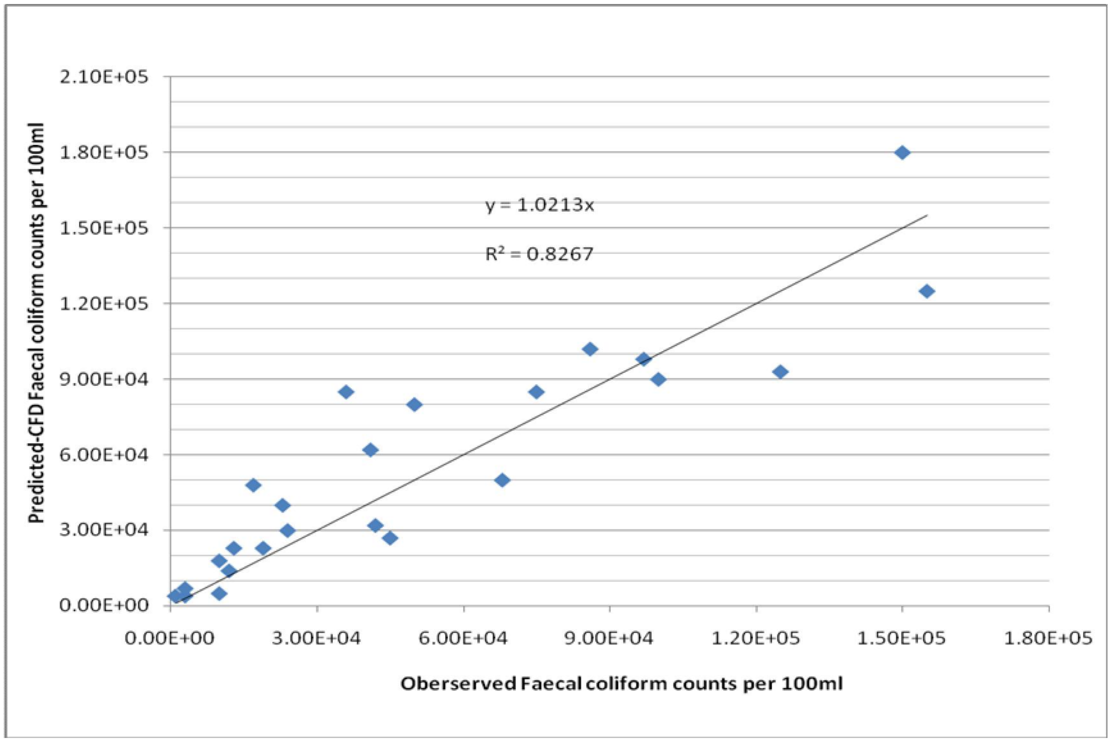

Figure 3.14 Correlation data of the predicted-CFD and observed effluent Faecal coliform counts in baffled pilot-scale ponds. Source: Banda (2007).

The standard case of the pond without baffles was simulated first to provide a basis against which the baffled designs could be evaluated. The hydraulic retention times of the ponds are 0.165 day, 0.563 day and 0.683 day which correspond to a flow rate of 0.12 m^3 per day. It is always good practice to evaluate the Reynolds number related to the specific flow conditions of the model, because its magnitude guides in choosing the appropriate flow model and corresponding application mode.

Using the Reynolds equation $Re = \frac{\rho v d}{\eta}$ 3.22

where:

ρ = density of wastewater = 997.38 kg/m^3, v = influent velocity = 4.27×10^{-3} m/s, 6.17×10^{-3} m/s and 6.94×10^{-3} m/s for anaerobic, facultative and maturation ponds respectively.

η = wastewater viscosity = 9.11×10^{-4} kg/m/s,

d = depth of the pond inlet = 0.065 m, 0.045 m and 0.040 m respectively

The Reynolds numbers at the inlets are approximately 304 for all the reactors.

The Reynolds numbers are well within the limits of the laminar flow regime. This suggests that the flow characteristic of the wastewater in the pond is laminar flow regime. The predicted faecal coliform count at the outlets was 5.53×10^{7}per 100 ml, 1.64×10^{7}per 100 ml and 0.44×10^{7}per 100 ml for anaerobic, facultative and maturation respectively. The effluent concentration of the anaerobic was used as the influent concentration into the facultative pond and the same was done for the maturation pond.

3.12.2.6 Model test for the simulation of faecal coliform inactivation in the baffled reactors.

A number of previous studies have concluded that inclusion of baffles in pond design gives better hydraulic efficiency. There is also a general belief that increasing the length to width ratio of a pond helps force its hydraulic behavior towards plug flow. It is with this knowledge that baffles were introduced and the effect of such introduction was verified. Baffle numbers were increased up to 6 (giving the length to width ratios approaching 21).

In this research, a series of models were tested using evenly spaced, 50%, 60%, 70%, 80% and 90% width baffles for the transverse arrangement and 60%, 70%, 80% and 90% width baffles for the longitudinal arrangement. The inlet and outlet structures were located at the diagonal corners of the reactor to follow the recommendations of geometric design procedures (Mara, 2004). The general flow pattern was studied and the treatment efficiency was quantified by integrating first order bacterial decay kinetics within the computer model. Figures 3.15-3.17 show the general longitudinal arrangement of conventional baffles of different lengths and numbers in each simulation that was undertaken. Where W is the flow channel width, L_b is the length of baffle and L_o is the baffle openings.

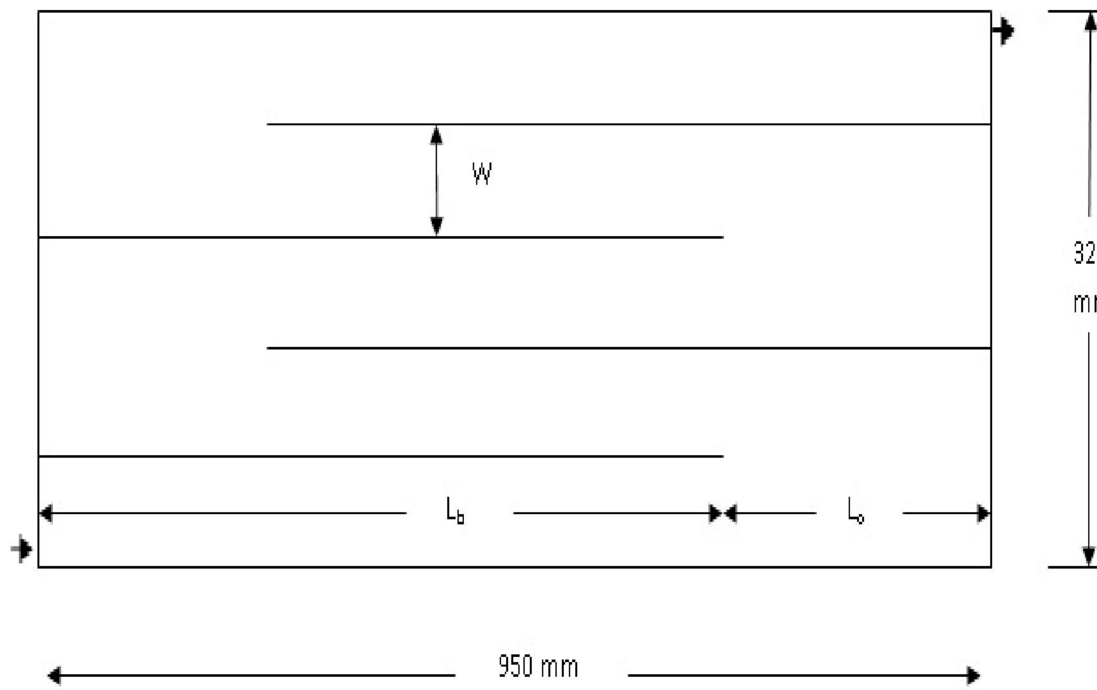

Figure 3.15 General arrangements of conventional longitudinal baffles of different lengths in the anaerobic reactor

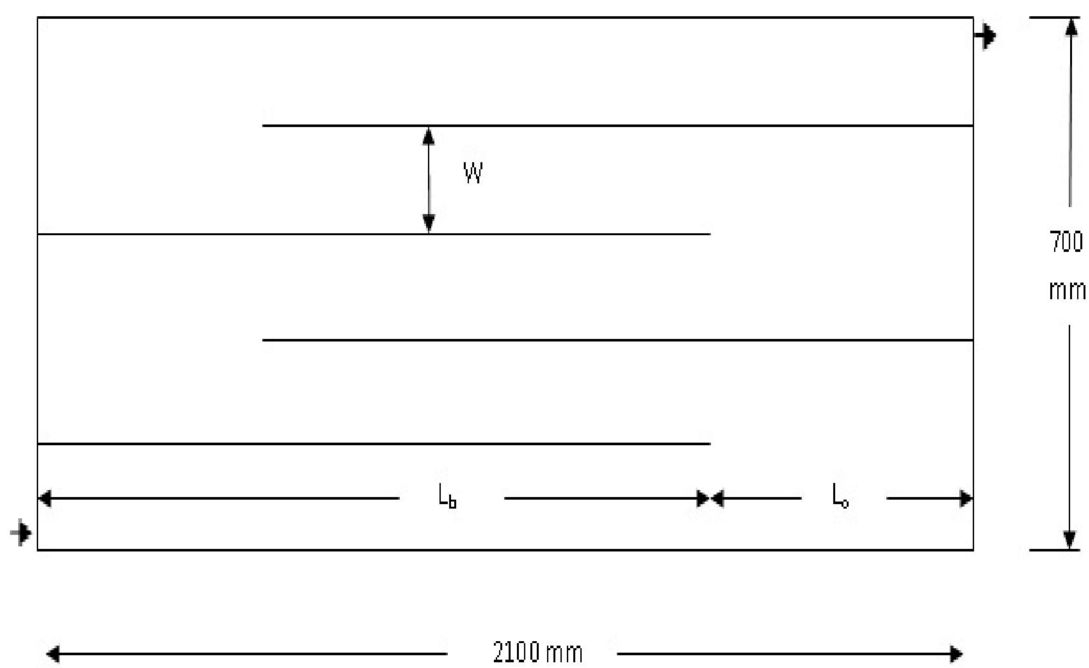

Figure 3.16 General arrangements of conventional longitudinal baffles of different lengths in the facultative reactor.

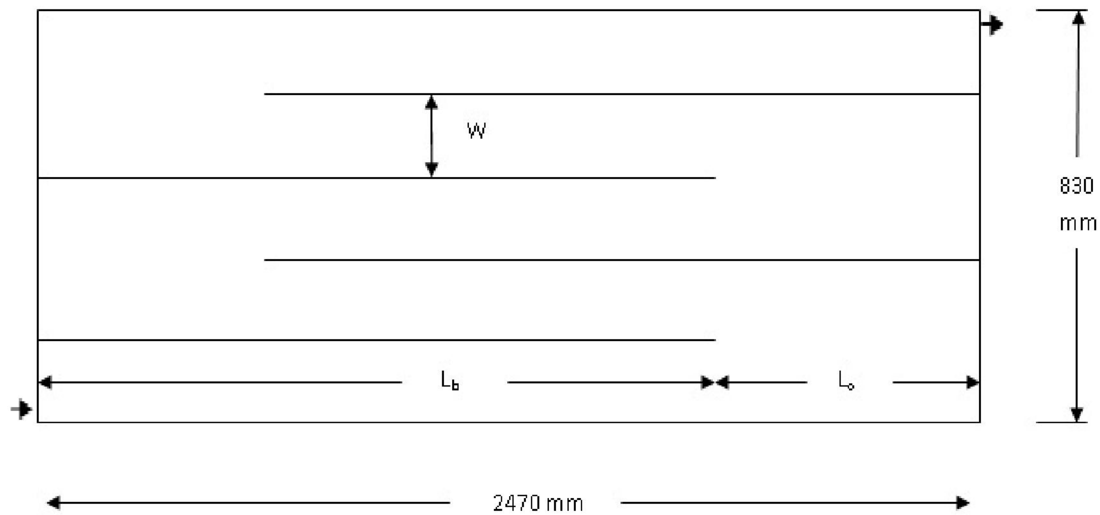

Figure 3.17 General arrangements of conventional longitudinal baffles of different lengths in the maturation reactor.

For uniformity with results of mesh dependent test for unbaffled pond model, the mesh size of 5 percent of each reactor width was used for all subsequent models of the baffled laboratory-scale models that were undertaken. Figures 3.18-3.23 describe the mesh layout while Table 3.4 shows the mesh statistics in the baffled laboratory-scale models.

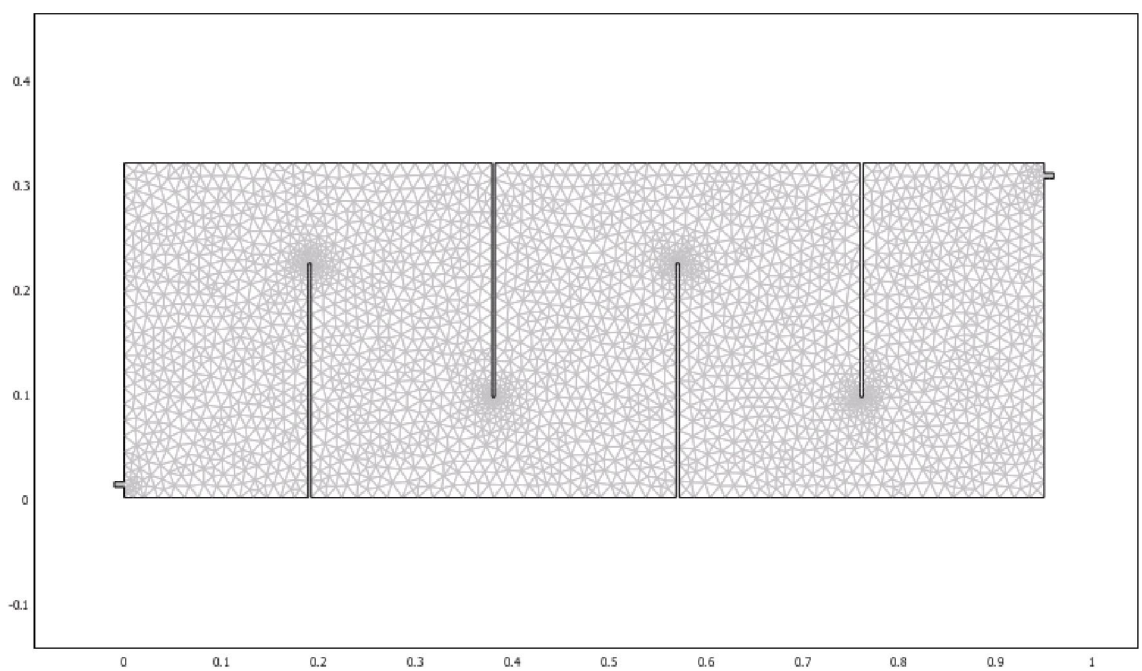

Figure 3.18 Mesh structure in a 4 baffled 70% Transverse Anaerobic reactor

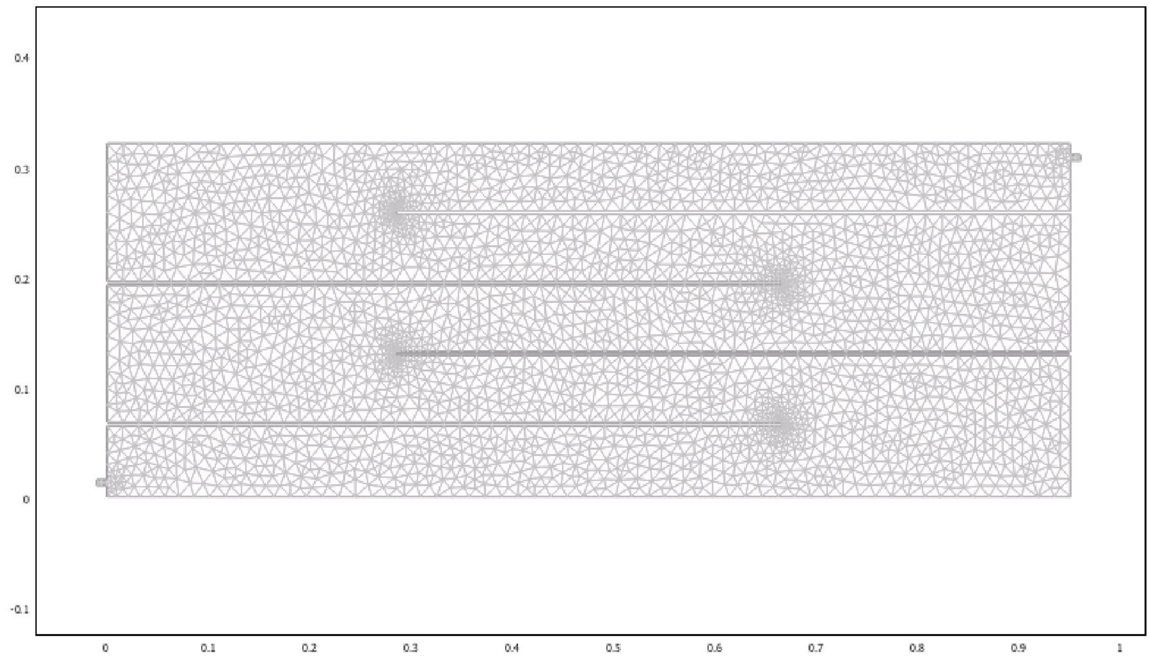

Figure 3.19 Mesh structure in a 4 baffled 70% Longitudinal Anaerobic reactor

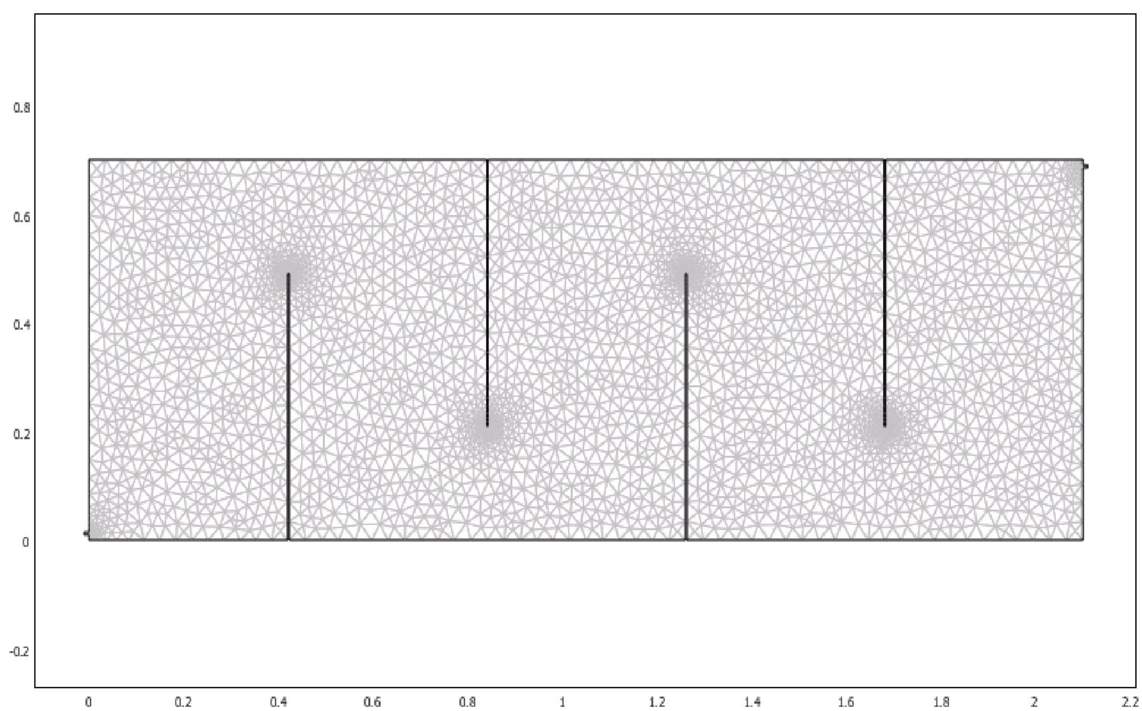

Figure 3.20 Mesh structure in a 4 baffled 70% Transverse Facultative reactor

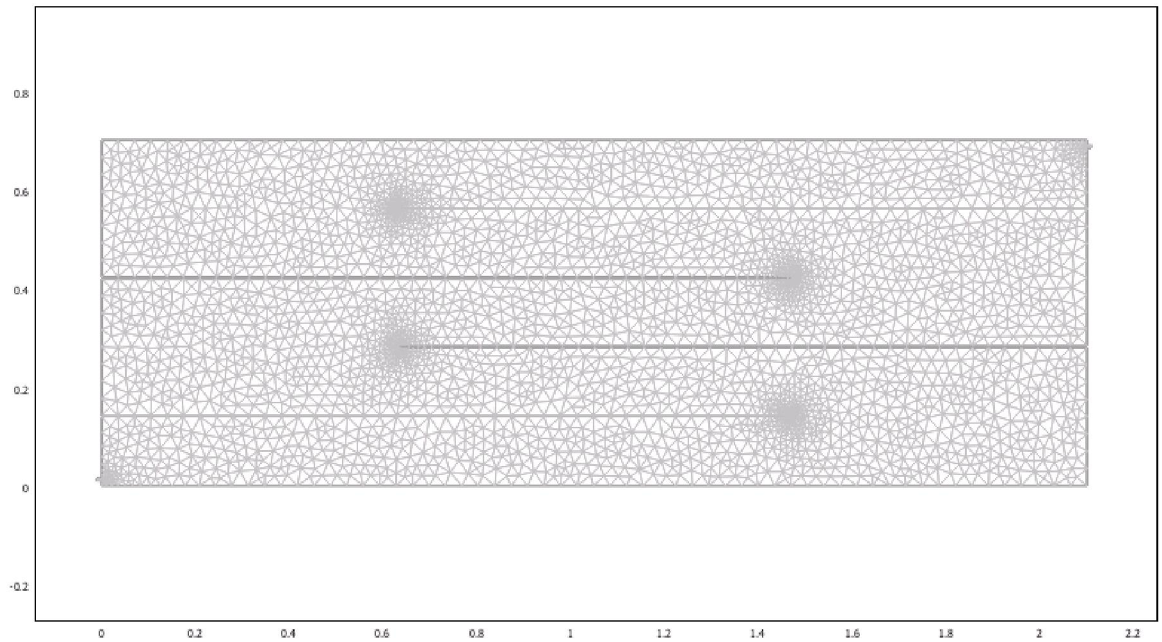

Figure 3.21 Mesh structure in a 4 baffled 70% Longitudinal Facultative reactor

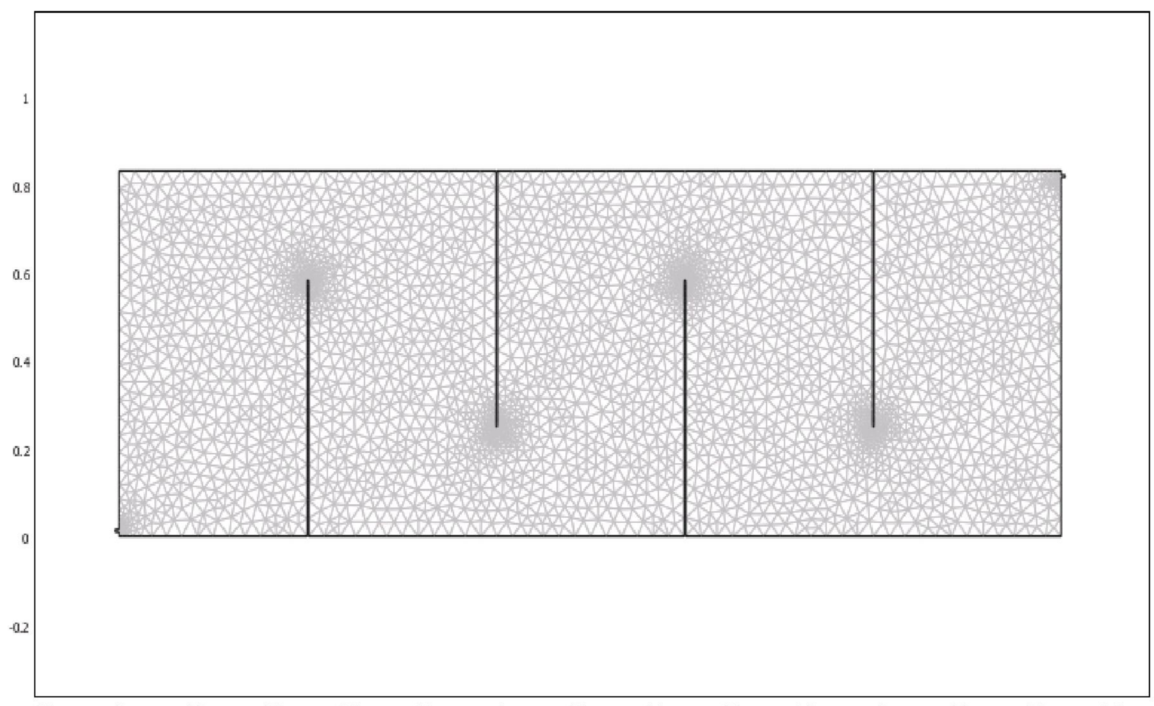

Figure 3.22 Mesh structure in a 4 baffled 70% Transverse Maturation reactor

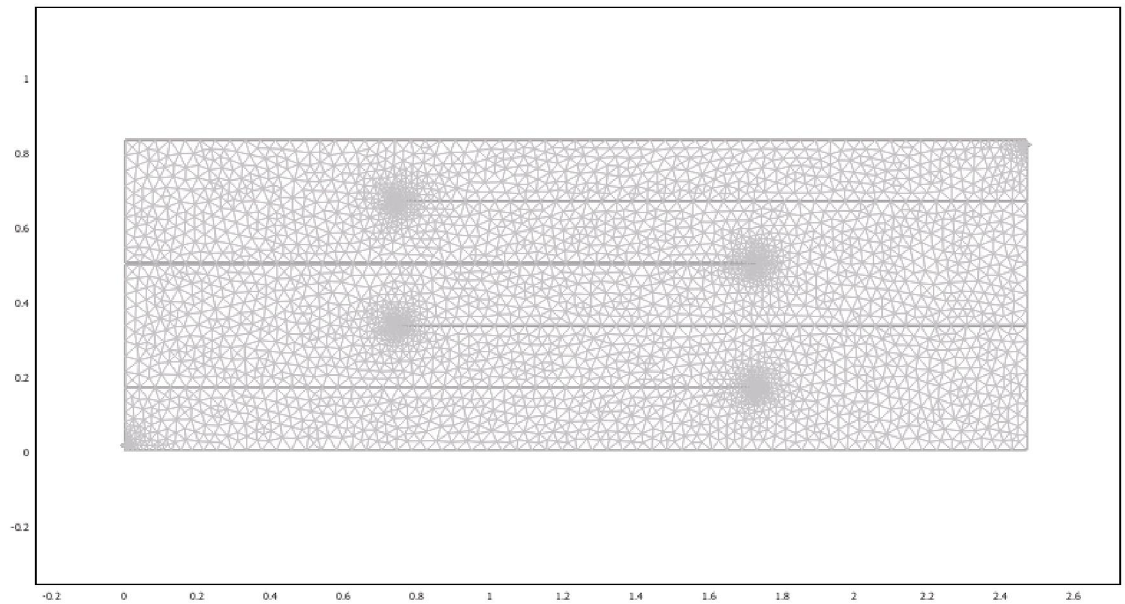

Figure 3.23 Mesh structure in a 4 baffled 70% Longitudinal Maturation

Table 3.4 Mesh statistics for laboratory-scale pond models with four baffles

	Anaerobic Transverse	Anaerobic Longitudinal	Facultative Transverse	Facultative Longitudinal	Maturation Transverse	Maturation Longitudinal
Number of degrees of freedom	48786	52509	56548	59841	56627	60913
Number of mesh points	2946	3221	3408	3661	3415	3725
Number of elements	5569	5881	6470	6724	6474	6848
Number of boundary elements	321	559	344	596	354	600
Number of vertex elements	28	28	28	28	28	28
Minimum element quality	0.652	0.683	0.683	0.683	0.683	0.675
Element area ratio	0.012	0.012	0.002	0.002	0.002	0.002

Following the recommendation of Banda (2007), the first-order rate constant removal of faecal coliform ($4.55 \times 1.19^{(T-20)}$; where T is temperature) was included for the baffled reactor in the source term function together with the wastewater density at that temperature. The predicted faecal coliform counts at the outlets are presented in Table 4.2 for anaerobic, facultative and maturation respectively. The effluent concentration of the anaerobic was used as the influent concentration into the facultative pond and the same was done for the maturation pond.

3.12.3 Application of segregated flow model to compare RTD prediction and the CFD predictions for feacal coliform reduction

The application of the segregated flow model to compare the RTD prediction and the CFD predictions for feacal coliform reduction has given a better insight about mixing and performance of the reactors. The residence time distribution (RTD) of the reactor is a characteristic of the mixing that occurs in the reactor. Each exit wastewater corresponds to a specific residence time in the reactor and batches of coliform are removed from the reactor at different locations along the reactor in a manner as to duplicate the RTD function. The coliform concentrations checked near the entrance to the reactors correspond to those having short residence times while the effluent corresponds to the coliform that channel through the reactor. The farther the coliform travel along the reactor before being removed, the longer there residence time and the points at which the various coliform batches are removed correspond to RTD function for the reactor.

In order to determine the mean conversion/degree of microbial inactivation in the effluent wastewater, the conversion of all coliform were averaged in the exit wastewater following (Fogler, 2006):

$$d\overline{X} = X(t) \times E(t)dt \tag{3.23}$$

where:

$d\overline{X}$ = Mean conversion/ degree of microbial inactivation of the wastewater spending time between t and dt in the reactor

$X(t)$ = Conversion achieved/ microbial survival ratio in the wastewater after spending a time t in the reactor

$E(t)dt =$ Fraction of wastewater that spends a given time between t and t+dt in the reactor

Equation 3.23 is further expressed as:

$$\frac{d\overline{X}}{dt} = X(t)E(t) \qquad 3.24$$

Summing over all the fluids, the mean conversion/degree of microbial inactivation is expressed as:

$$\overline{X} = \int_0^\infty X(t)E(t)dt \qquad 3.25$$

Since the residence time distribution and the reaction rate expression are available, there is sufficient information to calculate the conversion for the segregated flow model.

The conversion achieved or microbial survival ratio, $X(t)$, in the wastewater after spending a time t in the reactor can be expressed following (Fogler, 2006) as:

$$X(t) = 1 - e^{-kt} \qquad 3.26$$

For the assumption of a complete mix reactor, the flow at the inlet is completely and instantly mixed into the bulk of the reactor and the RTD function is given as:

$$E(t) = \frac{1}{\tau} e^{-t/\tau} \qquad 3.27$$

Substituting equation 3.26 into 3.25, the mean conversion for a first-order reaction is given as:

$$\overline{X} = \int_0^\infty (1 - e^{-kt}) E(t) dt \qquad 3.28$$

$$\overline{X} = \int_0^\infty E(t) dt - \int_o^\infty e^{-kt} E(t) dt$$

$$\overline{X} = 1 - \int_0^\infty e^{-kt} E(t) dt \qquad 3.29$$

Substituting equation 3.27 into 3.29, the mean conversion for a first-order reaction gives

$$\overline{X} = 1 - \int_0^\infty \frac{e^{-(1/\tau + k)t}}{\tau} dt \qquad 3.30$$

$$\overline{X} = 1 + \frac{1}{k + \frac{1}{\tau}} e^{-\left(k + \frac{1}{\tau}\right)t} \Bigg|_0^\infty$$

Finally, the mean conversion/ degree of microbial inactivation equals

$$\overline{X} = \frac{\tau k}{1 + \tau k} \qquad 3.31$$

where:

τ = Hydraulic retention time (HRT)

k = reaction rate constant (9.124 d^{-1})

3.12.4 Summary of the CFD model methodology

It has been demonstrated that CFD can be used appropriately in simulating the faecal coliform removal, velocity distribution and residence time distribution in the model of a laboratory–scale waste stabilization pond. The source term functions that represent the faecal coliform, velocity distribution and residence time distribution have been developed into a form consistent with the source term function for the mass and momentum transport equations in the CFD. Dimensional analysis has been utilized to ensure correct dimensions and units

The Reynolds number related to specific flow conditions of the model guided in choosing the flow model and the corresponding application mode while the transient equation was solved by using the converged current solution of the incompressible Navier-Stokes equation for velocity distribution. The solution converged and produced a residence time distribution curve in the CFD simulation. The use of 2D model in this research has allowed easy verification and validation of results because they are easier to generate and interpret.

3.13.1 Optimization methodology and application

ModeFRONTIER, an optimization tool that uses genetic algorithm (GA) has been identified and this has been used to solve many difficult engineering problems and is particularly effective for combinatorial optimization problems with large, complex search spaces. Within the reliability field, however to the best of the knowledge of the researcher, there has been no example of its use in wastewater treatment system design optimization to find a maximum reliability solution to satisfy specific cost objective. The goal of this research is to apply modeFRONTIER tool that uses genetic algorithm based on single and multi-objective optimization to efficiently optimize the selection of the best configuration that gives the minimum cost without compromising the treatment efficiency of the model laboratory-scale WSP. The two optimization objectives are to minimize the cost of construction material of the reactor and also to minimize the effluent faecal coliform in all the reactors. The optimal designs would be selected based on the outcome from the optimization algorithm.

3.13.1.1 Integration of Computational Fluid Dynamics tool with ModeFRONTIER optimization tool

Having established that the CFD model gave a good representation of the model laboratory-scale reactors, the purpose of this aspect of the research is to minimize cost of constructing the modeled reactors and achieving a target faecal coliform removal which is toward a plug flow pattern. This can be transformed or applied to a field scale prototype WSP construction. The output in .mfile of the reactor model that is based on a CFD methodology of solving the time dependent flow and transport equations on a 2D depth-integrated model was used as the input parameter in the optimization tool. The outcome is an optimized model geometries that can predict precisely the velocity distribution, residence time distribution and faecal coliform concentration at all points in the reactor of which the effluent and the cost that could be incurred is of utmost interest to the researcher.

The use of modeFRONTIER version 4.1.2 has allowed the running of the output from CFD modeled reactors. CFD model output was customized and saved as a .mfile, and all the information (relative to parameterization, run and post processing computation of output) remained in a ASCII file that was run in batch mode and used by modeFRONTIER for the optimization. Appendix B1-B6 describes in detail the .mfile model for longitudinal and transverse baffle arrangement that was attached to the input template editor for the three reactors in modeFRONTIER workflow canvas.

The software has a customized user interface and functionality for input of different nodes in the work flow environment. It consists of a number of nodes in the toolbar and in the panel which are divided in categories: Logic nodes used in defining the workflow logic actions, Variable nodes used in defining the input, output and transfer variables, Goals nodes used in defining the users defined goals (objectives and constraints) for the given problem, File nodes used in handling, transferring or extracting the files needed by the application nodes, Application nodes which define a list of specific process flow applications, Script nodes which define the most common script facilities, CAD nodes for defining specific nodes and the Networking nodes used for network based activities

3.13.1.2 The workflow pattern

The Workflow canvas was used to define the process and the data flow for each design evaluation. The process flow defines the chronological sequence of the optimizations used for the design while the data flow defines all the necessary actions for merging the input variables into the input template files and the mining operation to extract the output variables from the output files. During the optimization loop the workflow was run every time a design evaluation was needed. Every Workflow Pattern has a particular feature that can be used as the starting point for developing workflows for specific problems. Figure 3.24 describes the workflow pattern for the optimization in this research. This shows how to handle an external application that uses an input file and produces an output file containing the results.

The four design configuration types that were evaluated are even transverse, even longitudinal, odd transverse and odd longitudinal baffle arrangements to investigate the effects of these configurations on the effluent quality from the three reactors. The odd and longitudinal configurations were included because of their limited research from literature. The positions of the outlet in the odd configuration are placed on the same side with inlet for the longitudinal baffle arrangement. The inclusion of the odd transverse and longitudinal arrangements allow the total investigation into the entire scenario possible to achieve the optimal reactor configuration that could give the best wastewater treatment.

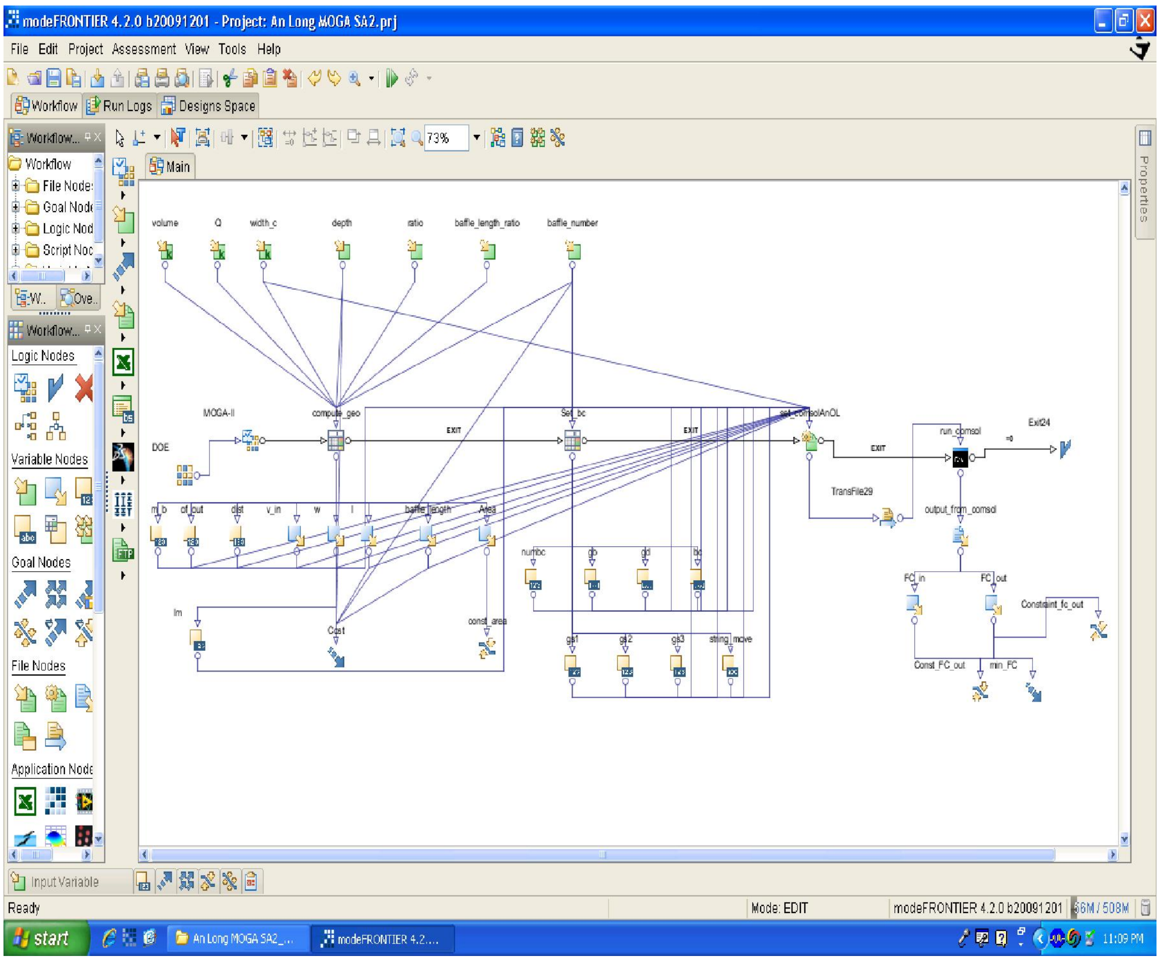

Figure 3.24 Workflow showing all links and nodes in the user application interface

3.13.1.3 Building the process flow

The configuration panel that displayed all the properties relative to the node and the connections between the nodes were inserted and configured. All other nodes linked to it were adjusted to suit the optimization objectives for this research. The building blocks for the workflow are two: nodes and links, with each link connecting two nodes. One starting and ending point was set up: The scheduler object as the starting point and the logic end as the end point which was connected to the application node.

The application exit condition was specified for the process output connector linked to the logic end node. This made the design to be considered successfully evaluated when the exit condition was zero. The application exit condition < > 0 was specified for the process output connector linked to the logic fail node. When the exit condition is not equal to zero, the evaluated designs would be marked as error in which the author did not have any such case. Figure 3.25 describes the logic end properties dialogue interface.

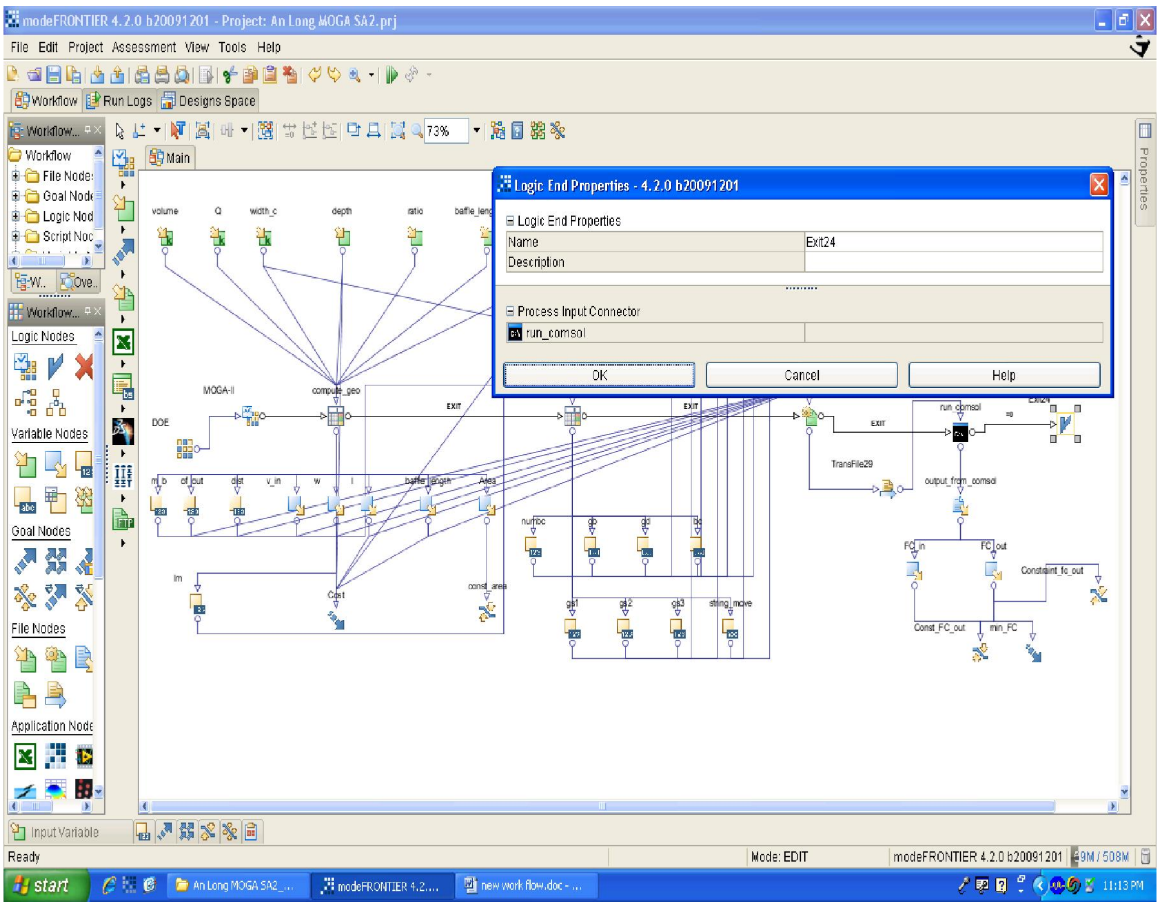

Figure 3.25 Logic End properties dialogue interface

3.13.1.4 Creating the application script

Two scripts that performed the external tasks were created in the JavaScript editor. These were created for the purpose of optimization for both transverse and longitudinal baffle arrangements. These scripts have an interactive processor for a language that resembles C but provides unlimited-precision arithmetic. It takes input from any files given and the application script has a set of simple actions that allows the loading, saving and editing of external scripts or batch files. Details of the developed scripts for both transverse and longitudinal baffle arrangements are presented in Appendix C1 and C2.

3.13.1.5 Creating the data flow

Seven input variables were created and configured in the workflow environment. The variables are reactor volume, wastewater flow rate (Q), flow width (width_c), depth, area ratio, baffle length ratio and baffle numbers. The first three were made constant while the latter four were specified to vary within the range that would give the best results. All of these nodes were connected by a link to the compute geometry node in the process flow. Figure 3.26 describes the links between all the variables and the compute geometry nodes on the process flow link.

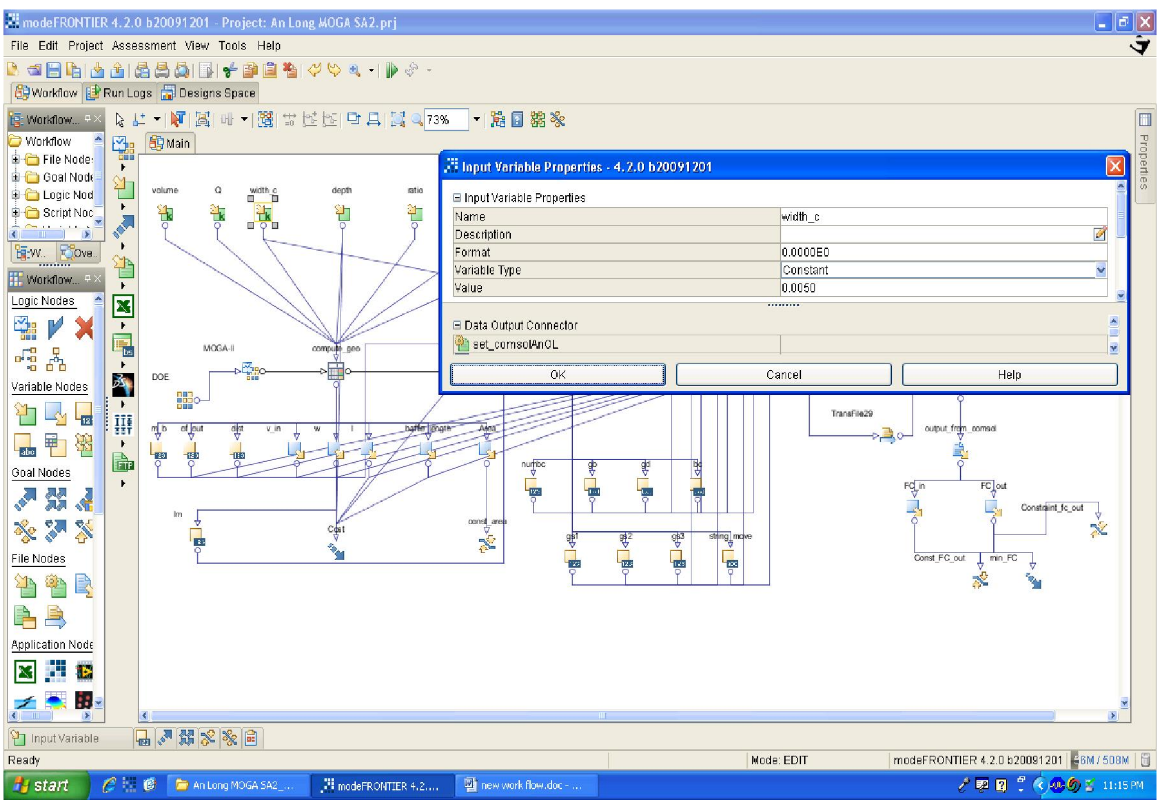

Figure 3.26 Data variable carrying nodes and the input variable properties dialogue interface

3.13.1.6 Creating the template input

The compute geometry node was developed to solve the equations that connected the variable nodes together. The outputs from the compute geometry node were linked to the input template properties node where the COMSOL batch files were run. Figure 3.27 shows the calculator properties and the JavaScript expression editor interface template. The JavaScript expression editor details the mathematical calculations involved in the design configuration within each reactor. The number of baffles, length of baffles and the dimensions of each reactor are developed by the expression.

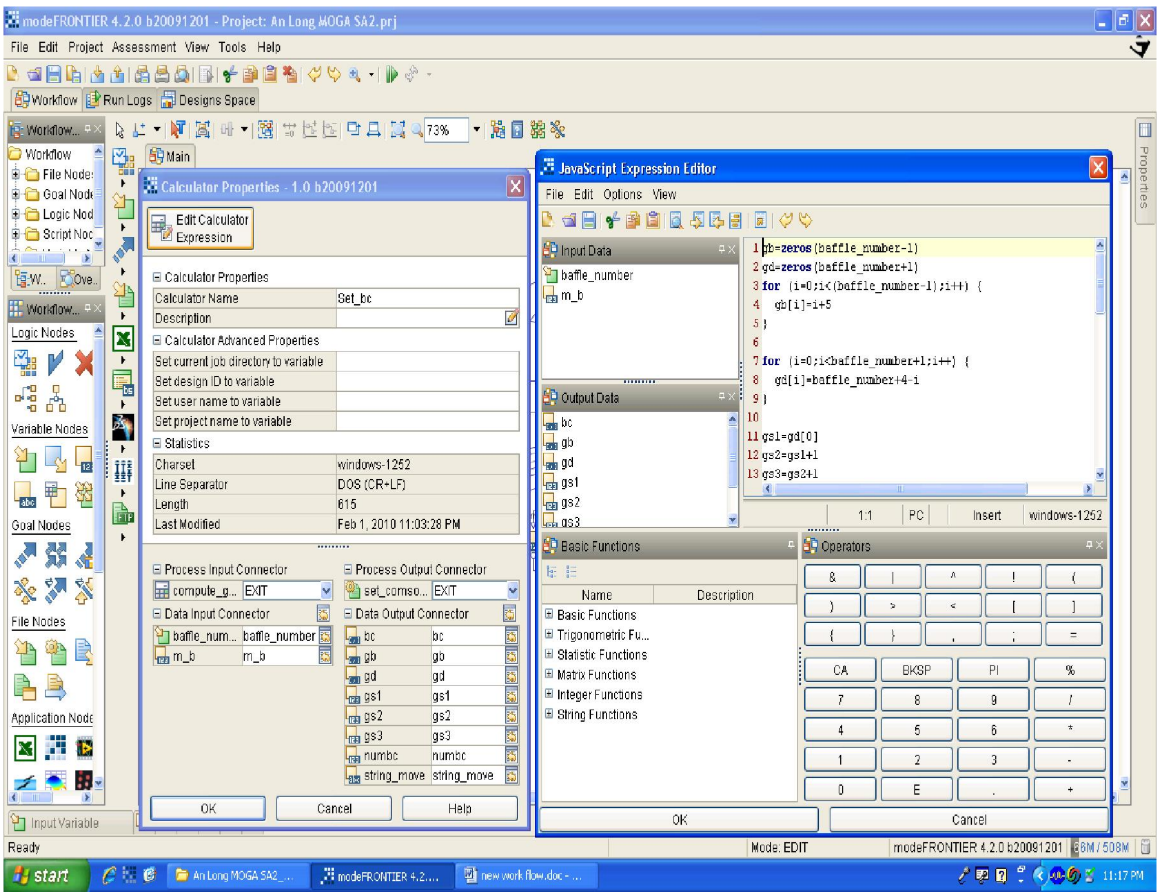

Figure 3.27 Template for the calculator properties and JavaScript expression editor

3.13.1.7 Mining the output variables from the output files

The Mining Rule is a rule which defines the way the output variable is extracted from the output file. In order to apply the mining rule to the output variable from COMSOL, files saved in .mfile from CFD model were selected (Appendix B1-B6). This serves as input into the modeFRONTIER project and all the variable parameters connected to the .mfile were ensured to be correctly configured before the program was run. With this arrangement, the workflow logic is completed, and a DOE based exploration of the design space can be performed. The variables are area, depth, and baffle number and baffle lengths. Figure 3.28 describes the input template properties and the editor for mining the output variables that were saved as both ASCII file and .mfile while Figure 3.29 shows the DOS batch test dialogue interface for mining data

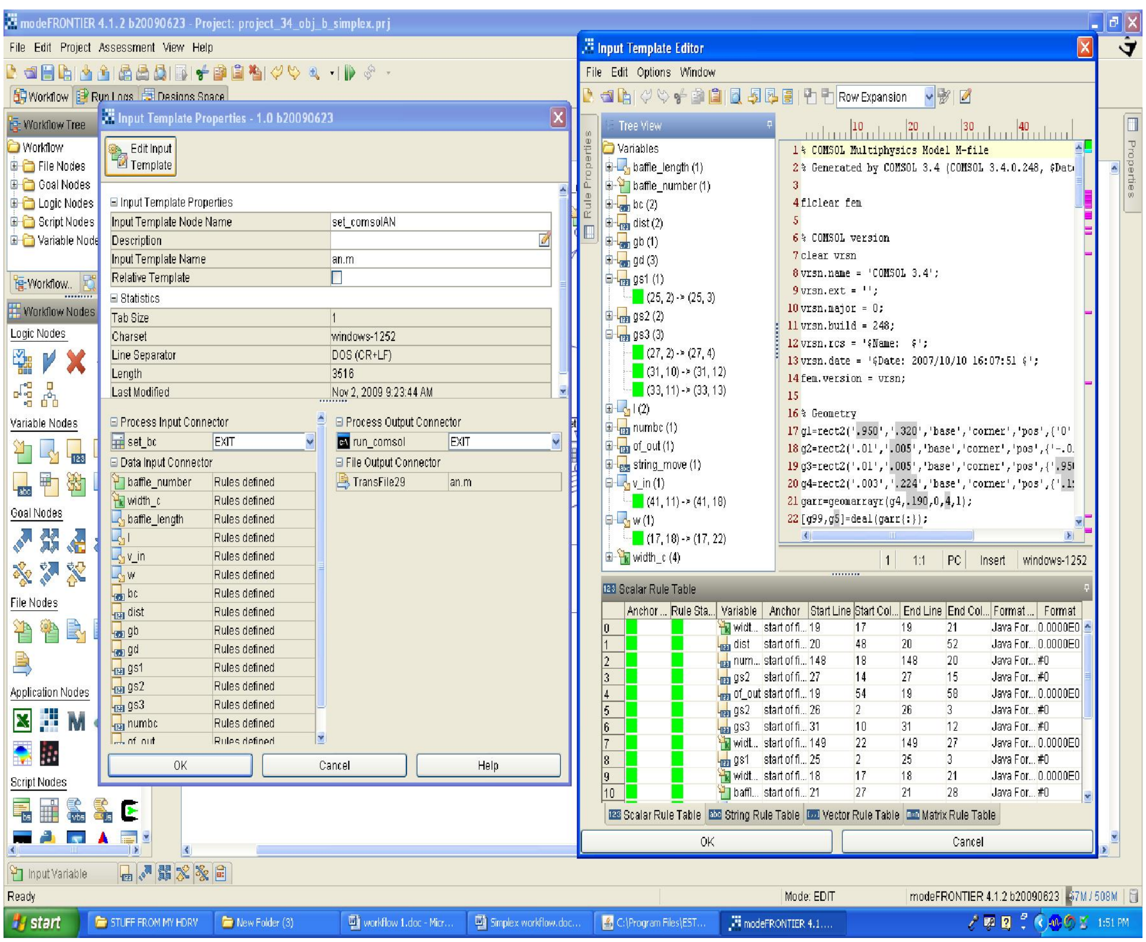

Figure 3.28 Output variable mining interface and input template editor

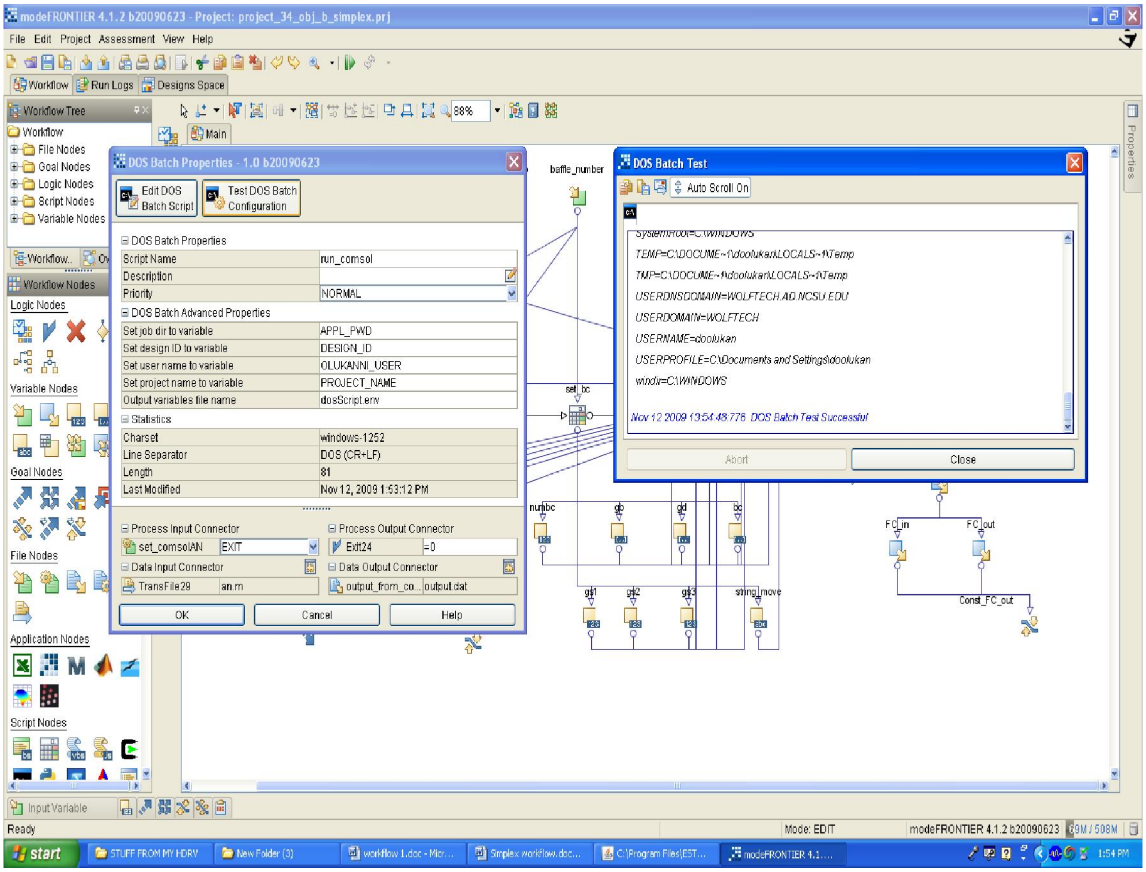

Figure 3.29 DOS Batch properties and batch test editor for mined data

3.13.2 Defining the goals

Two objectives were defined during the optimization loop in order for the DOE and the scheduler set-up to be completed. The objectives to be achieved were to minimize construction material cost and the faecal coliform in the effluent from the reactors. Some constraints were specified in order to meet the authors target hydraulic flow pattern in this research. The two objectives nodes were linked to the output variables. These objectives could be edited by using the properties dialog.

3.13.2.1 The Objective functions for the optimization loop

1. Minimize the cost of construction material of the Laboratory-Scale reactors based on length, width and depth including the baffles since they are all made of the same material. (The least cost that gives the optimal treatment within the range specified).

2. Minimize the values of the effluent (FC out) that comes out of the reactor. The lower the value of the effluent, the better it is and thereby linking it to the cost based on all the parameters included in the variables.

3.13.2.2 Constraints for the optimization loop

1. The area of the pond which should be at a minimum range specified as $A<=A_0$
2. The number of baffles/ baffles configurations to be installed within the reactors, looking at a range of baffles between 2 to 6 (2<= B <=8) for the transverse arrangement and 2 to 7 for the longitudinal arrangement.
3. The depth of the reactors which translates to the velocity of wastewater into the reactors. The depth has a range for both anaerobic, facultative and maturation reactors and was specified as $h_{min}<= h <=h_{max}$ according to literature design specification.
4. The efficiency of the ponds in terms of coliform removal in effluent are specified as FC log kill > = 0.6, 1.5 and 1.5 log removal in the anaerobic, facultative and maturation reactors respectively to achieve a treatment close to plug flow pattern.

3.13.2.3 Cost objective Optimization

The purpose of this aspect is to get the best reactor combination that will give the maximum treatment performance with minimum cost of reactor construction.

A metal sheet of size (4ft by 8ft) for constructing the reactor cost N8, 000.

If 4ft = 1.2192 m, then the metal sheet area is (1.2192m by 2.4384m).

Then the cost per unit area will be 8000 / (2438.4×1219.2) = N 0.002691 / mm^2.

Per (mm) square plate is approximately N0.003 which is N3, 000 per (m) square plate.

So any size/quantity of material used for any of the reactor can be quantified to give the objective function.

The sizes of the reactors are length to width (950 by 320) mm^2, (2100 by 700) mm^2 and (2470 by 830) mm^2, anaerobic, facultative and maturation respectively.

The heights/depths of water level are 65mm, 45mm and 40mm respectively which was used in velocity calculation. Any change in the depth will reflect in the velocity.

The flow rate of $0.12 m^3/day = 1.39\times10^{-6} m^3/s$ was used for the running of the models

So the velocity into the first reactor is Q = AV.

Velocity = Q/A = $1.39\times10^{-6} / (0.005\times0.065) = 4.278\times10^{-3}$ m/s into Anaerobic reactor

The depth of Anaerobic reactor varies between 0.048 <= d <= 0.12

Q/A= $1.39\times10^{-6} / (0.005 \times 0.045) = 6.17\times10^{-3}$ m/s into Facultative reactor

The depth of Facultative reactor varies between 0.024 <= d <= 0.048 while maturation depth was varied as 0.024 <= d <= 0.040.

Q/A= velocity = $1.39\times10^{-6} / (0.005 \times 0.040) = 6.94\times10^{-3}$ m/s into Maturation reactor

The user expression on the cost objective properties was expressed as:

3000 × (2 × depth × length + 2 × depth × width + length × width + baffle number × baffle length× depth) in order to determine each model design cost for the optimization.

The cost estimate is solely for the material and it excludes labor and other costs associated with constructing the ponds. Other associated cost could be estimated based on a unit area of the plate used for each pond type construction.

Table 3.5 displays the geometric and dynamic parameters that were adjusted during the simulation. The pond length to width (L/W) ratio was allowed to increase from 1:1 to 1:4. This L/W ratio was based on the limitation of land availability. The optimization model simulations were also performed for different baffle length ratio between 5% and 95% as well as manipulating the number of baffles from 0 to 8.

Table 3.5 Range of adjusted parameter values

Parameters	Anaerobic	Facultative	Maturation
Volume	$0.0197 m^3$	$0.0662 m^3$	$0.0820 m^3$
Flow Rate (constant)	$1.39\times10^{-6} m^3/s$	$1.39\times10^{-6} m^3/s$	$1.39\times10^{-6} m^3/s$
Reactor L/W ratio (r)	1:4	1:4	1:4
Baffle length ratio(L_b)	5% - 95%	5% - 95%	5% - 95%
Baffle number (bn)	0 - 8	0 - 8	0 - 8
Depth (h)	0.048m – 0.12m	0.024m – 0.048m	0.024m – 0.040m

Figures 3.30 and 3.31 describe the constraint and the objective properties dialog respectively.

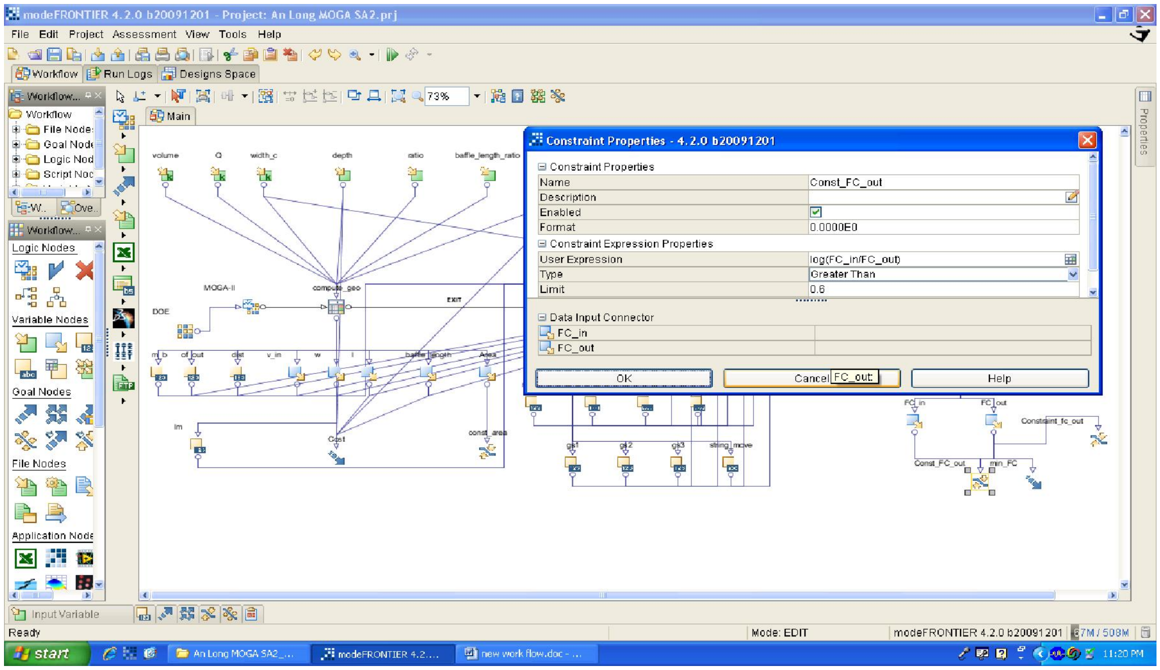

Figure 3.30 Constraint properties dialogue in the workflow canvas

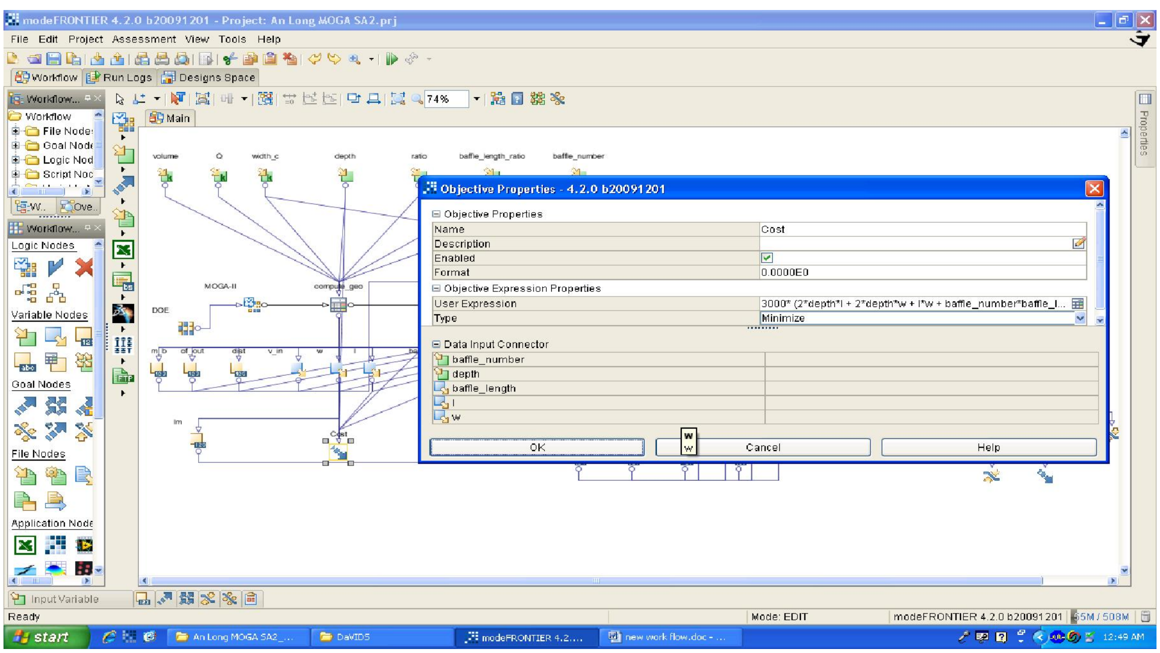

Figure 3.31 Objective properties dialogue in the workflow canvas

3.13.2.4 The DOE and scheduler nodes set up

The use of Design of Experiments (DOE) is extremely important in experimental settings to identify which input variables most affect the experiment being run (modeFRONTIER user manual, 2009). DOE is a methodology that maximizes the knowledge gained from experimental data. It provides a strong tool to design and analyze experiments which eliminates redundant observations and reduces the time and resources to make the experiments. Hence, DOE techniques allow the user to try to extract as much information as possible from a limited number of test runs.

Design of Experiments (DOE) is generally used in two ways. First of all, the use of DOE is extremely important in experimental settings to identify which input variables most affect the experiment being run. Since it is frequently not feasible in a multi-variable problem to test all combinations of input parameters, two optimization algorithms were used which provided an initial population of designs from which the algorithm learned. The DOE was used to provide the initial data point. Figures 3.32 and 3.33 show the DOE properties dialogue box and scheduler properties box respectively.

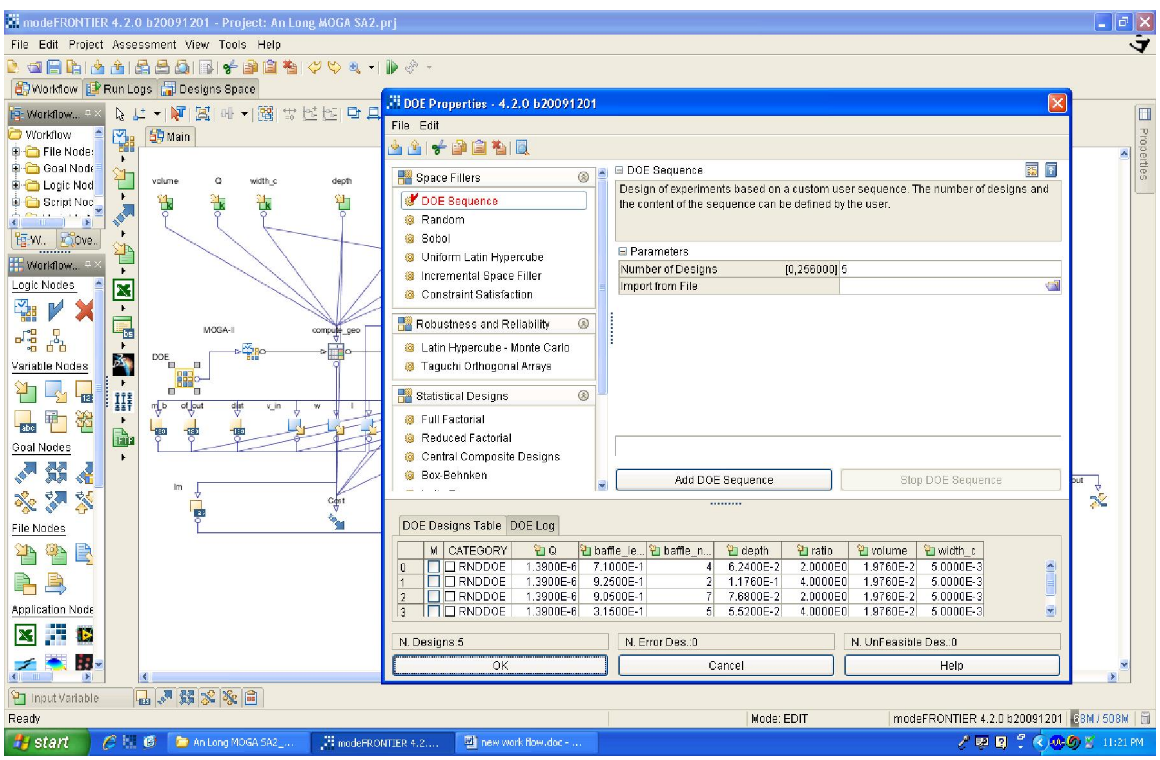

Fig 3.32 DOE properties dialog showing the initial population of designs

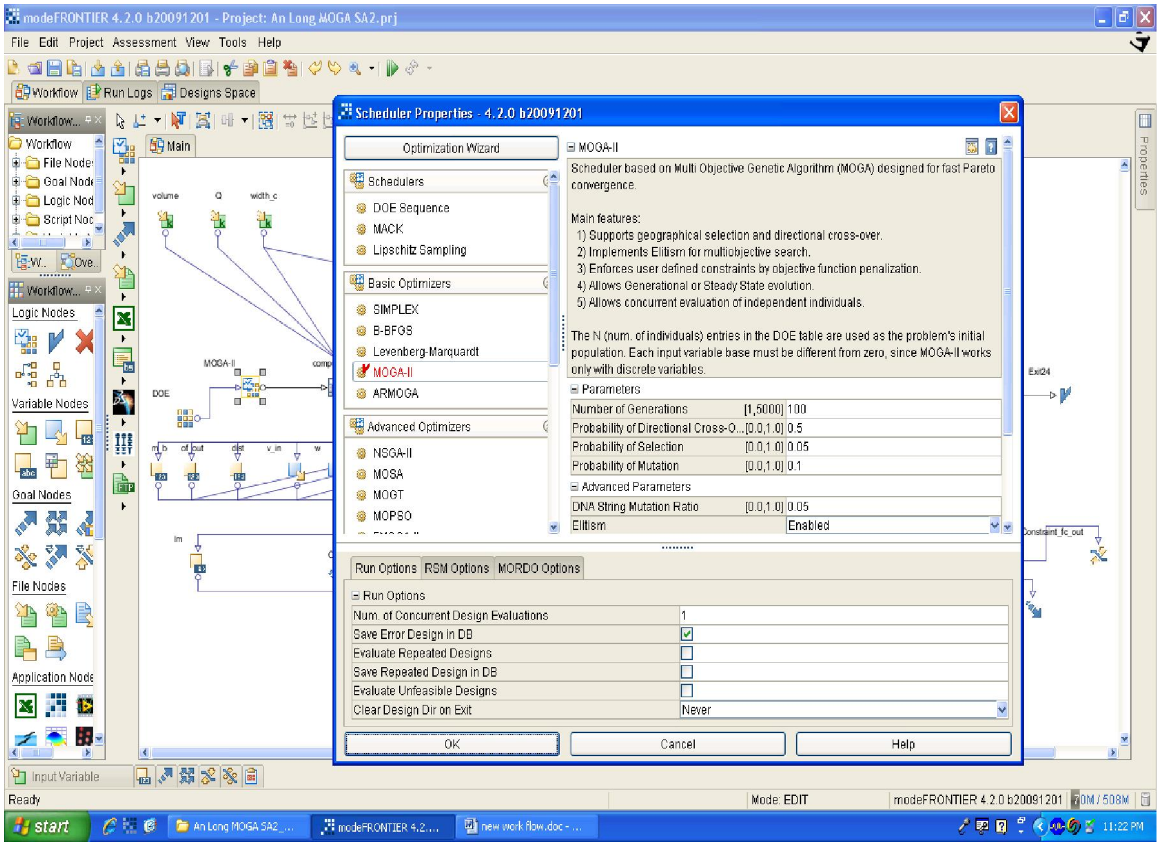

Figure 3.33 Scheduler properties dialog showing optimization wizards

3.13.2.5 Model parameterization of input variables

In order to determine the faecal coliform decay within the reactors, an assumption of a dimensionless concentration was made so that a faster optimization operation within the modeFRONTIER tool can be achieved. Therefore, the value of 1×10^{8} was expressed as a dimensionless term and the log removal of coliform was determined for each reactor. Equation 3.29 expresses the inactivation of faecal concentration in the reactor.

Using the expression $\dfrac{dc}{dt} = -kc$ 3.30

where:

dc/dt = rate of change in concentration with time

-k = the rate constant

c = concentration of wastewater

The expression for dimensionless concentration was expressed as:

$$c_1 = \frac{c}{c_o} \qquad 3.31$$

where:

c_1 = Dimensionless concentration

c = Concentration of effluent

c_o = Concentration of influent

Equation 3.30 was further expressed as

$$c = c_1 c_o \qquad 3.32$$

Substituting equation 3.32 into 3.30 gives the expression in equation 3.33 as:

$$\frac{d(c_1 c_o)}{dt} = -kc_1 c_o \qquad 3.33$$

Since the derivative of a constant is zero, then

$$\frac{c_o d(c_1)}{dt} = -kc_1 c_o$$

where:

c_o on the RHS and LHS cancels each other

Therefore, finally have $\frac{dc_1}{dt} = -kc_1$ 3.34

Equation 3.34 represents the first order faecal coliform inactivation used in the optimization process. With this expression, a unit value 1 for the inlet concentration of the faecal coliform was specified so that the total faecal coliform log removal in all the reactors can be estimated.

Equation 3.35 gives the expression for the faecal coliform log kill as:

$$\log kill = -\log\left(\frac{c}{c_O}\right) \qquad 3.35$$

where:

c_o = influent concentration and

c = effluent concentration.

The principal aim of the optimization is to design towards a plug flow pattern

Equation 3.36 expresses the log removal in a reactor for plug flow pattern as:

$$\frac{c}{c_O} = e^{-kt} \tag{3.36}$$

where: t = 0.165 (day) in the anaerobic reactor,

$$\frac{c}{c_O} = e^{-9.124(0.165)}$$

Therefore, we have 0.65 log removal in the anaerobic reactor

where t = 0.551(day) in the facultative reactor,

$$\frac{c}{c_O} = e^{-9.124(0.551)}$$

Therefore, we have 2.184 log removal in the facultative reactor

where t = 0.683 (day) in the maturation reactor,

$$\frac{c}{c_O} = e^{-9.124(0.683)}$$

Therefore, we have 2.706 log removal in the maturation reactor.

Equation 3.37 gives the expression for log removals in case of complete mix reactor as

$$\frac{c}{c_o} = \frac{1}{1+kt} \tag{3.37}$$

where t = 0.165 (day) in anaerobic reactor

$$\frac{c}{c_o} = \frac{1}{1+9.124(0.165)}$$

Log removal in anaerobic reactor is 0.40

where t = 0.551(day) in facultative reactor,

$$\frac{c}{c_o} = \frac{1}{1+9.124(0.551)}$$

Log removal in the facultative reactor is 0.78

where t = 0.683 (day)

$$\frac{c}{c_o} = \frac{1}{1 + 9.124 \ (0.683 \ )}$$

Log removal in the maturation reactor is 0.859.

These results indicate that the highest removal of faecal coliform takes place in the maturation pond. More so, it shows that plug flow pattern gives better faecal coliform removal in all the reactors. The minimum values specified for the optimization objective are set of values in between the plug flow and the completely mixed patterns. This serves as a base towards designing for a plug flow which has been identified to be the best condition for wastewater treatment system design. The values used were; 0.6, 1.5 and 1.5 for anaerobic, facultative and maturation reactors respectively.

3.13.2.6 DOE Algorithm

ModeFRONTIER provides the user with a wide selection of DOE algorithms for sampling the design space. The advantages of using modeFRONTIER's design of experiments includes smart exploration of the design space, time and money saving for experiments, check for robust solution, identify sources of variation and provide models of the problem which helps in making better decisions. Two Schedulers were used in the optimization process: The single objective SIMPLEX and Multiobjective Algorithm (MOGA-II)

3.13.2.7 Simplex algorithm

This is a scheduler based on a modified single objective SIMPLEX algorithm updated to take into account discrete variables and constraints. The algorithm is commonly used for multidimensional minimization problems (mode-FRONTIER Manual, 2009). The simplex scheduler was initialized by 5 variables. This gave the first design configurations in the DOE table. The selection of this algorithm allowed the assigning of the cost minimizing objective into the workflow project. The scheduler follows an algorithm for moving the initial points along with their function values, closer to the optimal point of the objective. The points are moved toward the optimal point until the scheduler exceeds its maximum number of iterations or converges. The movement of the simplex is given by three operations: reflection, expansion and contraction. The new candidate designs produced by the simplex algorithm are rounded to the nearest discrete values defined by the base value given to each variable.

Maximum number of iterations specified was 100 × 5 while final termination accuracy and the constraint penalty were specified based on the recommendation from the manufacturer and expert in the use of modeling tool. The algorithm always stops when the maximum number of designs has been evaluated. This was done independently from any other convergence limits. Default value of 1.0E-5 was specified for the final termination accuracy and an automatic constraint penalty policy was specified for the scheduler. The automatic checkbox allowed the scheduler to choose the most appropriate penalty value for the search algorithm. The algorithm stopped when it could not find solution with better improvements than the convergence default value of 1.0E-5. The results of the optimized, minimum and maximum faecal coliform removal designs are presented in the results chapter.

3.13.2.8 Multi-Objective Genetic Algorithm II (MOGA-II)

The selection of MOGA-II multi-objective algorithm was done in order to compare the results of the two algorithms. MOGA-II is an efficient multi-objective genetic algorithm (MOGA) that uses a smart multi-search elitism for robustness and directional crossover for fast convergence. This new elitism operator is able to preserve some excellent solutions without bringing premature convergence to local-optimal frontiers (modeFRONTIER Manual, 2009). The algorithm requires only very few user-provided parameters while several other parameters are internally settled in order to provide robustness and efficiency to the optimizer. The algorithm attempts a total number of evaluations that is equal to the number of points in the design of experiment table (the initial population) multiplied by the number of generations.

The number of generation, probability of directional cross-over, probability of selection, probability of mutation, elitism, treat constraints, algorithm type, and the random generated seed were all specified in the scheduler (modeFRONTIER Manual, 2009). Table 3.6 describes in detail the main features of MOGA-II scheduler for the optimization loop used in this research. Other main features that make MOGA-II to be considered is its support for geographical selection and directional cross-over and enforcement of user defined constraints by objective function penalization which allows steady state evolution.

Table 3.6 Scheduler based on Multi Objective Algorithm (MOGA-II) design table

The N (number of individuals) entries in the DOE table are used as the problem`s initial population		
	-Parameters	Specifications
1	Number of Generations	100
2	Probability of Directional Cross-Over	0.5
3	Probability of Selection	0.05
4	Probability of Mutation	0.1
	-Advanced parameters	
5	DNA String Mutation Ratio	0.05
6	Elitism	Enabled
7	Treat Constraints	Penalizing Objectives
8	Algorithm Type	MOGA-Generational Evolution
9	Random Generator Seed	1

3.13.2.9 Faecal coliform log-removal for transverse and longitudinal baffle arrangements

The optimization tool was set up such that the area ratio was manipulated by increasing the ratio between rectangular width and length as one, two, three and four times respectively. The model was also run for different rectangular shapes with baffle length ratio between 5% and 95%. Also included in the set up was the number of baffles which ranges from 2 to 8 including odd and even baffle configurations. The depth was set up to take variable values based on design specifications as evidenced in previous research works.

This exercise was carried out to investigate whether the variation of all the parameters could be utilized to improve the hydraulic performance and treatment efficiency of waste stabilization pond since they all represent the dynamics involved in the treatment efficiency of a reactor. The outcome of this exercise shows that the variable parameters could all be used to optimize the cost of the construction material for any reactor configuration without compromising the treatment efficiency. For the transverse design arrangement from COMSOL model with 4 baffles at a baffle length of 70% pond width, the log removal was 0.611, 1.489 and 1.706 for anaerobic, facultative and maturation reactors respectively. Also for the longitudinal design arrangement with 4 baffles at a baffle length of 70% pond length, the log removal was 0.51, 1.496 and 1.807 anaerobic, facultative and maturation reactors respectively.

Figures 3.34 and 3.35 describe the designs generated by modeFRONTIER from which the optimized, minimum and maximum faecal removal designs were selected for the purpose of comparison and evaluation of the history cost on design chart. The optimized design gives the least cost with the optimal treatment efficiency while the maximum faecal coliform removal designs in many cases give the highest cost of the treatment system construction material. It will be the decision of the designer to choose among various factors that could necessitate the selection of any of the designs. The results of all the selected designs are presented in chapter 4.

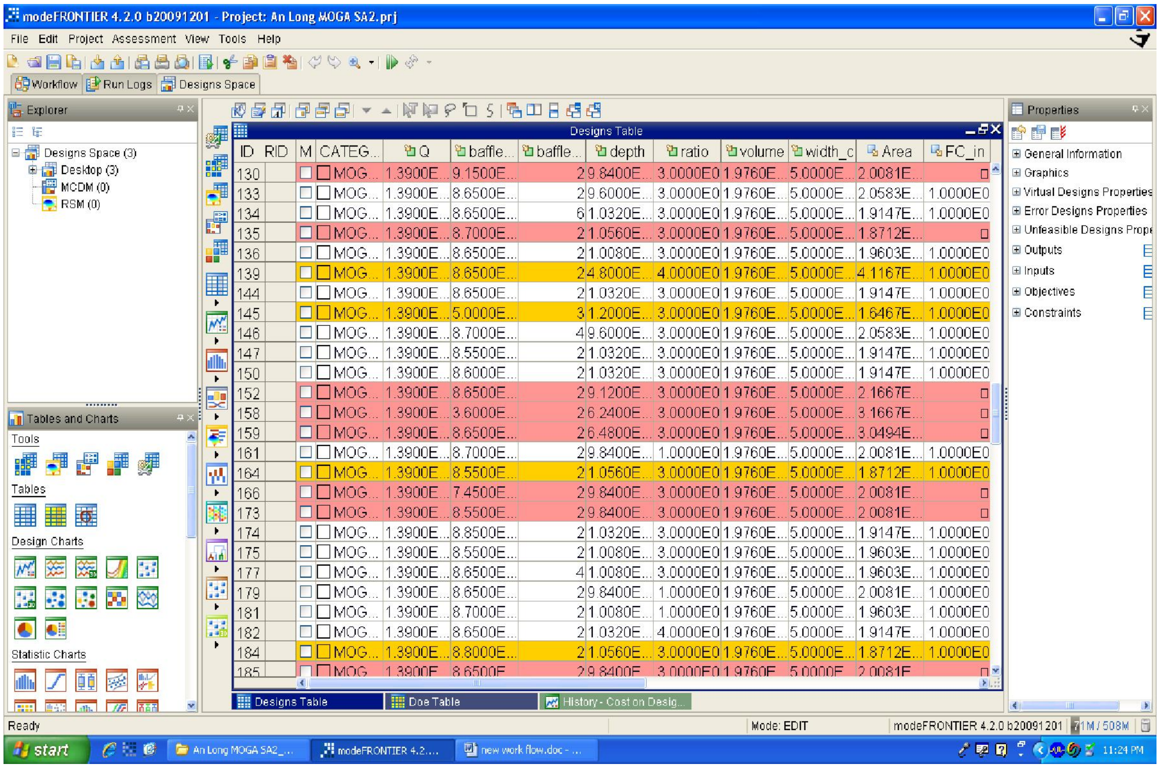

Figure 3.34 Designs table showing the outcomes of different reactor configurations

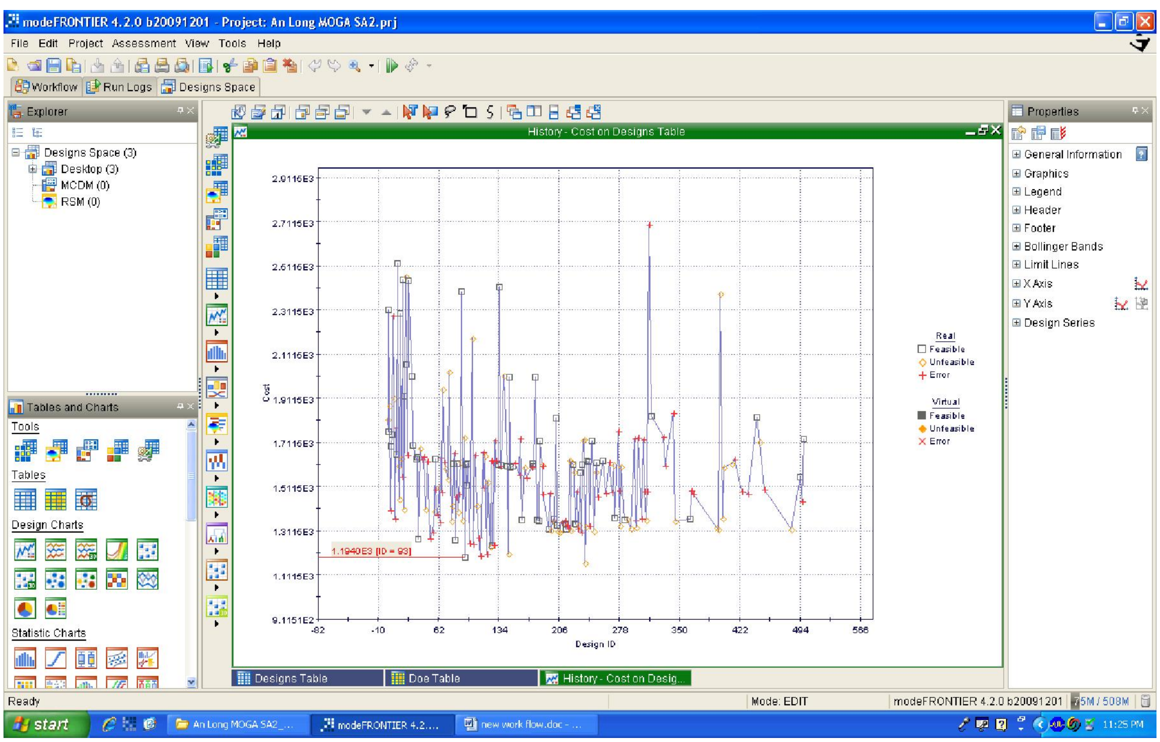

Figure 3.35 History cost on designs table showing the optimized cost

3.13.3 Sensitivity Analysis on the model parameters

This research focuses on parameter sensitivity in which the researcher sets two different parameter values to see how a change in the parameter causes a change in the dynamic behavior of the configuration of the model and its flow pattern. A sensitivity analysis was performed on the model parameters to determine the influence of first order constant (k) and temperature (T) on the design configurations. The various model parameters (baffle length, baffle number, reactor area, depth, effluent faecal coliform, and the cost) were checked, one at a time, to evaluate the effects of the changing two parameters on the other fixed parameters.

The results obtained from the optimization algorithm considering all the parameters indicated that changing the two parameters have effect on the effluent faecal coliform and the entire ponds configuration. Difference in values of the pond size, baffle length, baffle numbers and other parameters were observed. A 50 percent variation was made to the value of K^1 proposed and adopted by Banda (2007) in the equation, $K = 4.55(1.19)^{T-20}$ where $K^1 = 4.55$ and $T = 24^0C$, the room temperature value used for the model designs. Equation 3.38 expresses the relationship between the K and K^1 as:

$$K = K^1(1.19)^{T-20} \qquad 3.38$$

where:

$$K^1 = K^1 \pm 0.5K^1 old \text{ or}$$

$$K^1 = K^1_{old}(1 \pm 0.5)$$

Therefore, $K = 6.825(1.19)^{24-20} = 13.686\ (day^{-1})$ at the upper bound

$K = 2.275(1.19)^{24-20} = 4.562\ (day^{-1})$ at the lower bound

Also for temperature, a 20 percent variation was evaluated and equation 3.39 expresses how T value was varied while keeping the old K^1 of 4.55.

$$K = K^1_{old}(1.19)^{T^1-20} \qquad 3.39$$

where

$$T^1 = T^1_{old} \pm 4^0C$$

Therefore, $K = 4.55(1.19)^{28-20} = 18.297\ (day^{-1})$ at the upper bound

$K = 4.55(1.19)^{20-20} = 4.55\ (day^{-1})$ at the lower bound

The optimization models run with a population and generations which took time range between 45 minutes and 48 hours for the SIMPLEX and MOGA-II algorithm on a desktop computer (Intel ® Core(TM) 2 Duo CPU E6550) of 2048MB RAM. The results of geometrical configuration of all the optimal designs and the history cost on design for the reactors based on Simplex and MOGA II algorithms for the sensitivity tests are presented in the results chapter. This indicates that the reactor configuration is sensitive to its kinetics.

3.13.4 Running of output results from modeFRONTIER with the CFD tool

The design output from the optimization tool are labeled with identifications of which some designs were selected and run with the CFD tool in order to evaluate the physical design geometry. The selected designs are the optimal design, minimum faecal coliform removal design and the maximum faecal coliform designs. The history cost on design chart and the results of the selected geometries are presented in the results chapter.

3.13.5 Summary of the optimization methodology

Four design configuration types that were evaluated are even transverse, even longitudinal, odd transverse and odd longitudinal baffle arrangements to investigate the effects of these configurations on the effluent quality from the three reactors. The odd and longitudinal configurations were included because of limited research on them found in the literature. The positions of the outlet in the odd baffle configuration are placed on the same side with inlet for the longitudinal baffle arrangement as against the alternate side positioning in even transverse arrangement. The inclusion of the odd transverse and longitudinal arrangements allow the total investigation into the entire possible scenarios for selecting the optimal reactor configuration that could give the best wastewater treatment.

CHAPTER 4

MODELING RESULTS AND ANALYSIS

4.1 Model results for the RTD curve and FC inactivation for unbaffled reactors

The residence time distribution and fecal coliform inactivation of the reactors have been used to reveal the hydraulic characteristics of each pond. The solution of the model test for simulation of the unbaffled reactors converged and produced a residence time distribution curve in the CFD for anaerobic, facultative and maturation reactors respectively. Figures 4.1, 4.3 and 4.5 show the shapes of RTD curves and it was observed that the response curves at the rising limb and at the exit are identical and being approximately exponential, but there is an initial lag due to the reality of physical mixing in the reactor. The pick of the rising limbs in the three unbaffled reactors depicts an element of bypassing/short circuiting and the long tails shows that there are dead zones in which the simulated tracer in the CFD slowly diffuses out of the reactor. Therefore, degree of mixing is characterized by the effect of short circuiting and dead spaces in the reactors.

Short-circuiting fluid within the reactors appear in the RTD curves as a pulse of concentrated tracer that reaches the outlet shortly after injection. The residence time distribution of these reactors clearly does not fit the pattern of either the ideal cases of Plug Flow and Completely Mixed reactors, but lies somewhere between them. This implies that some mixing is occurring within the basin, but not complete mixing. Intuitively, this behavior was part of the thinking and development of reactor models as reported in literature that incorporation of baffles could give a better hydraulic efficiency.

The effluent concentration of the anaerobic was used as the influent concentration into the facultative pond and the same was done for the maturation pond. This gave a total log removal of 1.35 of faecal coliform at the end of the maturation pond treatment. The flow pattern produced in the un-baffled reactors was a large swirl. Figures 4.2, 4.4 and 4.6 describe the distribution of faecal coliform concentration and the hydraulic flow patterns in the reactor models. As would be seen from the figures, there are indications of short-circuiting and an introduction of spatial mixing which is the more reason why it is difficult to achieve a perfect plug flow condition due to these recirculation zones. The Physical intervention that could be employed is through the introduction of baffles to solve the hydraulic challenges. The RTD are in dimensionless time while the scale on the right shows the inactivation of coliform from the top to the bottom as concentration decreases.

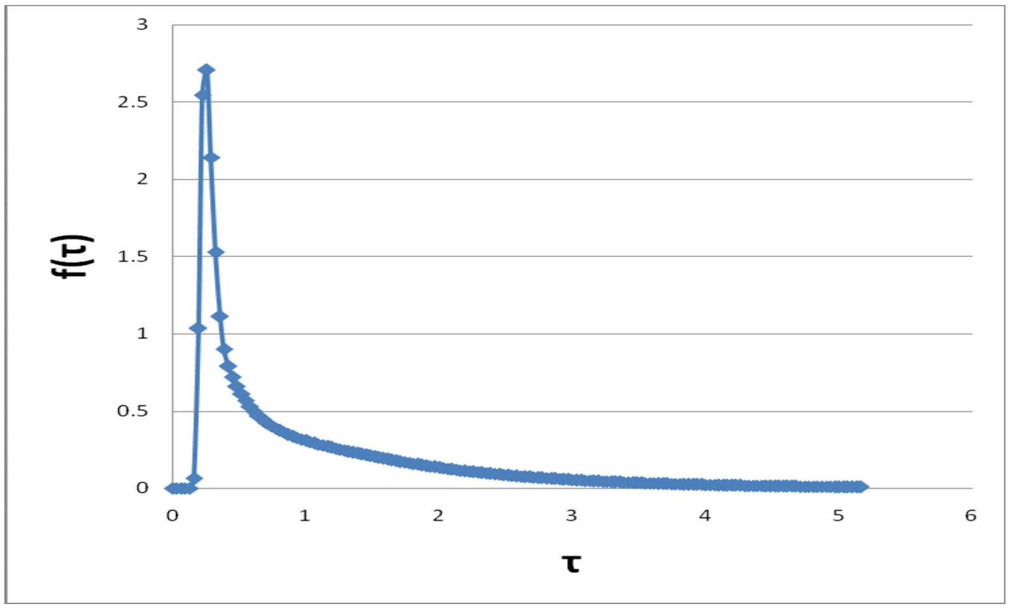

Figure 4.1 Residence time distribution curve for unbaffled anaerobic reactor

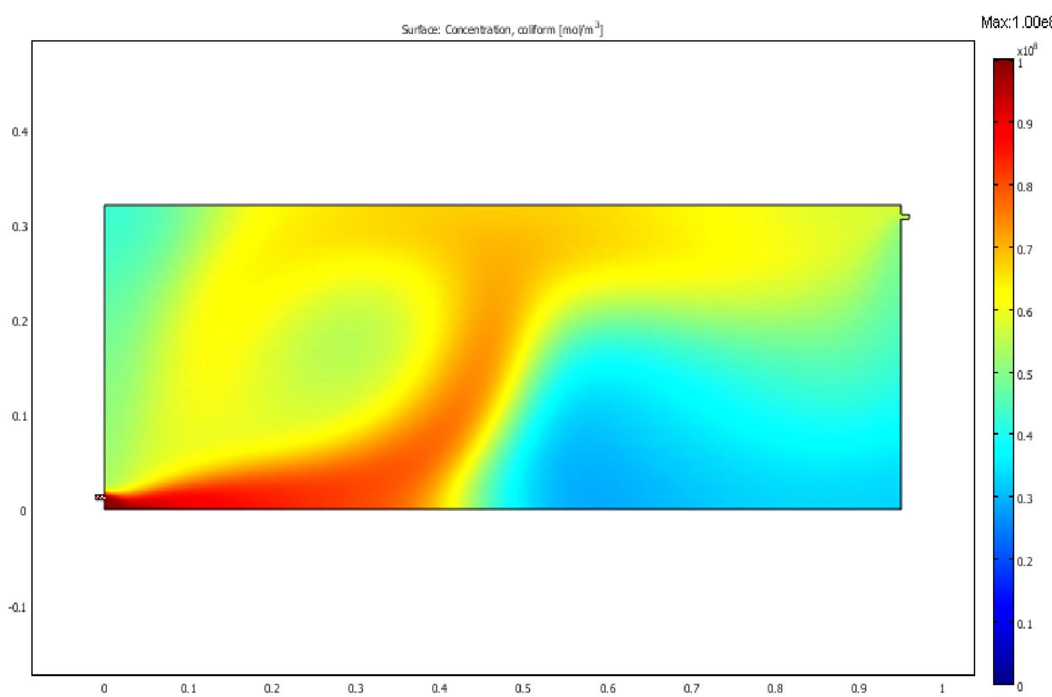

Figure 4.2 Anaerobic unbaffl**e**d reactor coliform inactivation

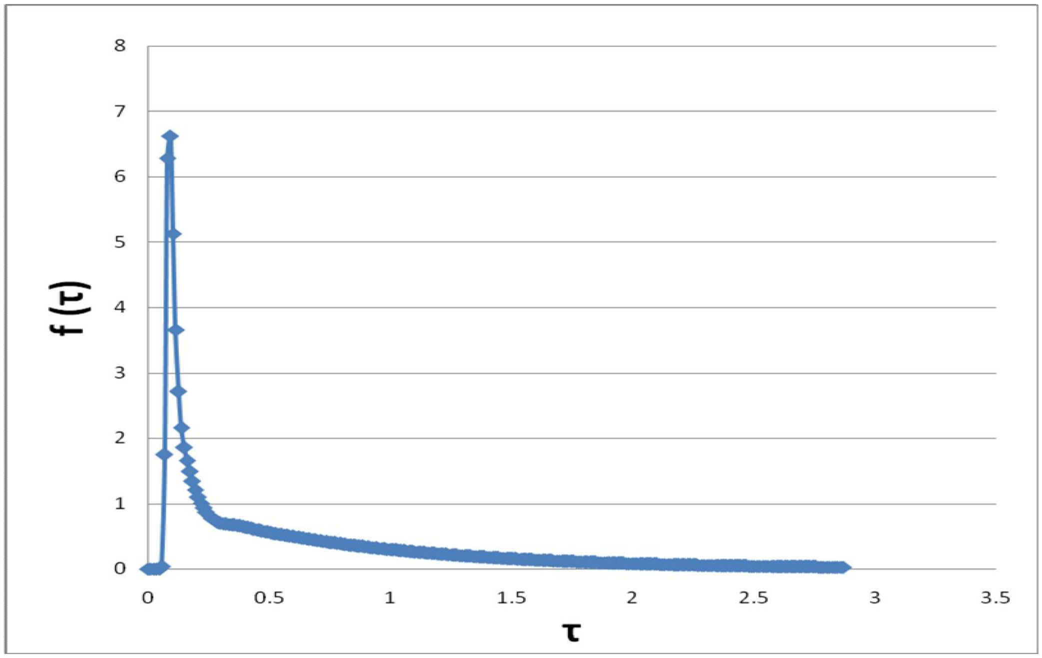

Figure 4.3 Residence time distribution curve for unbaffled facultative reactor

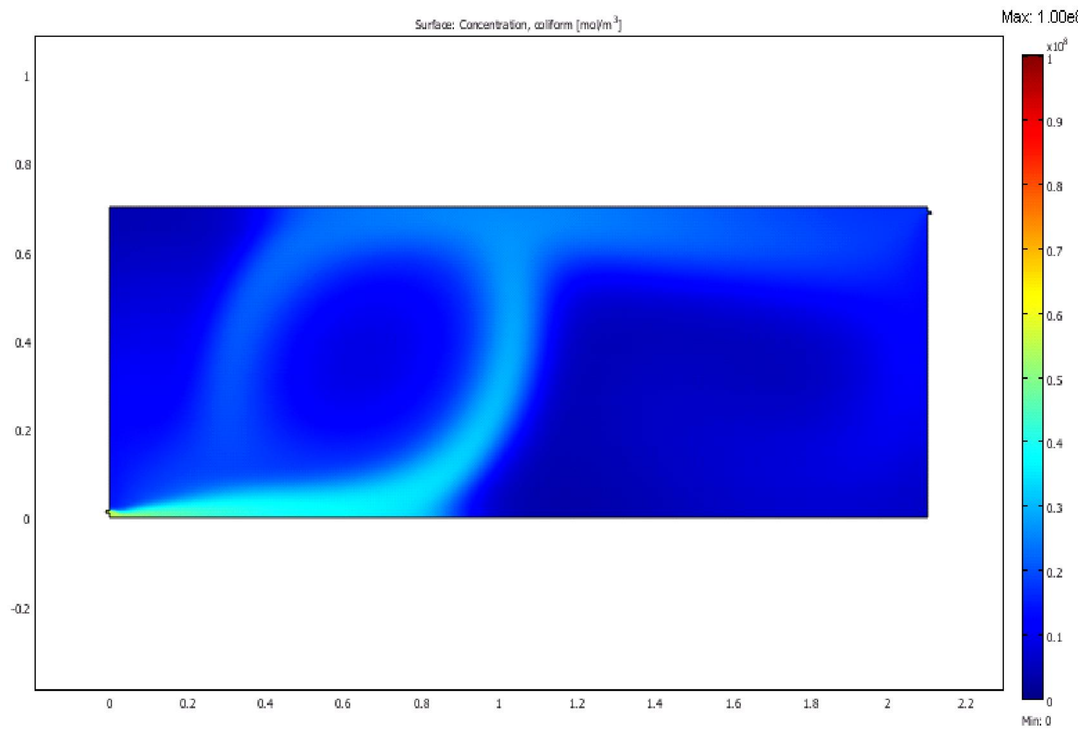

Figure 4.4 Facultative unbaffled reactor coliform inactivation

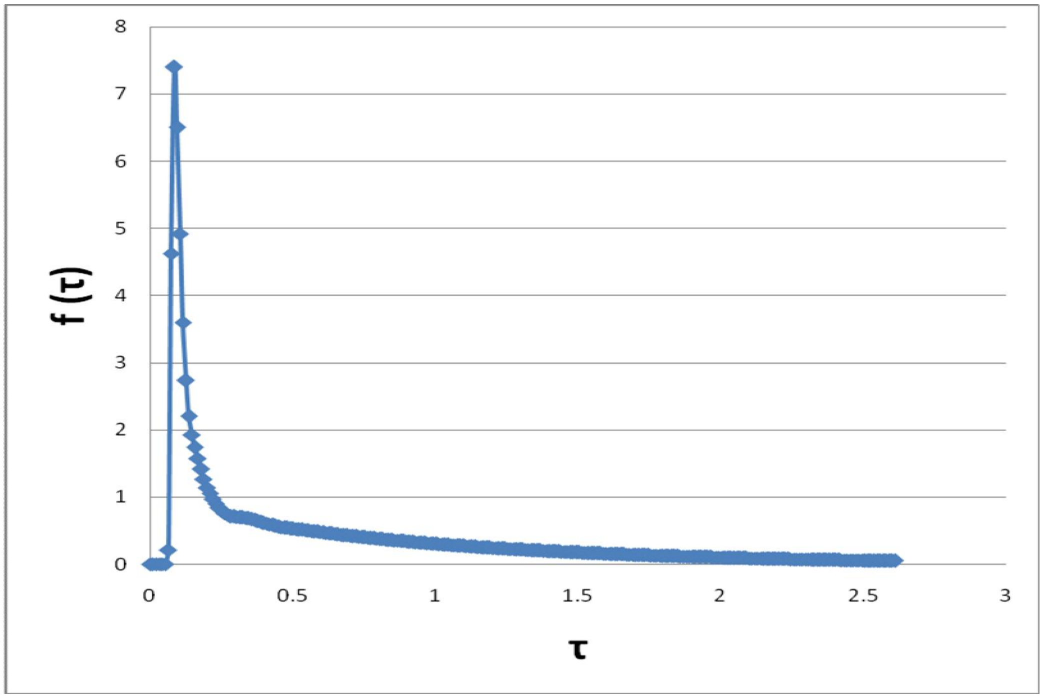

Figure 4.5 Residence time distribution curve for unbaffled maturation reactor

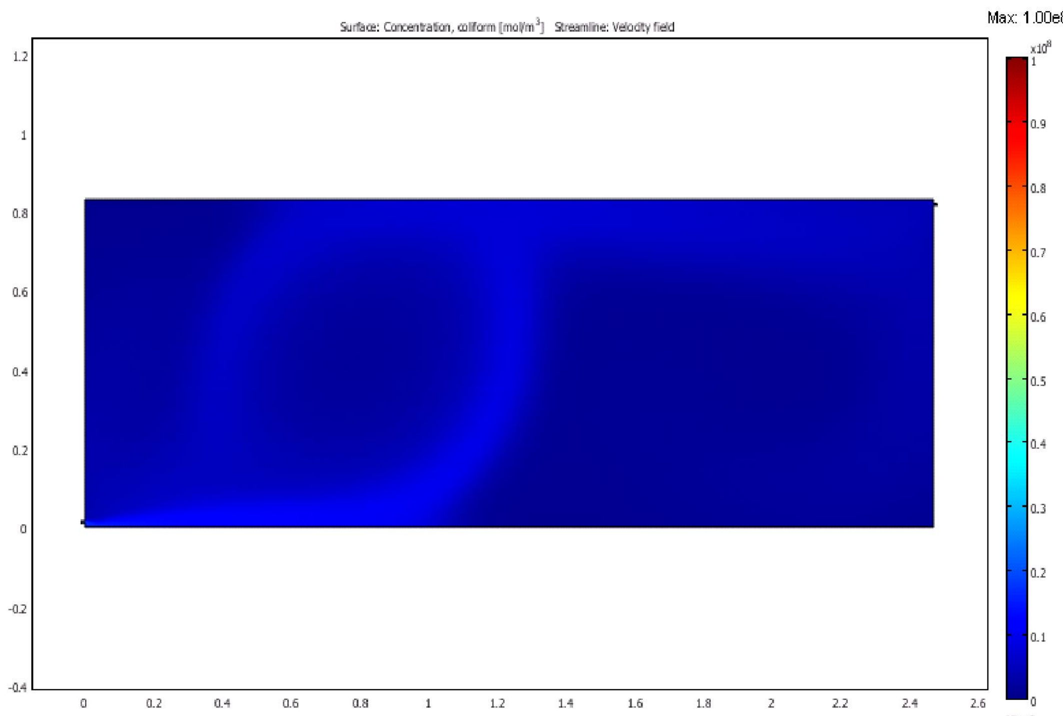

Figure 4.6 Maturation unbaffled reactor coliform inactivation

4.2 Initial Evaluation of baffled WSP designs in the absence of Cost using CFD

The use of baffles as a physical design intervention to improve the hydraulic efficiency of laboratory-scale waste stabilization pond and to address hydraulic problems has been explored. In addition, the residence time distribution of the wastewater in the four and six baffle reactors confirm the initiation of plug flow pattern as revealed from literature and it can be deduced that the use of more baffles will reduce significantly the hydraulic short-circuiting that was initiated when there was no baffle. An increase of the number of baffles, or of the length to width ratio (L/W ratio), was found generally to cause an increase of the actual retention time, and a decrease of the short circuit and dead-water regions of a reactor as reported by various reseracher (Kilani and Ogunrombi, 1984; Thackston *et al*., 1987; Muttamara and Puetpaiboon, 1997; Mangelson and Watters, 1972; Houang *et al*., 1989; Persson, 2000; Bracho et al., 2006). The six-baffle reactors were able to simulate the plug flow model better due to their approximate constant flow channel width when compared with the two and four baffles.

Table 4.2 presents the results of a series of models that were tested using evenly spaced, 60%, 70%, 80% and 90% pond-width for two-baffle and four-baffle longitudinal arrangement and 50%, 60%, 70%, 80% and 90% pond-width for two-baffle, four-baffle and six-baffle transverse arrangement respectively. These configurations were tested to investigate whether other baffles sizes can compete with the conventional 70% pond-width transverse baffle arrangement in improving the treatment efficiency of the laboratory-scale WSP.

The longitudinal arrangement seems promising but could be very costly in terms of construction. Watters et al (1973) expressed that baffles of 70% width gave superior performance compared to the 50% and 90% baffle width. He discovered that increasing the baffle width to 90% was found to give a lower hydraulic efficiency than was seen with the 70% width baffles. However, with cost in mind one would consider other options that have a closer performance.

The study carried out by Abbass et al. (2006) to examine the effect of the assumed longitudinal rectangular shapes and dimensions with a constant area for values of various water depth, flow rate and HRT of wastewater demonstrated an increased BOD removal efficiency from 16% at area ratio of 1:1 without baffles to 93% and 96% at area ratio of 4:1 with the inclusion of two and four baffles respectively. The authors' results also displayed a decreased DO effluent concentration from 0.5mg/l at area ratio 1:1 without baffles to 6mg/l with two transverse baffles and increased to 10mg/l when the area ratio was 4:1 with the inclusion of four transverse baffles at one-third and one-fifth of the baffle length respectively. The result showed that the area ratio 4:1 with two and four transverse baffles is most efficient in improving the overall water quality in the WSP hydrodynamics and BOD removal efficiency.

The use of the first order kinetics proposed by Marais (1974), Mara (2001) and Banda (2007) has been explored to investigate the decay of faecal coliform in this research. The Banda (2007) first order kinetics for coliform removal in baffled waste stabilization pond was chosen due to the fact the Marais equation did not consider the effect of baffles when the equation was developed. The short comings of this have been extensively researched by Banda (2007) and its outcome has been considered in this study.

The general flow pattern has been studied and the treatment efficiency has also been quantified by integrating first order kinetics within the model thereby giving the effluent faecal coliform count for various pond configurations. Table 4.1 shows that the longitudinal 2-baffle 90% pond width gave the best pond performance as against the finding of Watters et al (1973). The faecal coliform count 1×10^8 mol/m^3 was used as input for all the simulation at the unset. The effluent coliform count from the preceding reactor is fed as the influent into the next reactor in the simulation process. As the wastewater moves through the reactors in series, the inactivation of faecal coliform makes the value to reduce which gave the faecal coliform total log kill for each set of configurations (Table 4.1).

Table 4.1 Results of different simulated configurations using the CFD model

S/No.	No of Baffles/ Configuration	Anaerobic Effluent	Facultative Effluent	Maturation Effluent	Total Log Kill
1	2b L 60%	33510032	1682804.10	49393.97	3.3063
2	2b L 70%	27073288	670663.90	8395.28	4.0760
3	2b L 80%	22534788	277331.10	1470.84	4.8324
4	2b L 90%	20609958	143073.47	360.47	5.4431
5	4b L 60%	39581376	2906977.20	130222.54	2.8853
6	4b L 70%	30842584	984005.20	15350.12	3.8139
7	4b L 80%	24413772	331841.44	1839.03	4.7354
8	4b L 90%	20990248	2112413.00	4083.21	4.3890
9	2b T 50%	28953854	1937942.50	99224.47	3.0034
10	2b T 60%	28975650	1949568.00	96205.44	3.0168
11	2b T 70%	28996292	1900311.60	89976.45	3.0459
12	2b T 80%	29422114	1947204.90	93808.55	3.0278
13	2b T 90%	29720890	2224030.50	125535.74	2.9012
14	4b T 50%	25526230	896946.40	15878.70	3.7992
15	4b T 60%	24702388	822044.94	11075.98	3.9556
16	4b T 70%	24511018	801252.75	11431.66	3.9419
17	4b T 80%	24852566	830749.10	12957.81	3.8875
18	4b T 90%	25346972	1008013.75	20722.48	3.6836
19	6b T 50%	24924464	786442.44	15793.26	3.8015
20	6b T 60%	22605908	417691.30	4291.25	4.3674
21	6b T 70%	21814500	306853.22	2192.69	4.6590
22	6b T 80%	21575564	305415.10	2233.36	4.6510
23	6b T 90%	22016944	411628.94	4371.28	4.3594

where: T = transverse arrangement

L = longitudinal arrangement.

In the transverse arrangement from Table 4.1, it was discovered that the six baffle cases proved to be more efficient than the four baffle cases. This was attributed to channeling effects. However, when the results were compared against the longitudinal baffle arrangements, it was found that the 2-baffle 80% and 90% and 4-baffle 80% and 90% pond width configuration were more efficient. This outcome supports the report of Lloyd et al., (2003) that the increase of L/W ratio induced by longitudinal baffles can significantly increase fecal coliform removal efficiency in ponds. Figure 4.7 clearly shows that a high log reduction was achieved using a 2 baffle 90% baffle length arranged in the longitudinal configuration for the three reactors in series.

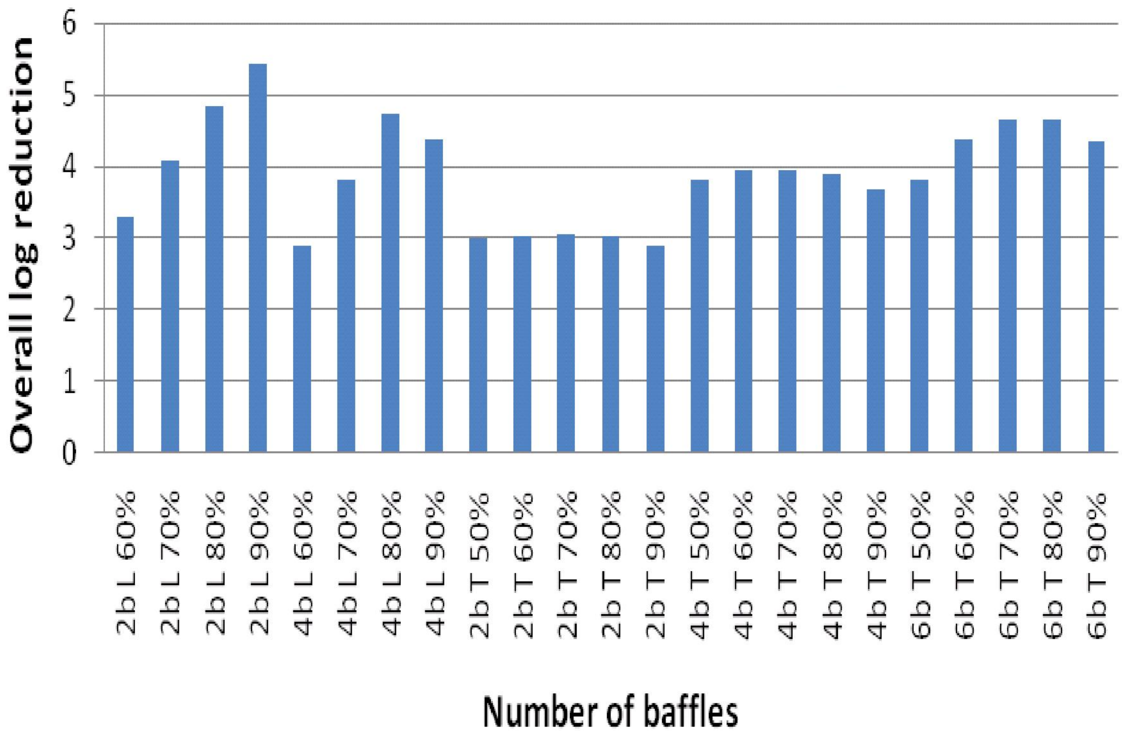

Figure 4.7: Overall log reductions for the WSP system over a range of baffle lengths tested

The model's sensitivity to changes in baffle length on the overall log reduction was greater with baffles arranged in the longitudinal configuration compared to the transverse configuration. The improved performance of the longitudinal baffles when long baffle lengths are used is likely due to the longer internal length to width ratios and consequent longer path lengths traveled by the fluid/fecal coliform through the pond. However, when longitudinal baffles are arranged parallel to the longer pond side, microbial transport down the narrow channel widths are more influenced by the mixing characteristics at the baffle ends. The overall comparison of the longitudinal baffling and the transverse baffling to some extent gave significant level of feacal log kill. It is also interesting to know that as the length of the baffle increases in the anaerobic, facultative and maturation reactors from 60% to 90% in the longitudinal arrangement, there is improvement in the performance of the reactors.

Tables 4.2 and 4.3 display the initial model simulations that involved manually adjusting the baffle lengths for a single pond dimension. Only the 70% baffle lengths for the transverse and longitudinal baffle arrangements are reported in Tables 4.2 and 4.3, respectively. These initial simulations were not based on minimizing cost but to review the impact of baffle number, placement, and arrangement on the effluent log reduction to mimic what has been reported in the literature. The results of several other configuration simulations as shown in Table 4.1 suggest that the 70 percent baffle width was not consistently the best design when considering only the optimal effluent fecal coliform log reduction.

Table 4.2 CFD Results and associated costs for 70% pond-width Transverse baffle arrangement

	Anaerobic Transverse	**Facultative Transverse**	**Maturation Transverse**	**Cumulative cost and log removal**
Cost (N)	1, 582	5, 431	7, 221	14, 234
Log removal	0.61	1.49	1.84	3.94
Reactor L/W ratio (r)	3:1	3:1	3:1	-
Depth (m)	6.5E-2	4.50E-2	4.0E-2	-
Baffle ratio	70%	70%	70%	-
Number of baffles	4	4	4	-

Table 4.3 CFD Results and associated costs for 70% pond-width Longitudinal baffle Arrangement

	Anaerobic Longitudinal	Facultative Longitudinal	Maturation Longitudinal	Cumulative cost and log removal
Cost (N)	1, 926	5, 960	7, 772	15,658
Log removal	0.51	1.50	1.81	3.82
Reactor L/W ratio (r)	3:1	3:1	3:1	-
Depth (m)	6.5E-2	4.50E-2	4.0E-2	-
Baffle ratio	70%	70%	70%	-
Number of baffles	4	4	4	-

The results in Table 4.1 seem to contradict previous research studies which showed that the 70% width is the appropriate design configuration for WSP treatment systems. This contradiction was due to the absence of the longitudinal baffle arrangement as part of the assessment in previous research studies. Consequently, it is possible that previous researchers may have overlooked a more cost efficient WSP design. The results in Tables 4.2 and 4.3 do reveal that a 4 baffle 70% transverse baffle length arrangement may produce a high fecal coliform log removal at a cost that is lower than the 4 baffle longitudinal arrangement. The question that remains is whether the log reduction achieved in Figure 4.7 with this baffle length, reactor L/W ratio, and baffle arrangement is a cost effective WSP design configuration. Figures 4.8- 4.19 display the normalized residence time distribution curve, velocity streamline and the coliform inactivation for the 4-baffles transverse and longitudinal arrangements in the three reactors presented in Tables 4.2 and 4.3. These describe the distribution of faecal coliform concentration and the hydraulic flow patterns in the reactor models.

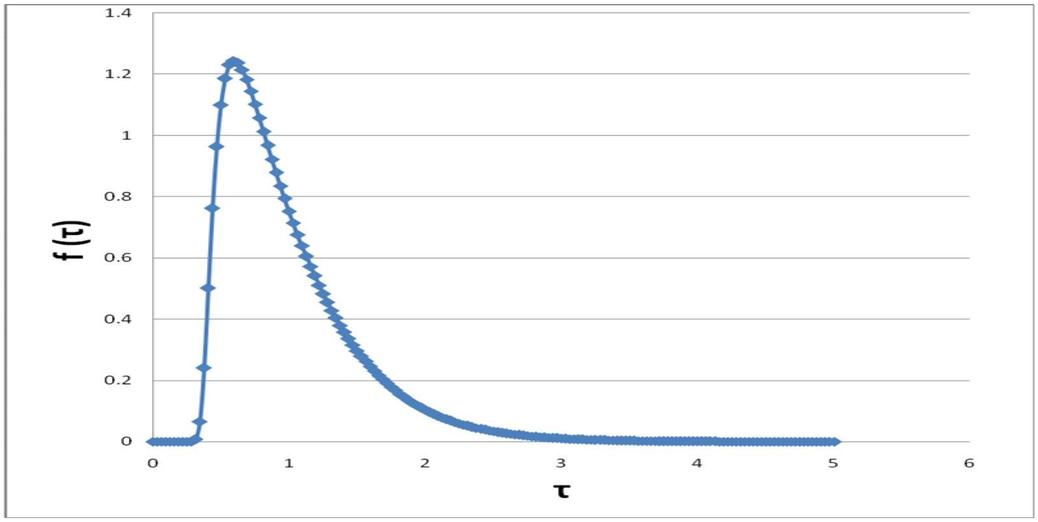

Figure 4.8 Residence time distribution curve for transverse 4-baffle with 70% Pond width anaerobic reactor

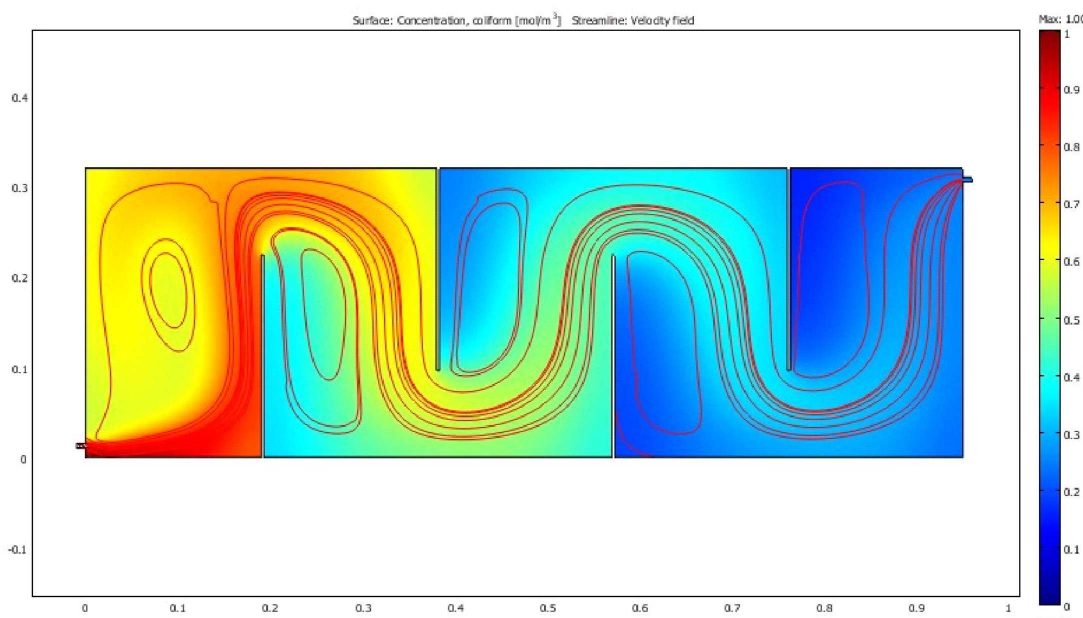

Figure 4.9 Velocity streamline and coliform inactivation for transverse 4 baffle 70% pond width anaerobic reactor.

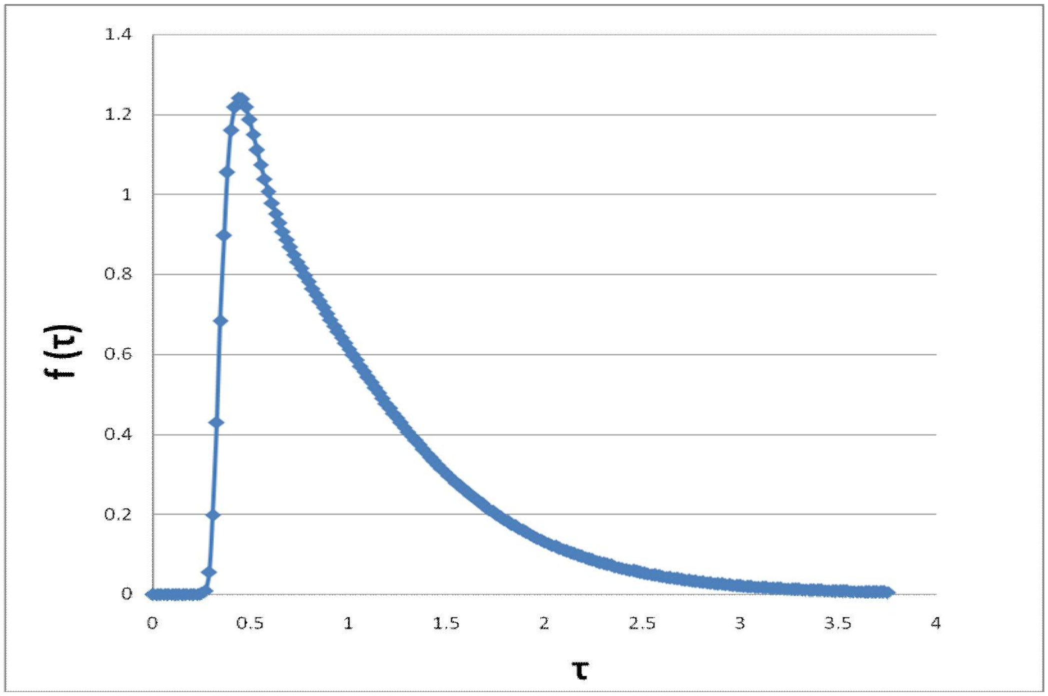

Figure 4.10 Residence time distribution curve for transverse 4-baffle with 70% pond width Facultative reactor

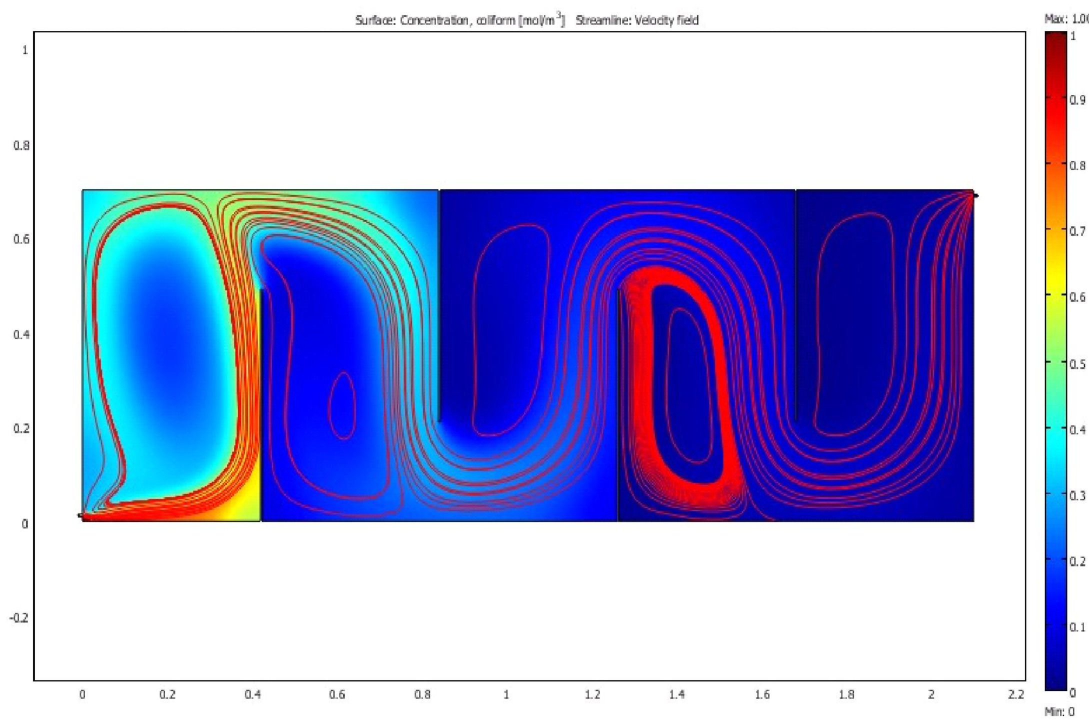

Figure 4.11 Velocity streamline and coliform inactivation for transverse 4 baffle 70% pond width facultative reactor

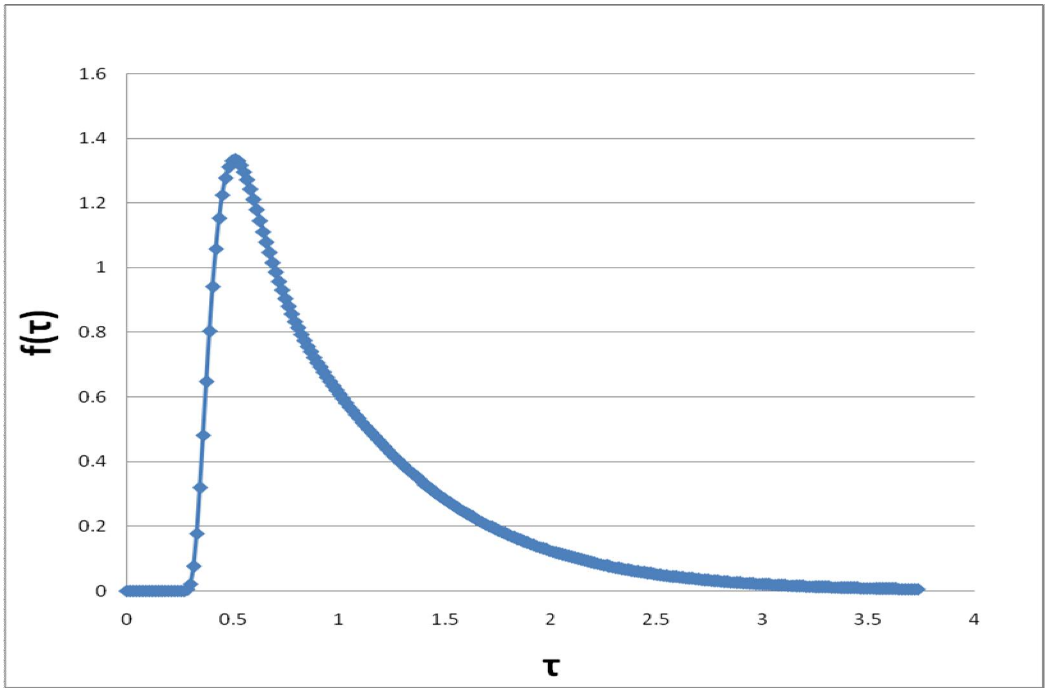

Figure 4.12 Residence time distribution curve for transverse 4-baffle with 70% Pond width Maturation reactor

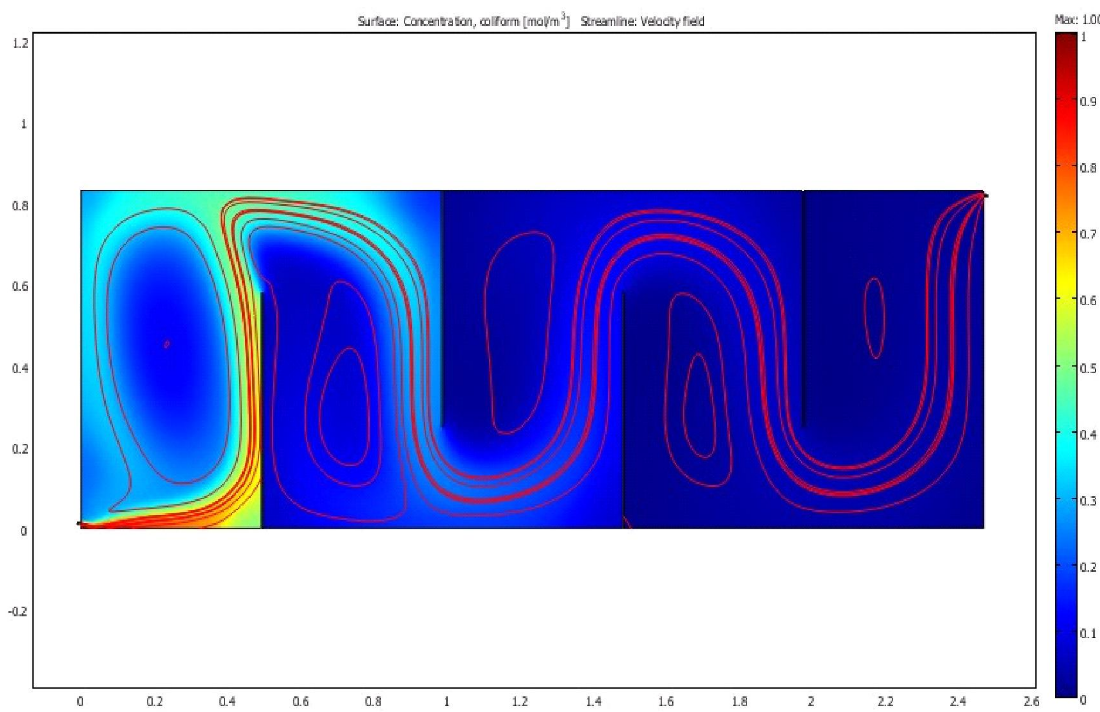

Figure 4.13 Velocity streamline and coliform inactivation for transverse 4 baffle 70% pond width Maturation reactor

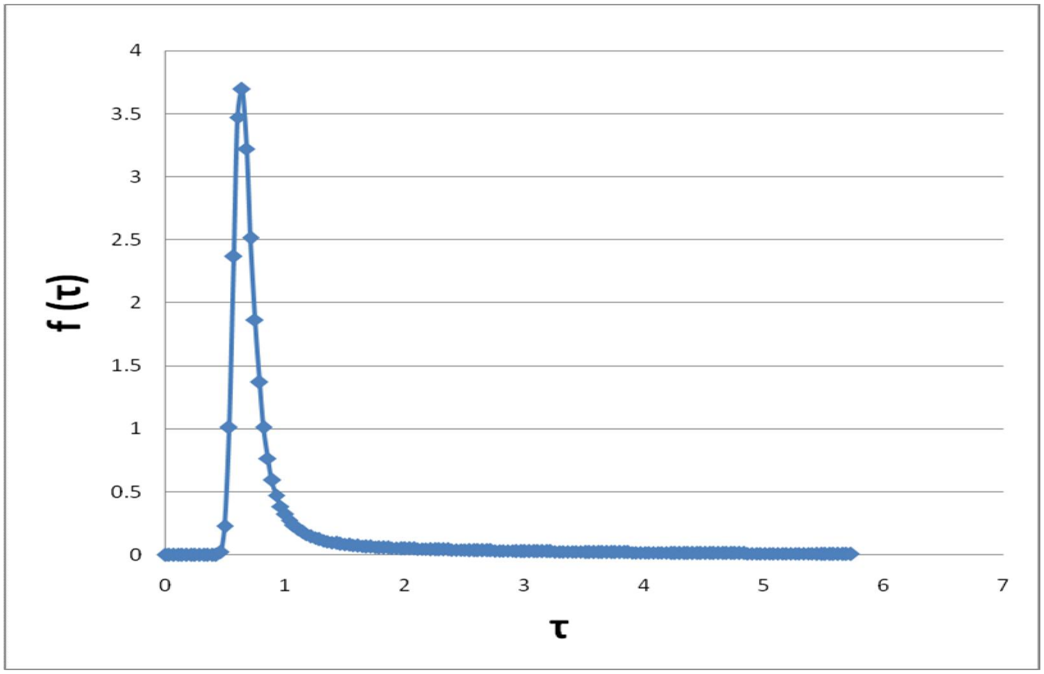

Figure 4.14 Residence time distribution curve for longitudinal 4-baffle with 70% pond width anaerobic reactor

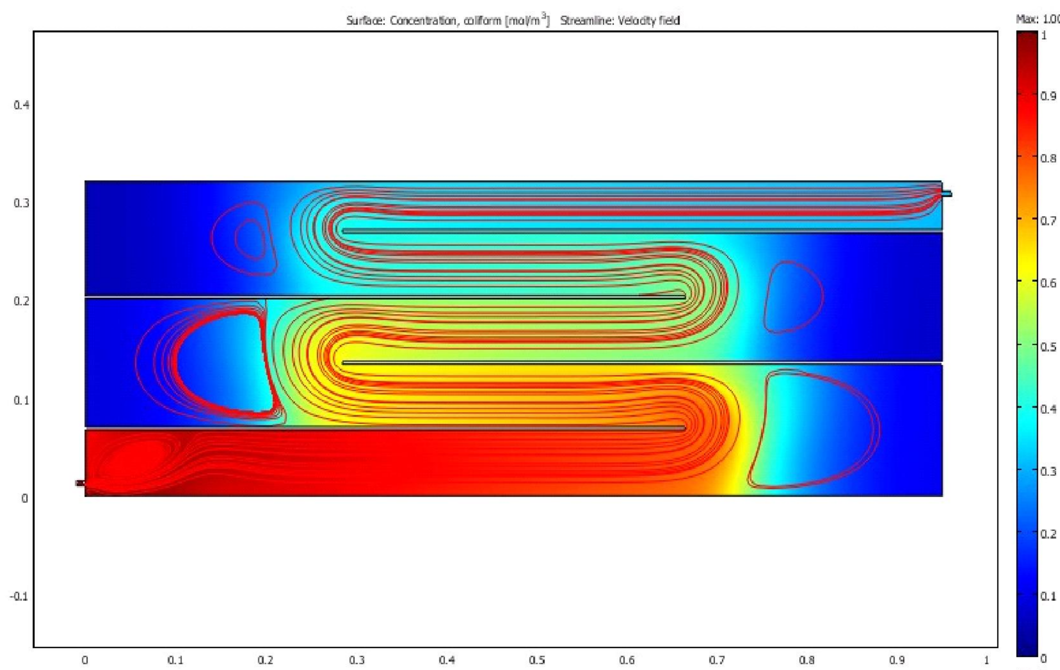

Figure 4.15 Velocity streamline and coliform inactivation for longitudinal 70% pond width anaerobic reactor.

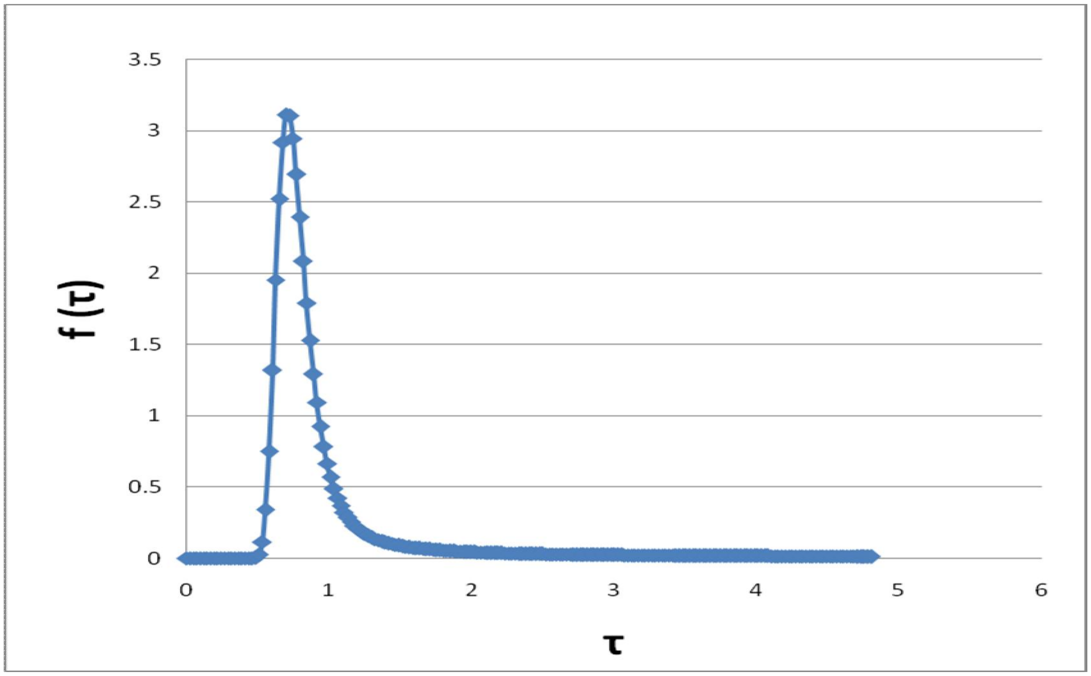

Figure 4.16 Residence time distribution curve for longitudinal 4-baffle with 70% pond width Facultative reactor

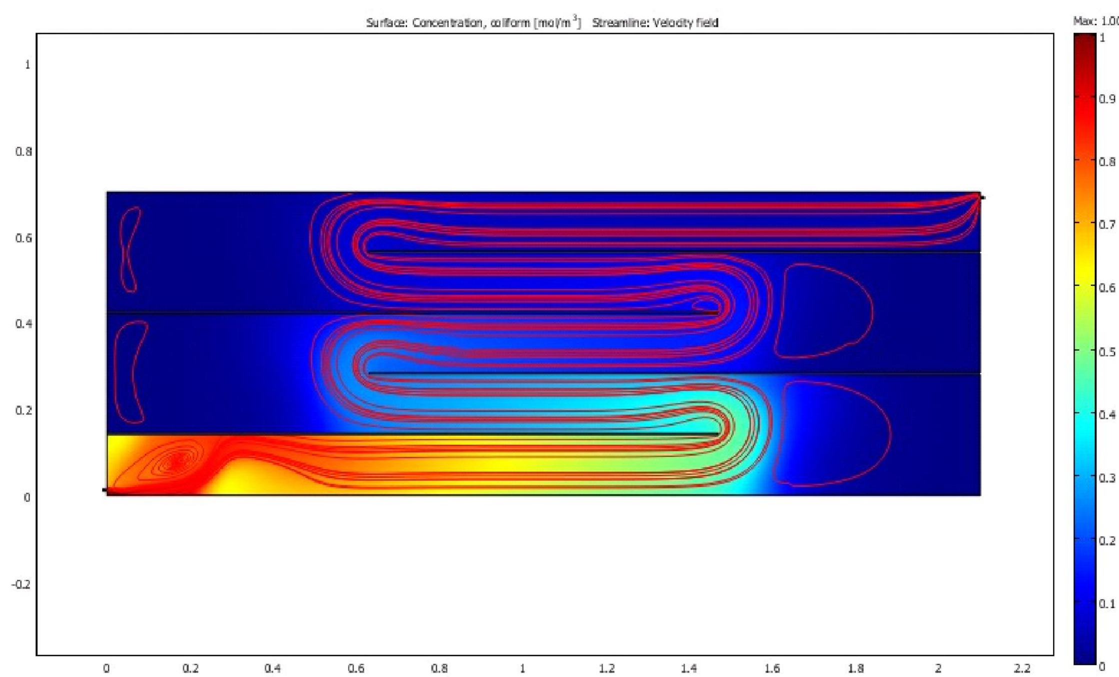

Figure 4.17 Velocity streamline and coliform inactivation for longitudinal 4 baffle 70% pond width facultative reactor

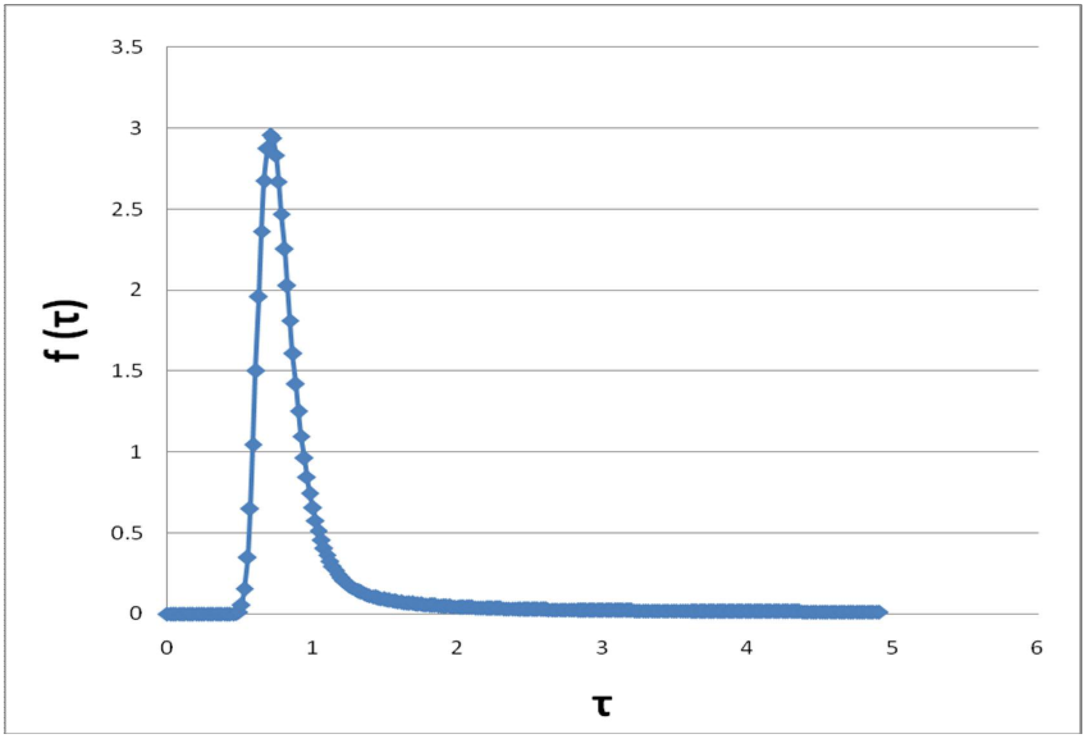

Figure 4.18 Residence time distribution curve for longitudinal 4-baffle 70% pond width Maturation reactor

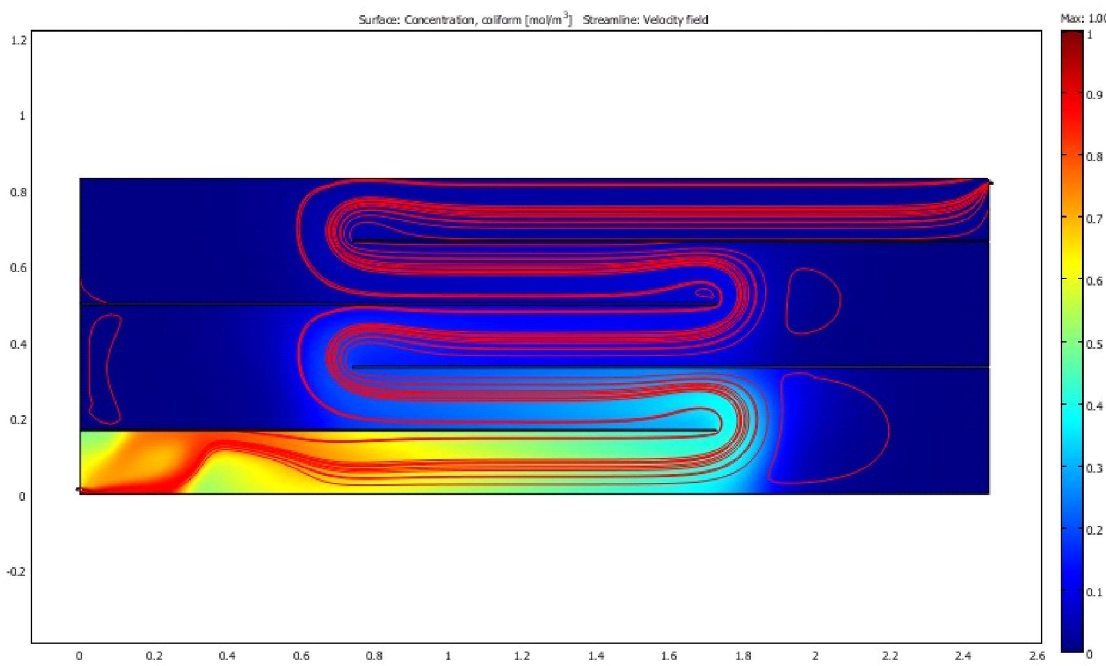

Figure 4.19 Velocity streamline and coliform inactivation for longitudinal 4-baffle 70% pond width maturation reactor

4.2.1 Application of segregated flow model to compare the result of RTD prediction and the CFD predictions for feacal coliform reduction for Figures 4.8-4.19

The application of the segregated flow model to compare the RTD prediction and the CFD predictions for feacal coliform reduction has given a better insight about mixing and performance of the reactors. Figures 4.8-4.19 makes it easy to visualize the flow pattern through the reactors. Each exit wastewater corresponds to a specific residence time in the reactor and batches of coliform are removed from the reactor at different locations along the reactor in a manner as to duplicate the RTD function. The coliform concentrations checked near the entrance to the reactors correspond to those having short residence times while the effluent corresponds to the coliform that channel through the reactor.

The mean conversion/ degree of microbial inactivation equals adopted from equation 3.38:

$$\overline{X} = \frac{\tau k}{1 + \tau k}$$

where:

τ = Hydraulic retention time (HRT)

k = reaction rate constant (9.124 d^{-1})

Table 4.4 gives the actual simulated hydraulic retention time (in days) for the three reactors in the transverse and longitudinal baffle arrangements. This values were extracted from the excel spread sheet that was used in determining the actual HRT for each of the reactor configurations. Numerical RTD data generated by the CFD simulation of feacal coliform inactivation were used so that comparisons could be made between the various process conversion models for the RTD curves. Therefore, substituting the values of different (τ) in Table 4.4 and constant value of (k) in the three reactors for the two baffle arrangements, comparison of the CFD predictions for feacal coliform reduction and simulated RTD was achieved. This was used in determining the actual HRT in the 4 baffle 70% pond-width for the three reactors in Figures 4.8- 4.19 respectively and are presented in Table 4.5.

Table 4.4 Actual simulated hydraulic retention time (in days) for the three reactors

Reactor type	Anaerobic 4-baffle 70% pond width		Facultative 4-baffle 70% pond width		Maturation 4-baffle 70% pond width	
Baffle arrangement	Transverse	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal
HRT	0.186	0.158	0.620	0.468	0.777	0.573

Substituting these values into equation 4.9, the mean conversion/ degree of microbial inactivation were determined for the three reactors:

Anaerobic transverse baffle arrangement

$$\frac{0.186\times 9.124}{1+0.186\times 9.124} = \underline{\mathbf{0.63}}$$

Anaerobic longitudinal baffle arrangement

$$\frac{0.151\times 9.124}{1+0.151\times 9.124} = 0.58$$

Facultative transverse baffle arrangement

$$\frac{0.62\times 9.124}{1+0.62\times 9.124} = \underline{\mathbf{0.85}}$$

Facultative longitudinal baffle arrangement

$$\frac{0.468\times 9.124}{1+0.468\times 9.124} = 0.81$$

Maturation transverse baffle arrangement

$$\frac{0.777\times 9.124}{1+0.777\times 9.124} = \underline{\mathbf{0.88}}$$

Maturation longitudinal baffle arrangement

$$\frac{0.573\times 9.124}{1+0.573\times 9.124} = 0.84$$

The results of the underlined mean conversion clearly shows the degree of microbial inactivation is greater in the 70% 4-baffle transverse and performs better than the longitudinal baffle. This also confirms the result of the hydraulic retention time for the three reactors and having an overall fecal log kill greater in transverse- than longitudinal arrangement. The reactor analysis demonstrates that inactivation efficiency is affected by both mixing and kinetics (and their interaction). As such, both mixing and kinetics should be considered in reactor design for inactivation processes.

The result of the segregated flow model gives an excellent prediction of the CFD predictions for feacal coliform reduction. With the summary of the results in Table 4.5, one can conclude that the RTD predictions have been found to be in good agreement with the established results of CFD prediction for feacal coliform reduction.

Table 4.5 Comparison of the RTD and the CFD predictions for feacal coliform reduction

	Anaerobic 4-baffle 70% pond width		Facultative 4-baffle 70% pond width		Maturation 4-baffle 70% pond width	
	Transverse	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal
HRT (days)	0.186	0.158	0.620	0.468	0.777	0.573
Degree of microbial inactivation	**0.63**	0.59	**0.85**	0.81	**0.88**	0.84

As shown in Figures 4.8 – 4.19 for the pond geometry, it could be seen that the inlet and outlet positions are at alternate ends due to the conclusion from research findings. Persson (2000), Shilton (2001) and Mara (2004) noted that the position and design of the inlet have a significant effect on the hydraulics of a pond. Pearson et al., (1995) also concluded that the positioning of the inlet and outlets may have a greater beneficial impact on treatment efficiency. A range of alternative inlet designs exists; however, most of these are simply methods for avoiding and minimizing the jetting effect of wastewater into the reactors. Little practical guidance exists on the design and positioning of inlets (Shilton and Harrison 2003a).

Many investigators have emphasized the importance of proper reactor inlet-outlet configurations in order to reduce short circuiting and enhance the overall performance (Mangelson and Watters, 1972; Moreno, 1990; Fredrick and Lloyd, 1996; Vorkas and Lloyd, 2000; Persson, 2000; Shilton *et al.*, 2000; Aldana et al., 2005). However, it is not clear whether the inlet-outlet configurations suggested by these authors can be adopted for older or newly designed ponds.

Recent research suggests that the inlet position and its relation to the outlet are more important than previously thought. Shilton (2001) made mention of the importance a combination between baffles and inlets. He reported that, in the presence of baffles, small and large horizontal inlets reduced the short circuiting of the pond over the use of vertical inlets. It was advised by Hamzeh and Ponce (2007) that the inlet should not discharge centrally in the pond as this maximizes hydraulic short-circuiting and a single inlet and outlet should be located in diagonally opposite corners of the pond. Shilton and Harrison (2003a) observed during a laboratory and modelling work, that the outlet had a localized influence. A vertical inlet was computer modeled and tested on a full-scale pond of somewhat different configuration to the laboratory experiments, it was not found to give any significant improvement over a horizontal inlet. This serves as a useful observation as it means that the inlet and baffling can be sorted out first which is to say that after the flow pattern has been determined by design of the inlet and the positioning of the baffles, then the outlet can be placed for maximum efficiency without the likelihood that it will subsequently alter the flow pattern.

It has generally been considered that the best position for an outlet is at the opposite end of the pond to the inlet (Hamzeh and Ponce 2007; Shilton and Harrison 2003a). Choosing the best location for the outlet in any pond is still going to require some reasonable degree of judgment from the designer by placing the outlet close into a corner or a region that obviously is out of the main flow path. It is with this fact that the inlet and the outlet structures into the ponds were located at alternate corners for both the transverse and longitudinal arrangement following the literature recommendations of geometric design procedures (Persson, 2000; Shilton, 2001; and Mara, 2004).

4.3 Results of the N-Tanks in series and CFD models

The section presents the N-Tanks in series and CFD model results of a series of models that were tested using evenly spaced, 70% and 80% pond-width in both facultative and maturation reactors for two-baffle, four-baffle and six-baffle transverse arrangement respectively. The objective is to find the value of N for which the N-Tanks in series model response curve best fits the CFD response curve for scaled reactors and to evaluate simple macroscale model in order to predict the mixing in the reactors. Figures 4.20 – 4.31 describe the comparison between the RTD curves produced using N-Tanks in series model and the CFD predicted RTD curves for the 70% and 80% pond width tested in the laboratory for 2-baffle, 4-baffle and 6-baffle in both facultative and maturation reactors. The N-Tanks in series predictions were performed by using the empirical relationship provided by Crozes et al., (1998) relating the number of tanks in series to the reactor length to width ratio when baffles are included. The residence time density curves give confidence that the CFD model reasonably predicts the residence time distribution in the reactors as represented by the N-Tanks in series model.

Baffle numbers were increased up to 6 (giving the length to width ratios approaching 21). It is shown in Figures 4.20- 4.31 that as more baffles are introduced, the two curves generated by both models tend to get closer both in shape and size.

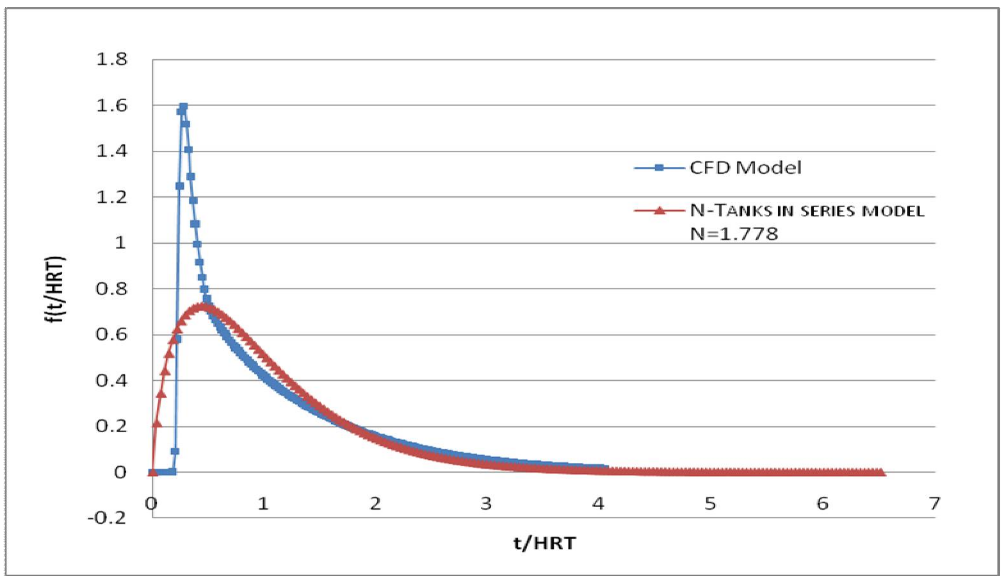

Figure 4.20 Comparion of residence time density curves in 2-baffles, 70% pond width Facultative reactor for CFD and N-tanks in series models.

For the 2 baffle 70% pond width, L = 3885mm and width = 700mm (Fig 4.20). L is the approximate flow length and W is the distance between baffles. The L/W ratio for the 2 baffled facultative pond was estimated as 5.55 which gives a corresponding value of N = 1.778 by adopting the equation in Fig 3.8.

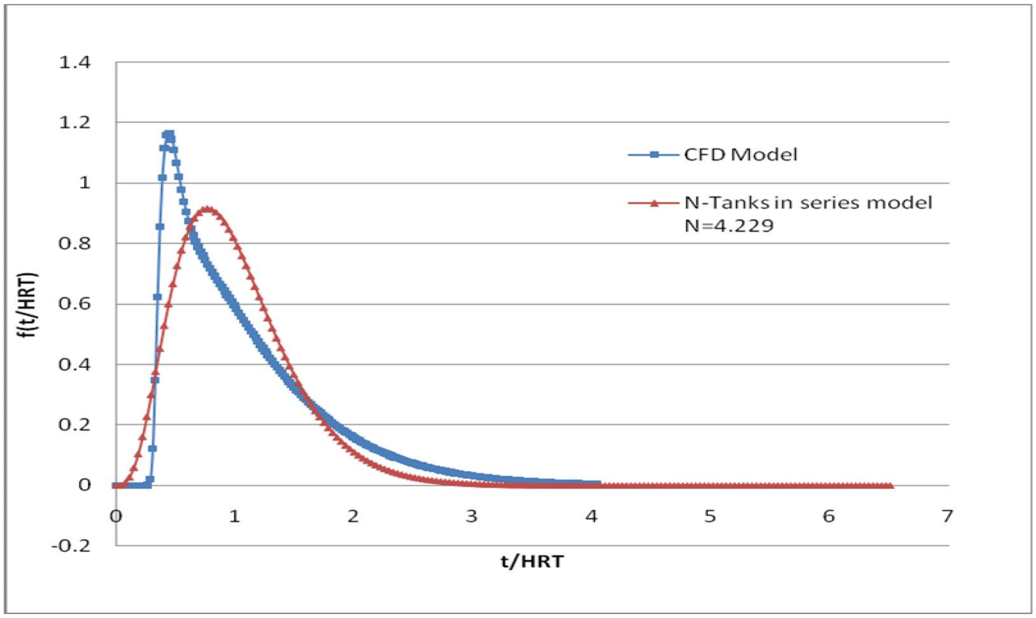

Figure 4.21 Comparion of residence time density curves in 4-baffles, 70% pond width Facultative reactor for CFD and N-tanks in series models.

For the 4 baffle 70% facultative pond width, L = 5075mm and width = 420mm (Fig 4.21). The L/W ratio was estimated as 12.08 which gives a corresponding value of N = 4.2289.

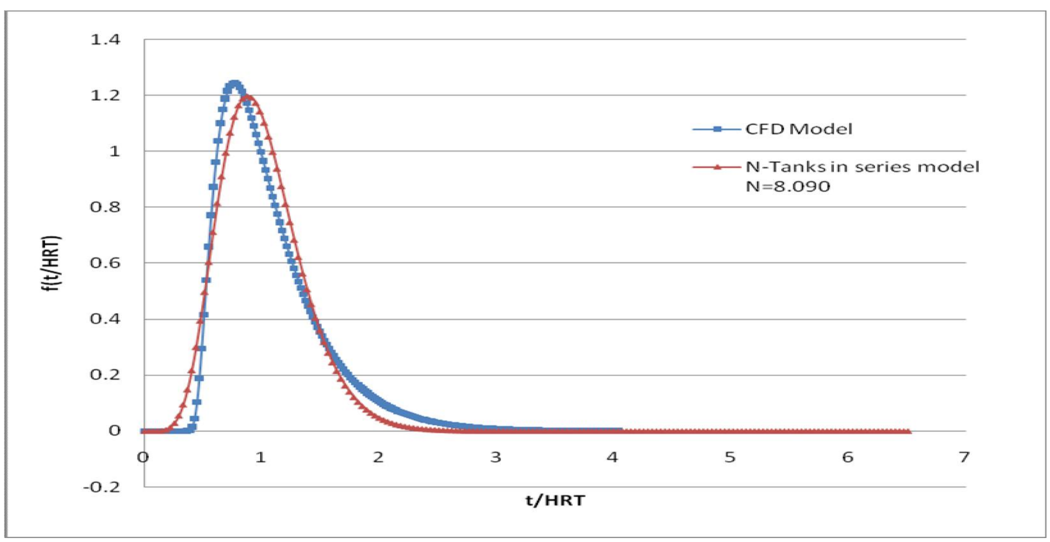

Figure 4.22 Comparion of residence time density curves in 6-baffles, 70% pond width Facultative reactor for CFD and N-tanks in series models

For the 6 baffle 70% facultative pond width, L = 6265mm and width = 300mm (Fig 4.22). The L/W ratio was estimated as 20.88 which gives a corresponding value of N = 8.09.

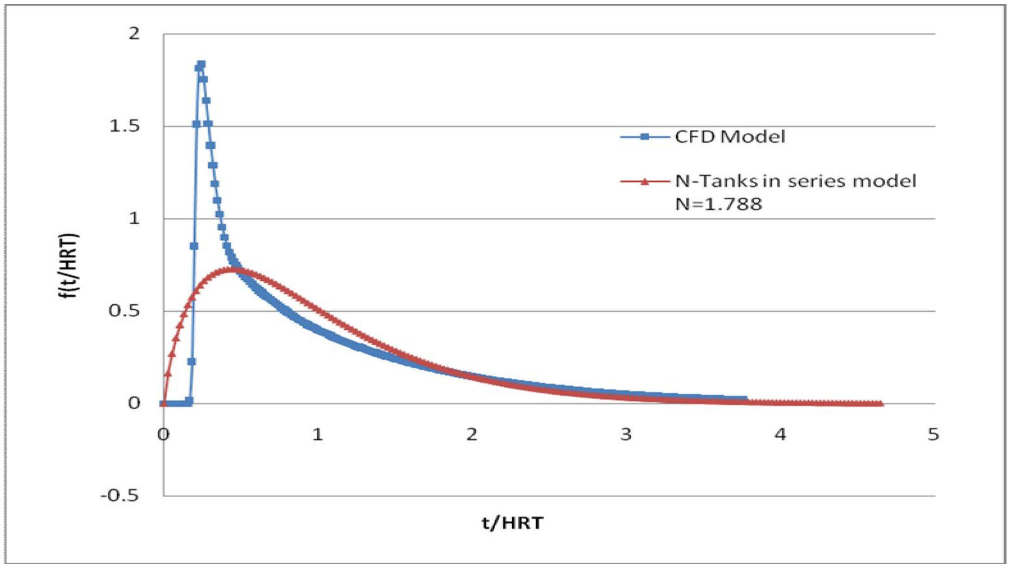

Figure 4.23 Comparion of residence time density curves in 2-baffles, 70% pond width Maturation reactor for CFD and N-tanks in series models

For the 2 baffle 70% maturation pond width, L = 4585.5mm and width = 823mm (Fig 4.23). The L/W ratio was estimated as 5.57 which gives a corresponding value of N = 1.788.

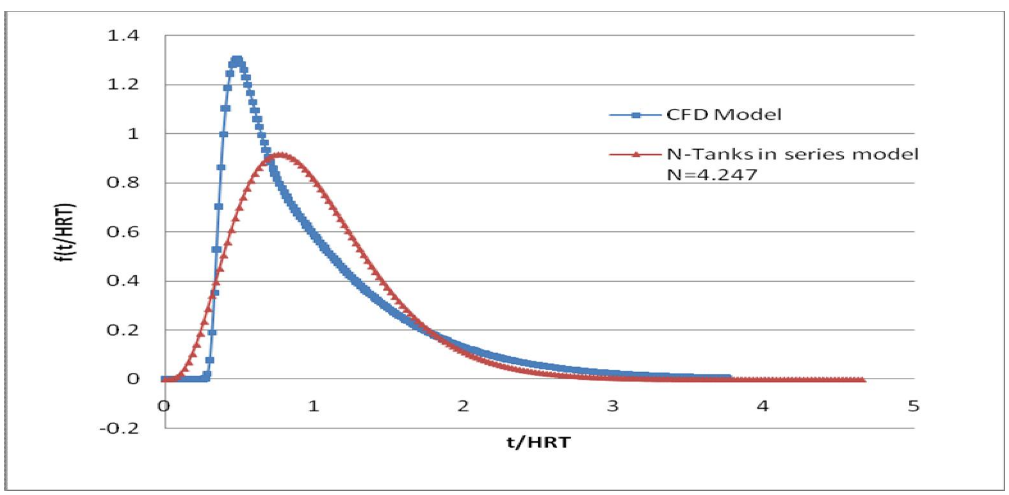

Figure 4.24 Comparion of residence time density curves in 4-baffles, 70% Pond width Maturation reactor for CFD and N-tanks in series models

For the 4 baffle 70% maturation pond width, L = 5997.5mm and width = 494mm (Fig 4.24). The L/W ratio was estimated as 12.14 which gives a corresponding value of N = 4.247.

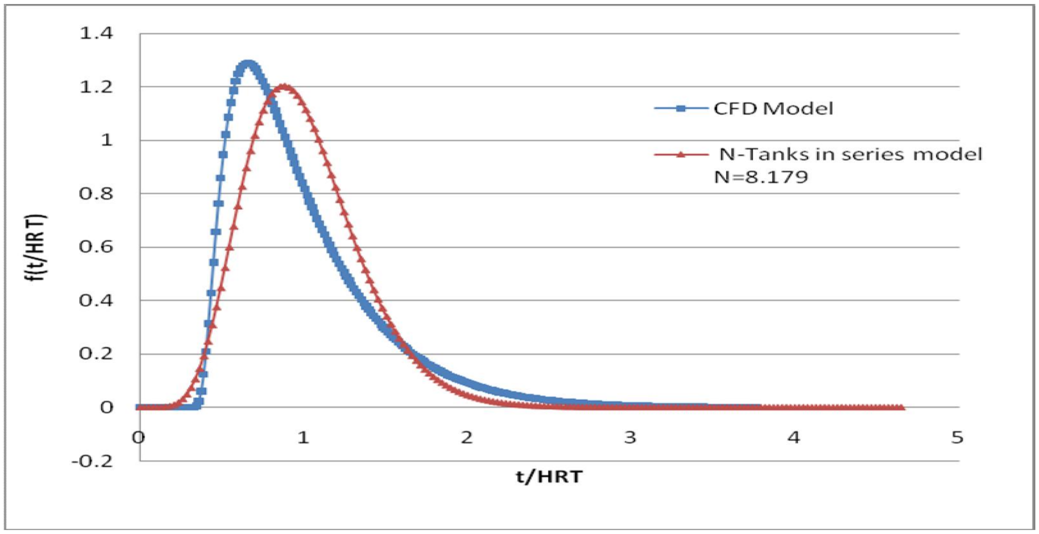

Figure 4.25 Comparion of residence time density curves in 6-baffles, 70% pond width Maturation reactor for CFD and N-tanks in series models

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For the 6 baffle 70% pond width, L = 7402.5mm and width = 352mm (Fig 4.25).

The L/W was estimated as 21.03 which gives a corresponding value of N = 8.179.

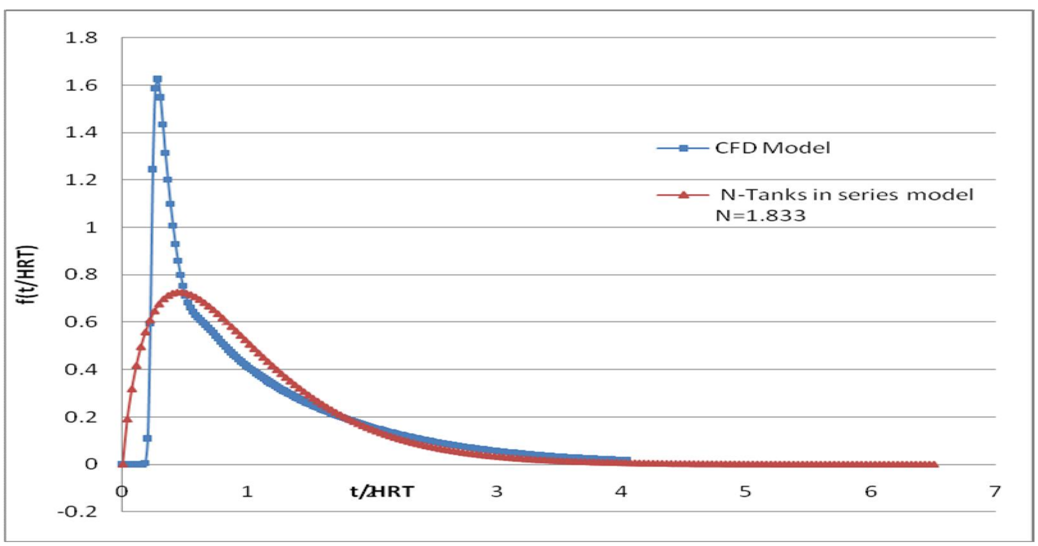

Figure 4.26 Comparion of residence time density curves in 2-baffles, 80% pond width Facultative reactor for CFD and N-tanks in series models

For the 2 baffle 80% facultative pond width, L = 3990mm and width = 700mm (Fig 4.26).

The L/W ratio was estimated as 5.7 which gives a corresponding value of N = 1.833.

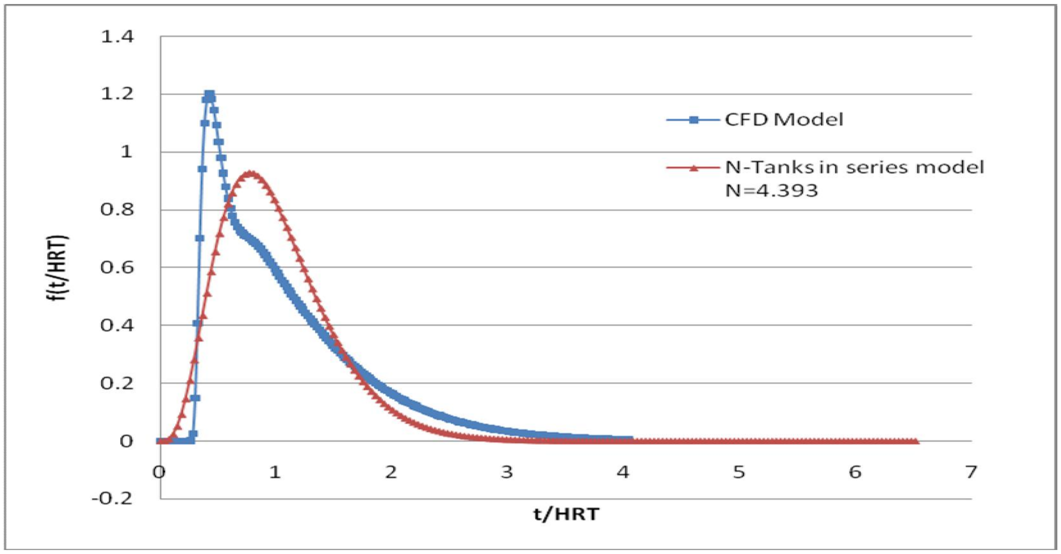

Figure 4.27 Comparion of residence time density curves in 4-baffles, 80% pond width Facultative reactor for CFD and N-tanks in series models

For the 4 baffle 80% facultative pond width, L = 5250mm and width = 420mm (Fig 4.27). The L/W ratio was estimated as 12.5 which gives a corresponding value of N = 4.393.

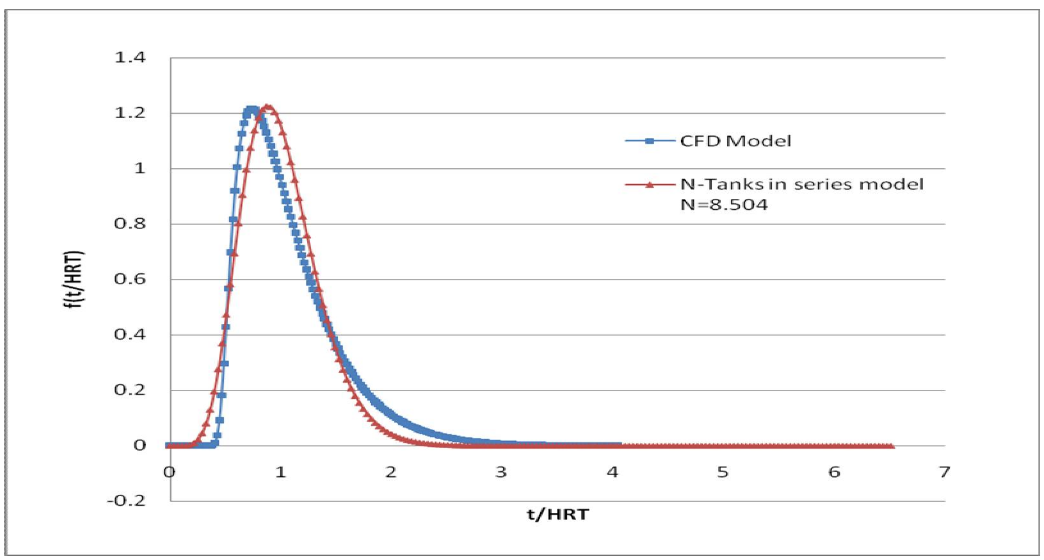

Figure 4.28 Comparion of residence time density curves in 6-baffles, 80% pond width Facultative reactor for CFD and N-tanks in series models.

For the 6 baffle 80% facultative pond width, L = 6510mm and width = 300mm (Fig 4.28). The L/W ratio was estimated as 21.7 which gives a corresponding value of N = 8.504.

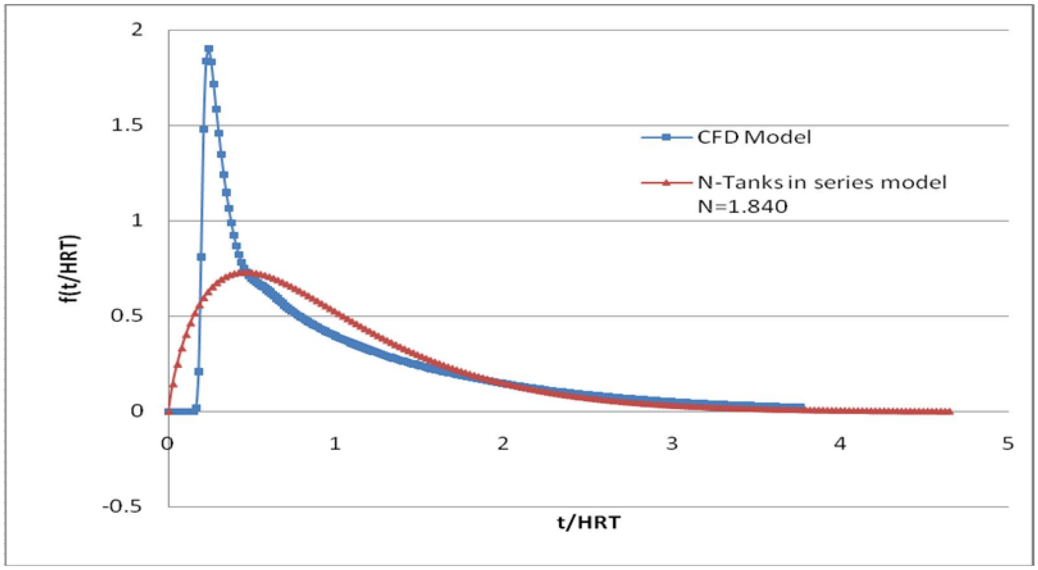

Figure 4.29 Comparion of residence time density curves in 2-baffles, 80% pond width Maturation reactor for CFD and N-tanks in series models.

For the 2 baffle 80% maturation pond width, L = 4710mm and width = 823mm (Fig 4.29). The L/W ratio was estimated as 5.723 which gives a corresponding value of N = 1.840.

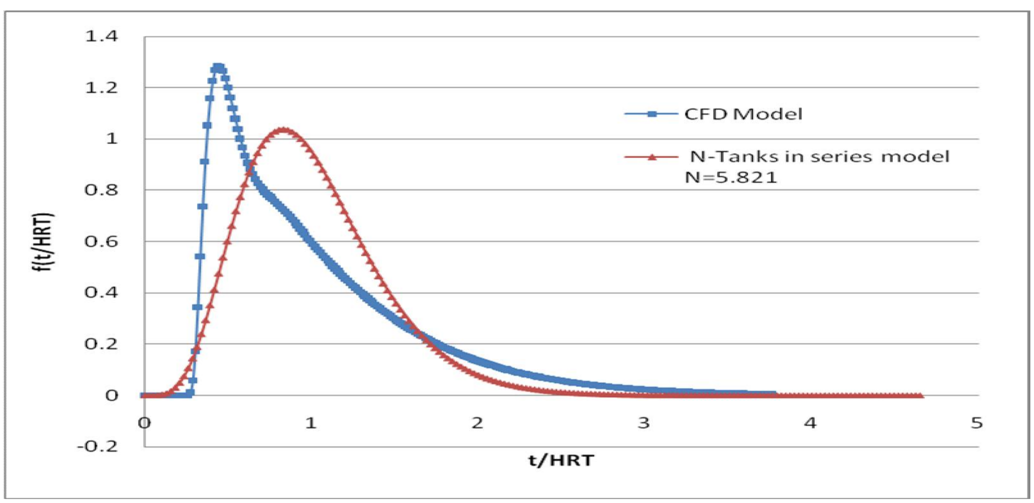

Figure 4.30 Comparion of residence time density curves in 4-baffles, 80% pond width Maturation reactor for CFD and N-tanks in series models.

For the 4 baffle 80% maturation pond width, L = 7850mm and width = 494mm (Fig 4.30). The L/W ratio was estimated as 15.891 which gives a corresponding value of N = 5.821.

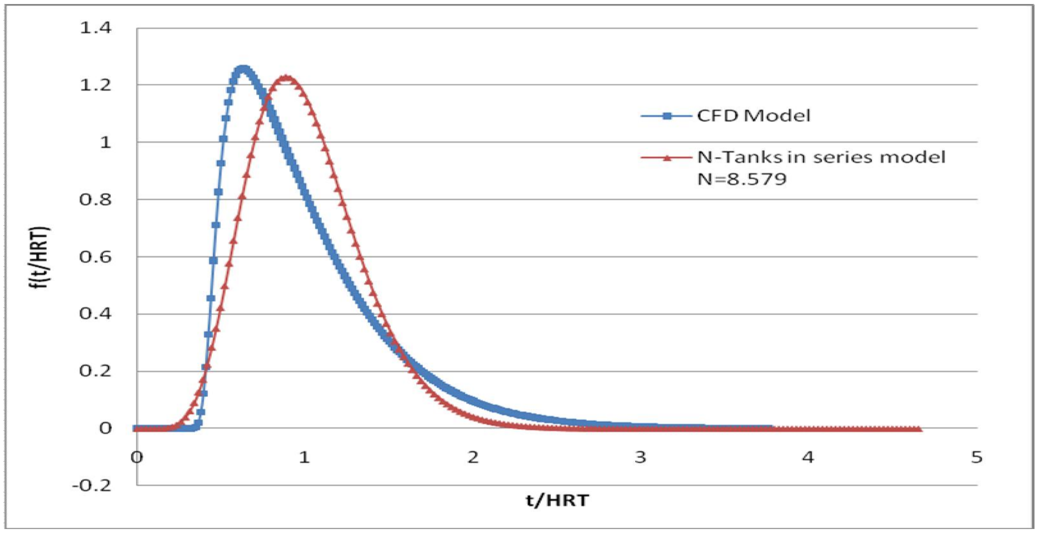

Figure 4.31 Comparion of residence time density curves in 6-baffles, 80% pond width Facultative reactor for CFD and N-tanks in series models.

For the 6 baffle 80% maturation pond width, L = 7693mm and width = 352mm (Fig 4.31). The L/W ratio was estimated as 21.855 which gives a corresponding value of N = 8.579.

4.3.1 General discussion on the results of the N-Tanks in series and CFD models

Varying the parameter N, the tanks in series characterizes mixing that varies between the complete mixer and the ideal plug-flow mixer. The Tanks in Series model simulate the actual reactor and the total volume of the tanks is the same as the volume of the actual reactor. Therefore, for a given flow rate, their total actual retention times are also the same. The model for the N-Tanks in series is taken as an empirical RTD with a single parameter N, which is considered a continuous variable as compared to the ideal plug-flow and perfect mixer reactor which are usually referred to as zero-parameter models (Clark, 1996). The Tanks-in-series model assumes a number of perfectly mixed tanks of equal size arranged in series. It is shown that as the number of baffle increases, the length to width ratio also increases and the residence-time distribution curve moves from the exponential distribution to a distribution that increasingly seem to be centered at $t/_{HRT} = 1$.

It is not surprising to see the 2-baffle residence time density curves generated by the N-Tanks in series model in both the 70% and 80% pond-width reactors deviate because, it shows that with reduction of baffles in the reactors, the hydraulic flow pattern will be much closer to that of a complete mix. The *N*-tanks in series model has been proven to be very simple to use and requires little input data to generate the RTD function. The larger the values of N, the more similar the shapes of the curves become. From Figures 4.20-4.31, it can be concluded that an increase in the number of baffles from 2 to 6, or of the length to width ratio (L/W ratio), approaching 21 was found generally to cause an increase of the actual retention time, and a decrease of the short circuit and dead zones of the reactor of which results was also achieved by other researchers (Kilani and Ogunrombi, 1984; Thackston *et al*., 1987; Muttamara and Puetpaiboon, 1997; Mangelson and Watters, 1972; Houang *et al*., 1989; Persson, 2000; Bracho et al., 2006). However, while Muttamara and Puetpaiboon (1997) concluded that the maximum removal efficiency of the ponds was reached at six baffles; Houang *et al*. (1989) reported that the effects of the L/W ratio do not become important until this ratio exceeds ten. At the same time, Thackston *et al*. (1987) also found that two baffles, producing a L/W ratio of 5-10, were usually sufficient.

It is interesting to know that a number of previous studies have concluded that inclusion of baffles in pond design gives better hydraulic efficiency and there is a general belief that increasing the length to width ratio of a pond helps force its hydraulic behavior towards plug flow. It is with this knowledge that baffles were introduced and the effect of such introduction has been verified. In order to modify the L/W ratio, two, four and six baffles were introduced. While the baffles were increased from two to four, for the 70% pond-width facultative pond, the effective length of the reactor was almost doubled, while the effective width was almost halved. Therefore, a value of L/W = 12.08 was obtained. Increasing the baffle length from four to six did not increase both the length and the width as much as compared to when it was increased from two to four. However, there was a significant increase in the L/W ratio (20.88). Similarly, the baffles were increased from two to four, for the 70% pond-width maturation pond, the effective length of the reactor changed from 4585.5mm to 5997.5mm, while the effective width was halved. Therefore, a value of L/W = 12.14 was obtained. Increasing the baffle length from four to six increased the length from 5997.5mm to 7402.5mm and reduced the width from 494mm to 352mm respectively which gave an L/W ratio of 21.03. Similar results were obtained for the 80% pond width facultative and maturation pond as detailed in Figures 4.32-4.37. A wide range of length to width ratio (L/W) and depth to width ratio (h/W) values have been reported for laboratory reactors. Some of these values are summarized in Tables 4.6.

Table 4.6: Laboratory system geometry and flow rate conditions in published literature.

Authors	L (m)	W (m)	h (m)	L/W	h/W	Q(m^3/d)	Re
Mangelson and Watters (1972)	12	6	0.45-0.9	2	0.06-0.15	218-517	343-840
Kilani and Ogunrombi (1984)	1	0.5	0.1	2	0.2	0.005	0.11
Silva et al (1995)	5-17	1.65-9	1.3-2.5	1-8	0.1-1.5	3.8-40	26-51
Ahmed et al (1996)	1	0.4	0.2	2.5	0.5	1.44	41
Muttamara and Puetpaiboon (1997)	1.5	0.5	0.15	3	0.1	0.075-0.75	1.7-17

where: L, W, h, Q and Re are Length, width, height, flow rate and Reynolds number respectively. For the present study, the range of length to width ratio for the facultative and maturation reactors modeled with the N-Tanks in series is presented in Table 4.7, as well as the flow rate and Reynolds number.

Table 4.7: Laboratory-scale system geometry and flow rate conditions in this study.

Author	L (m)	W (m)	h (m)	L/W	h/W	Q(m^3/d)	Re
Olukanni (2010)	3.9-7.4	0.35-0.7	0.40-0.65	5.55-21.7	0.9	0.12	303-304

Pedahzur *et al*. (1993) observed that baffles succeeded in channeling the influent flow but failed to increase considerably the retention time and the treatment efficiency of the pond.

While Aldana et al., (2005) reported that the hydraulic efficiency of a three-baffle basin increased as the gap between baffles and across reactor sides decreases, Lloyd et al., (2003) also reported that the increase of L/W ratio induced by longitudinal baffles can significantly increase fecal coliform removal efficiency in ponds. One of the main strength of the N-Tanks in series model is that as the N approaches ∞, the RTD curve for the tanks in series model approaches the RTD for ideal plug flow. This supports the fact that the shape of the residence time density function for the tanks in series is much more sensitive to changes in N.

4.4 Results of some selected simulation of faecal coliform inactivation for 80% Pond-width baffle Laboratory- scale reactors

Figures 4.32 - 4.37 describe the flow pattern and distribution of faecal coliform in the 80% pond with longitudinal and transverse arrangement for anaerobic, facultative and maturation reactors respectively. It can be seen from the diagrams that there are more red/yellow color in the first and second baffle compartments. This is an indication that as the wastewater travels round the baffles; the coliform inactivation process has been initiated until it gets to the minimum at the outlet where the color changes to blue. Shilton (2001) and Banda (2007) observed similar flow patterns using PHOENICS and FLUENT CFD tools respectively to simulate the hydraulic flow pattern in the laboratory ponds.

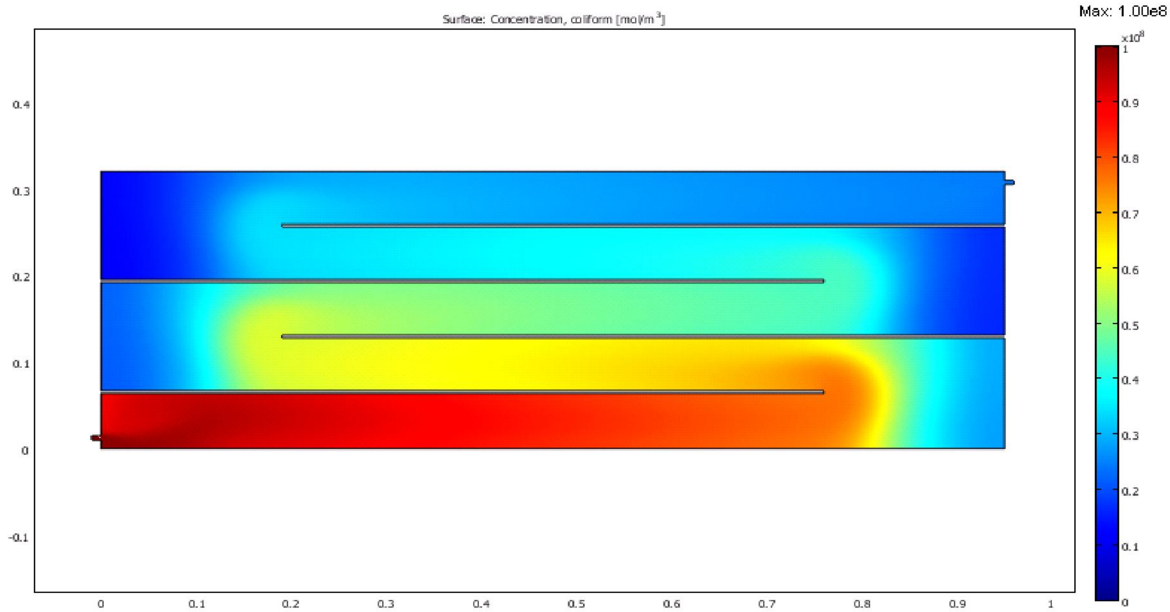

Figure 4.32 Faecal coliform inactivation in a longitudinal 4-baffle 80% pond width anaerobic reactor

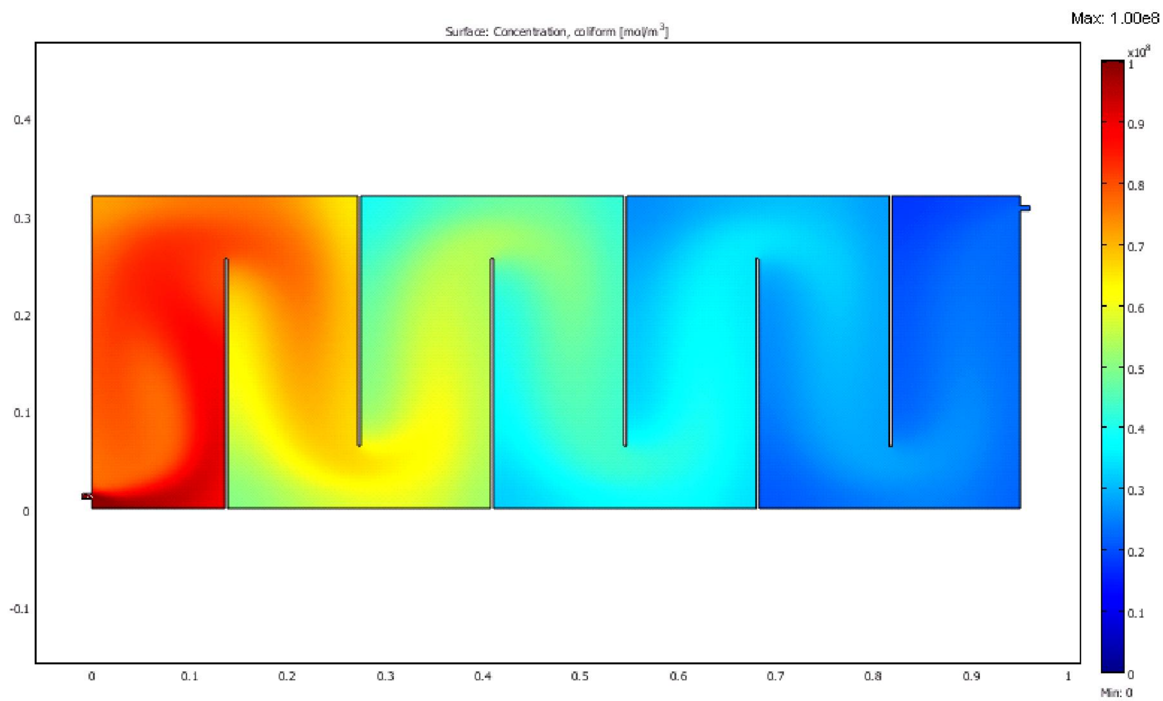

Figure 4.33 Faecal coliform inactivation in a transverse 6-baffle 80% pond width anaerobic reactor

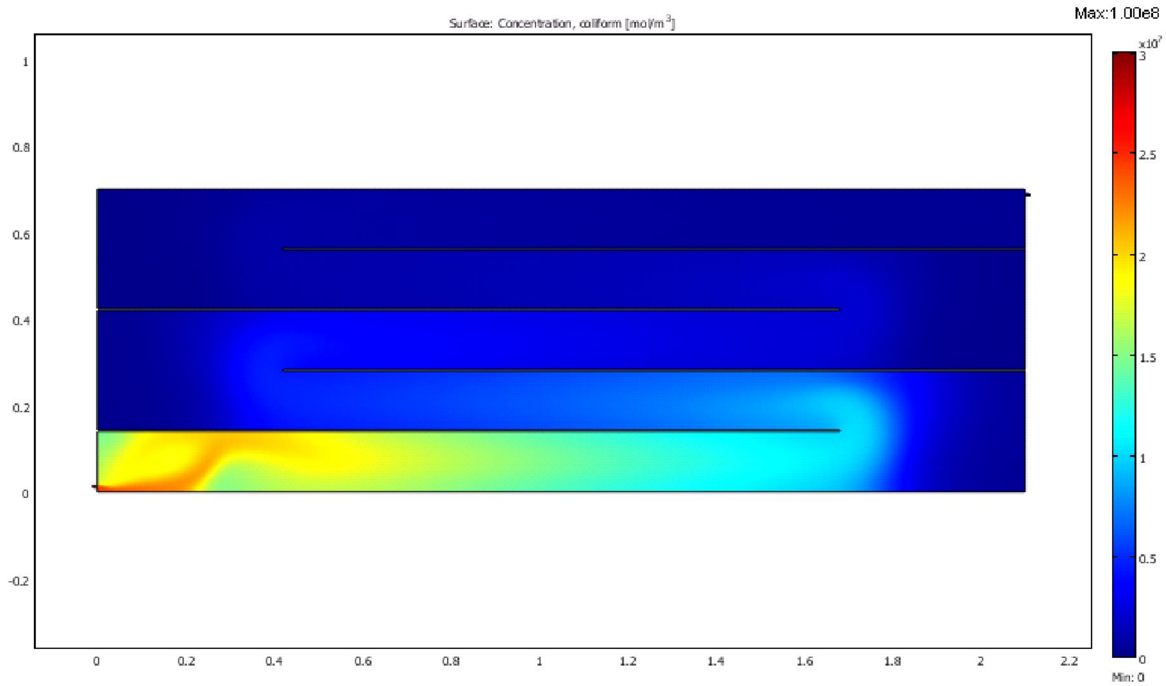

Figure 4.34 Faecal coliform inactivation in a longitudinal 4-baffle 80% pond width facultative reactor

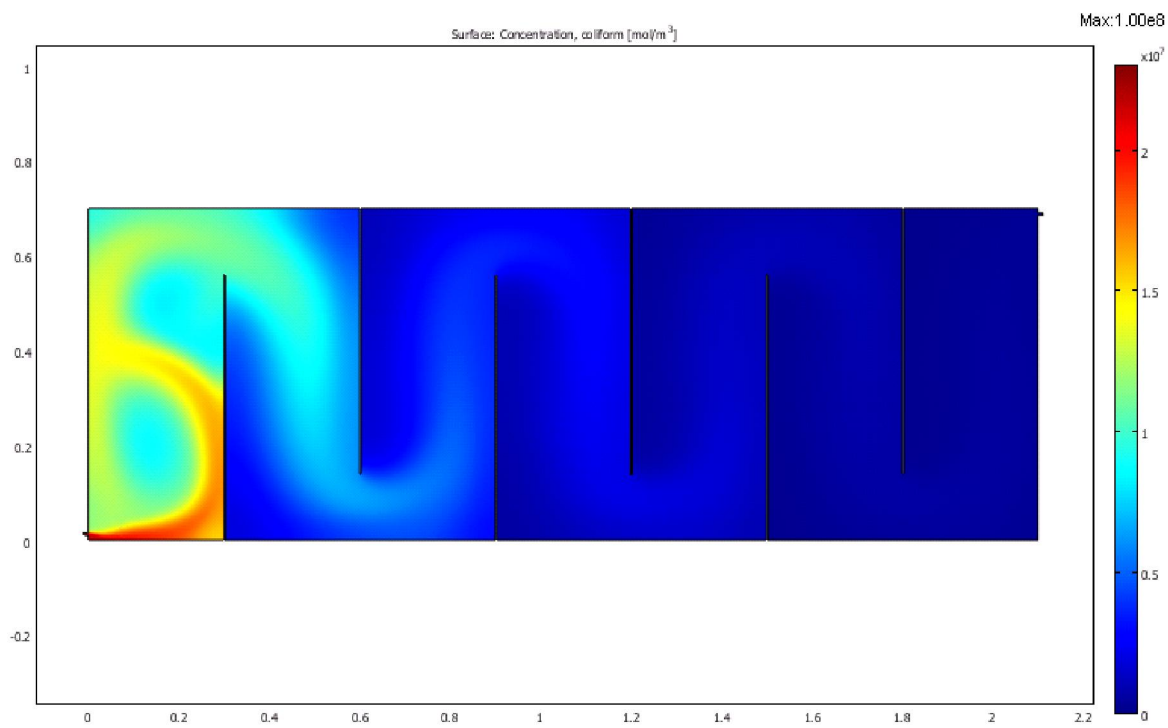

Figure 4.35 Faecal coliform inactivation in a transverse 6-baffle 80% pond width facultative reactor

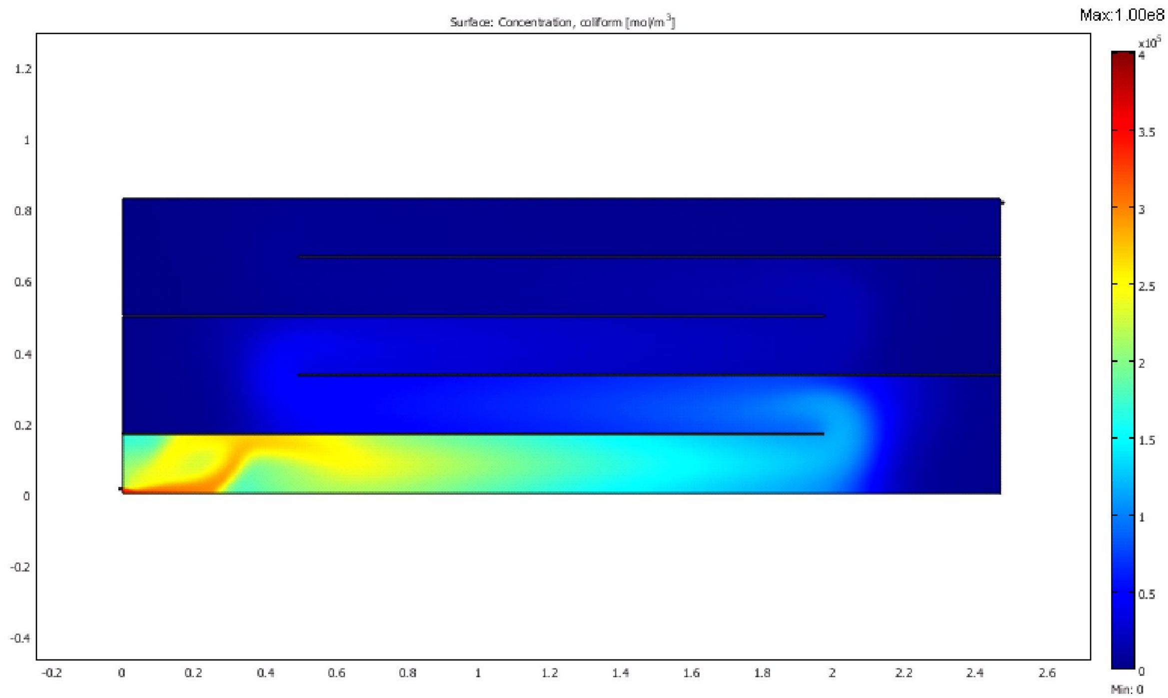

Figure 4.36 Faecal coliform inactivation in a longitudinal 4-baffle 80% pond width maturation reactor

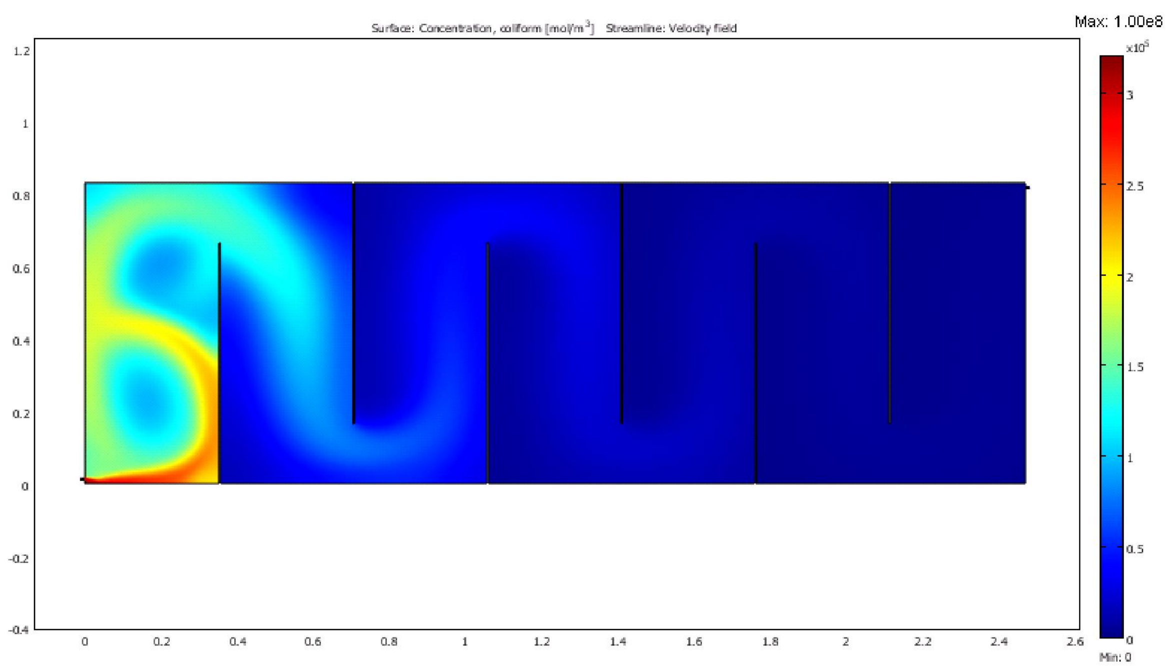

Figure 4.37 Faecal coliform inactivation in a transverse 4-baffle 80% pond width maturation reactor

The research investigation has shown that baffles can improve significantly the treatment efficiency and hydraulic performance of ponds and this can be one area of optimizing the design methods in reducing the land area requirements for the construction of waste stabilization ponds and also having optimal treatment. The result of the approach to optimization of waste stabilization pond design is considered in the next chapter.

4.5 Optimization model results

This section presents results of the model designs from the optimization process to determine the optimal designs that could be arrived at with minimum cost without compromising the treatment potentials of the reactors. It is important to estimate the economic costs and benefits of a range of selected configurations to improve effluent quality. The models were run based on the detailed illustrations described in chapter 3. Several designs were produced of which the optimal, minimum and maximum faecal coliform removal designs were selected for the purpose of comparison and for selection by any designer who would like to choose designs based on peculiar factors of interest.

The use of the optimization tool in this research has helped in finding a maximum reliability solution to satisfy specific cost objective. The outcome is an optimized model geometries that can predict precisely the velocity distribution, residence time distribution and faecal coliform concentration at all points in the reactor of which the reactor effluent and the accrued cost of material for construction are of utmost interest to the researcher.

4.5.1 The single objective SIMPLEX optimization configuration results

The history cost on designs for the even and odd transverse baffle arrangement together with the longitudinal baffle arrangement in anaerobic, facultative and maturation reactors based on SIMPLEX optimization are presented. This also includes the general arrangement of baffles of different length, configuration and numbers for optimal, minimum and maximum faecal coliform removal for each reactor in the optimization that was undertaken. The optimized designs, faecal coliform inactivation and flow pattern within the reactors for even and odd, transverse and longitudinal baffle arrangements are also presented (Figures 4.38–4.47). The optimization results of effluent faecal coliform with their associated costs in anaerobic, facultative and maturation reactors with conventional baffles of various configurations are presented in Tables 4.9 – 4.11.

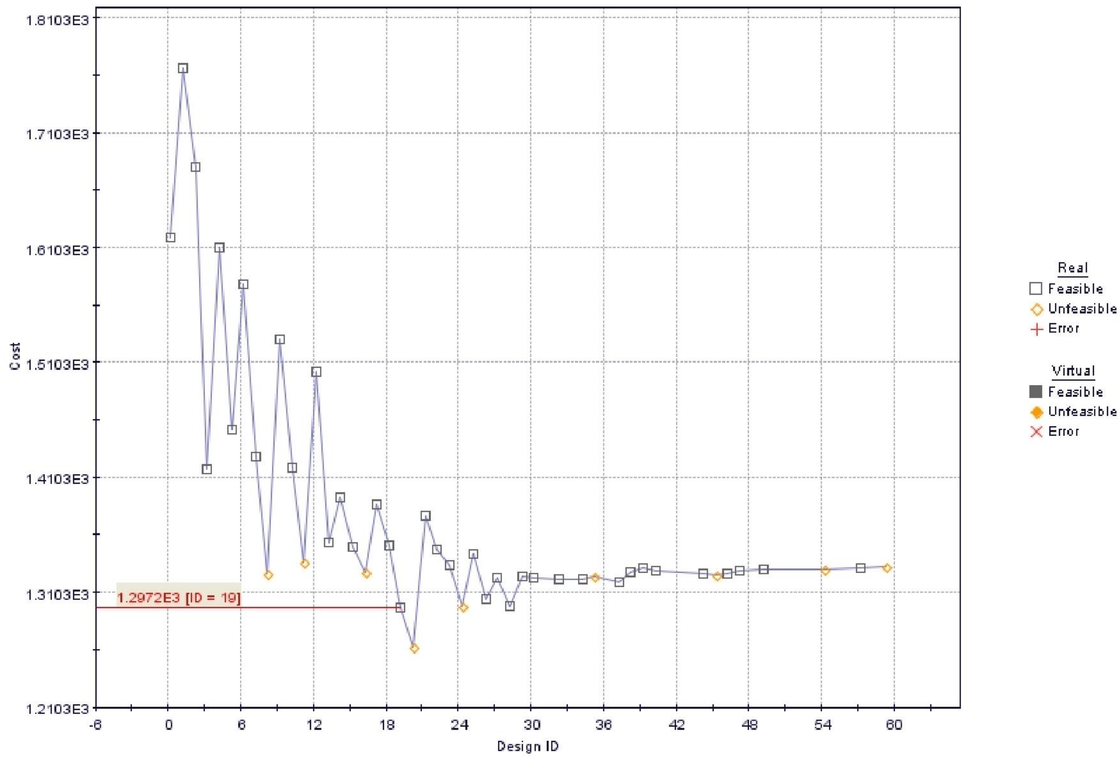

Figure 4.38 SIMPLEX history cost on design for the combination of even and odd Transverse baffle arrangement in anaerobic reactor

The history cost on designs for the combination of even and odd transverse baffle arrangement for the SIMPLEX algorithm which shows that the optimal design gives the optimized construction material cost of **N** 1,297.00 for a faecal coliform log removal of 0.61 with 3 baffles at 49% pond width and an area ratio of 3:1(Figure 4.38). Table 4.8 presents the results of transverse baffle arrangement of other configurations that were generated for the maximum, minimum and optimal designs for the three reactors while Figures 4.39-4.42 describes the CFD generated flow pattern of feacal coliform transport. The selected contours in Figures 4.39-4.42 are presented to demonstrate the uniqueness of the results provided in Table 4.8 to give more insight about the cost and performance data that were generated.

Table 4.8 SIMPLEX designs for transverse baffle arrangement

	Anaerobic Transverse			**Facultative Transverse**			**Maturation Transverse**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**2.45E-1**	2.45E-1	2.27E-1	2.27E-2	2.68E-2	8.39E-3	**1.54E-2**	2.11E-2	2.75E-3
Cost (N)	**1, 297**	1, 387	1, 419	5, 198	5, 676	5,404	**7, 986**	8, 095	8, 051
Log removal	**0.61**	0.61	0.64	1.64	1.57	2.08	**1.81**	1.68	2.51
Area ratio	**3:1**	4:1	3:1	2:1	2:1	2:1	**4:1**	4:1	4:1
Area (m^2)	**1.72E-1**	2.17E-1	1.65E-1	1.38E0	1.53E0	1.38E0	**2.28E0**	2.35E0	2.28E0
Depth (m)	**1.15E-1**	9.12E-2	1.20E-1	4.80E-2	4.32E-2	4.80E-2	**3.60E-2**	3.49E-2	3.60E-2
Length (m)	**7.17E-1**	9.31E-1	7.03E-1	1.66E0	1.75E0	1.66E0	**3.02E0**	3.07E0	3.02E0
Width (m)	**2.39E-1**	2.33E-1	2.34E-1	8.30E-1	8.75E-1	8.30E-1	**7.55E-1**	7.66E-1	7.55E-1
Velocity (m/s)	**2.41E-3**	3.05E-3	2.32E-3	5.79E-3	6.44E-3	5.79E-3	**7.72E-3**	7.96E-3	7.72E-3
Baffle length (m)	**1.17E-1**	9.19E-2	1.17E-1	4.82E-1	5.16E-1	6.39E-1	**5.21E-1**	4.67E-1	5.32E-1
Baffle ratio	**49%**	40%	50%	58%	59%	77%	**69%**	61%	71%
Number of baffles	**3**	4	6	5	6	6	**6**	5	7

Table 4.8 summarizes all the properties of the design configuration for the combination of even and odd transverse baffle arrangement using single objective SIMPLEX in achieving the optimization cost objective. It should be mentioned that though SIMPLEX is a singular objective solving tool and searches in the local optimal, it can be said that it predicted well the cost of all the materials for constructing the reactors.

The selection of any design would be based on the engineers' discretion if at all the optimal design which gives the minimum cost is not to be chosen for a particular reason. However, it should be borne in mind that when other cost (labor, construction, maintenance and other expenses) are included, the designer may be forced to consider the optimal design specification.

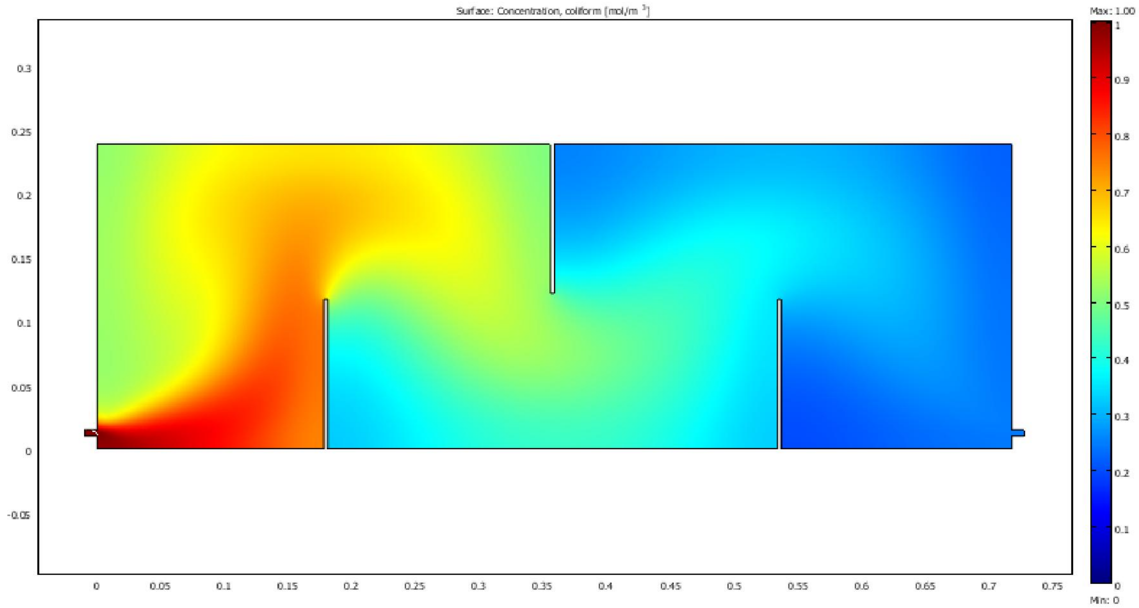

Figure 4.39 SIMPLEX optimal faecal coliform removal design with least cost for the combination of even and odd transverse baffle arrangement in anaerobic reactor

Figure 4.39 shows the Simplex design flow pattern and faecal coliform distribution on a ratio 3:1 surface plane at a depth 0.115 m below the anaerobic reactor surface. It consist of 3 baffles at about the average length of the reactor width (49% pond width) with a flow channel width of 0.177 m in the baffle compartment and a baffle opening of 0.122 m. This present a case of the inclusion of odd baffle configuration in the reactor geometry which placed the outlet on the same position with the inlet on the opposite side. With this configuration, a cummulatie log removal of 0.61 was achieved at the outlet. The velocity achieved at the inlet has a value of 2.41×10^{-3} m/s. It can be seen from the contour of faecal coliform distribution of wastewater that there are red and faint yellow colours at the inlet and as it moves from one baffle compartment to the other, the colour gradually changes through yellow to light blue at the outlet. The introduction of spatial mixing due to perpendicular placement of baffle to the flow direction in the first baffle compartment makes it difficult to achieve plug flow condition. The scale of 0 to 1 on the right side describes the extent of faecal coliform removal/ treatment in the reactor having used a dimensionless unit as detailed in chapter three. With the positioning of this baffles, the effect of short circuiting has been reduced and there is a significant visible difference of flow pattern as the wastewater travels from one baffle compartment to the other.

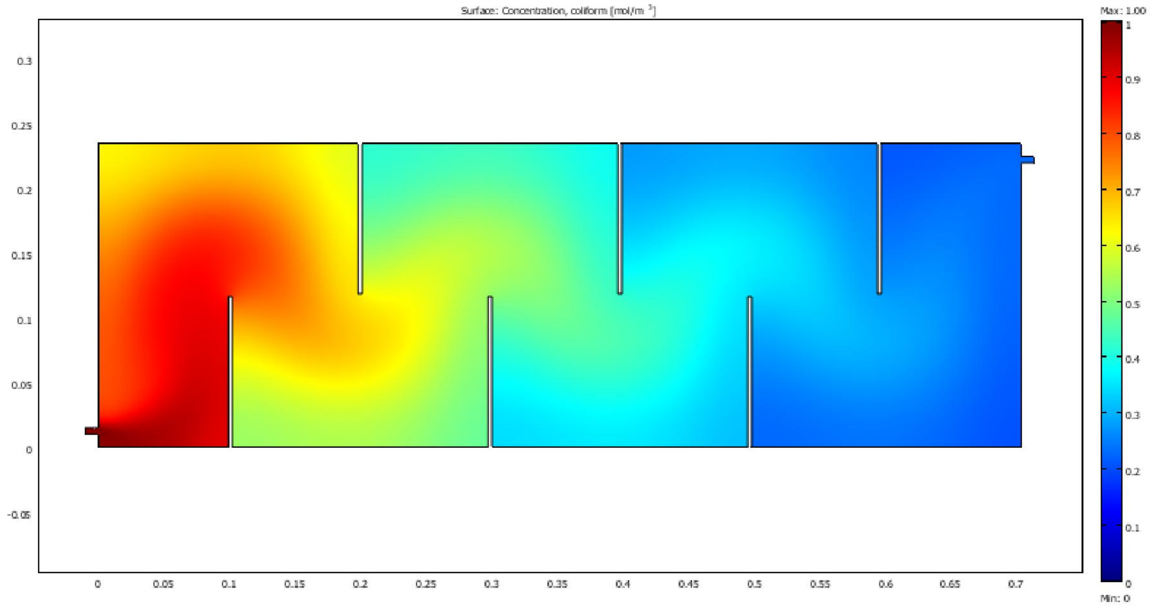

Figure 4.40 SIMPLEX maximum faecal coliform removal design for combination of even and odd transverse baffle arrangement in anaerobic reactor

Figure 4.40 shows the maximum FC removal simplex design flow pattern and faecal coliform distribution on a ratio 3:1 surface plane at a depth 0.12 m below the anaerobic reactor surface. It consist of 6 baffles with each baffle at 50% pond width. The flow channel width and the baffle opening are 0.098 m and 0.117 m respectively. This figure shows that when baffles are placed closed to the inlet, it forces the wastewater to circulate around in the first compartment and allow thorough mixing to occur. With this configuration, a cummulatie log removal of 0.64 was achieved at the outlet. The velocity achieved at the inlet has a value of 2.32×10^{-3} m/s. The contour of faecal coliform distribution of wastewater shows there is purely red colour at the inlet and as it moves from one baffle compartment to the other, the colour gradually changes through yellow to light blue at the outlet. The assertion that the use of 50% pond-width baffles as observed by Shilton and Harrison (2003a) when fitted along the longitudinal axis of the pond could deteriorate the hydraulic performance of baffled ponds could be questioned because the outcome in this research shows that the 50% pond-width baffle does better than what the authors claimed. This could be due to the inlet and outlet positioning of the system placed at the centre of one end and at the centre of another end of the pond in their research.

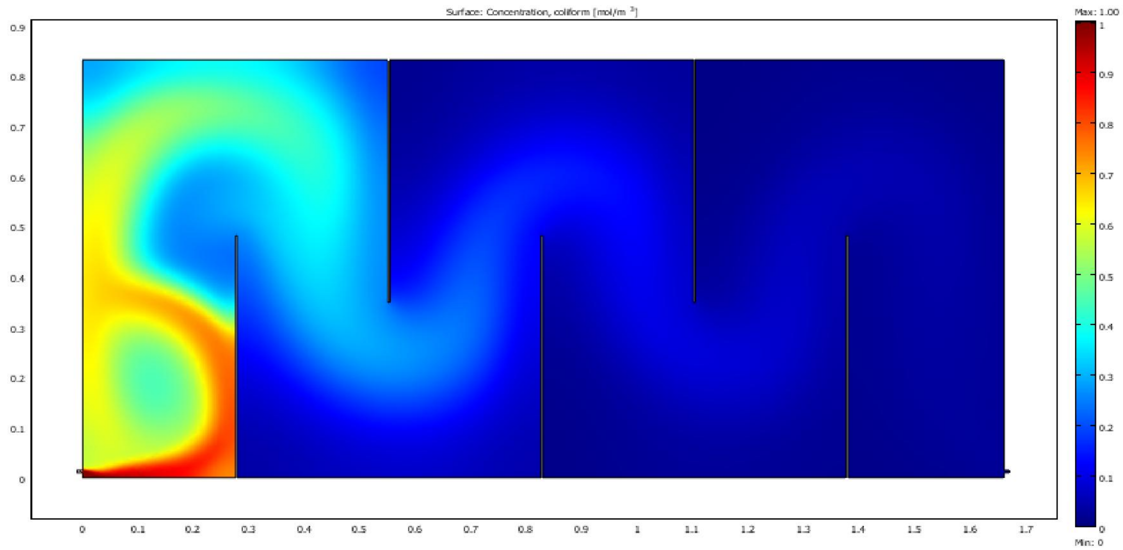

Figure 4.41 SIMPLEX optimal faecal coliform removal design with least cost for the combination of even and odd transverse baffle arrangemnt in facultative reactor

Figure 4.41 shows the Simplex design flow pattern and faecal coliform distribution on a ratio 2:1 surface plane at a depth 0.048 m below the facultative reactor surface. It consist of 5 baffles with baffle lengths of 0.482 m (58% pond-width) with a flow channel width of 0.274 m in the baffle compartment and a baffle opening of 0.348 m. This present a case of the inclusion of odd baffle configuration in the reactor geometry as represented in the anaerobic optimal design which placed the outlet on the same position with the inlet on the opposite side. The recirculating flow pattern caused by spatial mixing in the first baffle compartment makes it impossible to achieve a plug flow condition. This shows that odd baffle configurations are to be considered in baffled wastewater treatment system performance and efficiency. With this configuration, a cummulative log removal of 1.64 was achieved at the outlet. The velocity achieved at the inlet has a value of 5.79×10^{-3} m/s. It can be seen also that there are significant visible difference of flow pattern as the wastewater travels from one baffle compartment to the other. There are no contours directly linking the inlet to the outlet so no short-circuiting as could be compared to the case of unbaffled reactor. It is interesting to note that Shilton and Harrison (2003a, 2003b) and Banda (2007) using PHOENICS and FLUENT CFD softwares respectively predicted the same other of about 2 log-units removal with an isothermal condition.

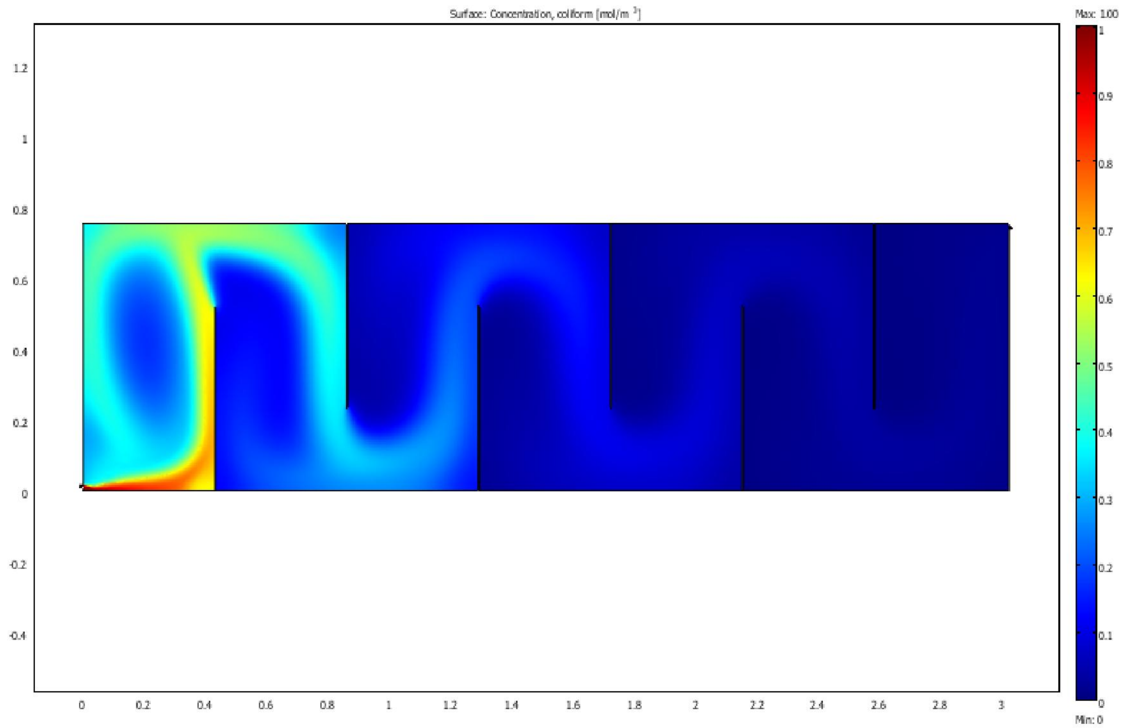

Figure 4.42 SIMPLEX optimal faecal coliform removal design with least cost for combination of even and odd transverse baffle arrangement in maturation reactor

Figure 4.42 shows the Simplex design flow pattern and faecal coliform distribution on a ratio 4:1 surface plane at a depth 0.036 m below the maturation reactor surface. It consist of 6 baffles with baffle lengths of 0.521 m (69% pond width) with a flow channel width of 0.429 m in the baffle compartment and a baffle opening of 0.234 m. With this configuration, a cummulative log removal of 1.81 was achieved at the outlet. This shows that an increase in the bafle length and numbers would increase the performance of the reactor. The velocity achieved at the inlet has a value of 7.72×10^{-3} m/s. There is significant visible difference of flow pattern in the first and last three baffle compartments. It can be seen that the baffle in the first compartment forced a circulating flow pattern that allows mixing to occur and there is a minimal stagnation region in the baffle compartment. It can be seen that there are no short-circuiting visible at this level as there are no contours directly linking the inlet to the outlet.

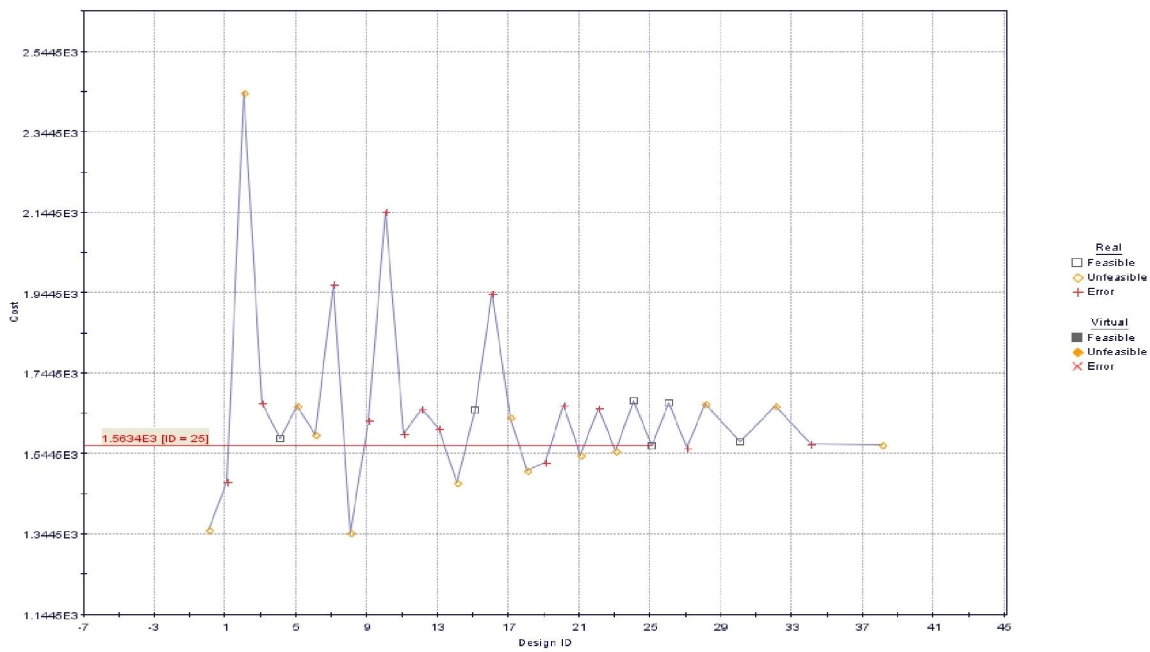

Figure 4.43 SIMPLEX history cost on design for combination of even and odd longitudinal baffle arrangement in anaerobic reactor

Figure 4.43 shows the history cost on designs for the combination of even and odd longitudinal baffle arrangement for the anaerobic reactor. SIMPLEX algorithm shows that optimal design selected gives the optimized construction material cost of **N** 1,563.00 for a faecal coliform log removal of 0.63 with 5 baffles at 67% pond width and an area ratio of 1:1. Table 4.9 describes other properties of this design.

The summary of all the properties of the design configuration for the combination of even and odd longitudinal baffle arrangement using simplex algorithm in achieving the optimization cost objective are presented in Table 4.9. Although the longitudinal baffle arrangement could seem expensive when compared with the transverse arrangement, the selection of any design would be based on the engineers' discretion if at all the optimal design which gives the minimum cost is not to be chosen for a particular reason.

Table 4.9 SIMPLEX designs for longitudinal baffle arrangement

	Anaerobic Longitudinal			Facultative Longitudinal			Maturation Longitudinal		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	2.36E-1	2.46E-1	2.28E-1	**2.40E-2**	2.40E-2	8.70E-3	3.78E-3	8.97E-3	2.59E-3
Cost (N)	1, 563	1, 653	1, 582	**5, 091**	5, 091	5, 270	8, 046	8, 134	8,183
Log removal	0.63	0.61	0.64	**1.62**	1.62	2.06	2.42	2.05	2.56
Area ratio	1:1	1:1	1:1	**1:1**	1:1	2:1	3:1	4:1	4:1
Area (m^2)	1.68E-1	1.75E-1	1.68E-1	**1.38E0**	1.38E0	1.38E0	2.27E0	2.28E0	2.28E0
Depth (m)	1.18E-1	1.13E-1	1.18E-1	**4.80E-2**	4.80E-2	4.80E-2	3.60E-2	3.60E-2	3.60E-2
Length (m)	4.10E-1	4.19E-1	4.10E-1	**1.17E0**	1.17E0	1.66E0	2.61E0	3.02E0	3.02E0
Width (m)	4.10E-1	4.19E-1	4.10E-1	**1.17E0**	1.17E0	8.30E-1	8.71E-1	7.55E-3	7.55E-3
Velocity (m/s)	2.36E-3	2.46E-3	2.36E-3	**5.79E-1**	5.79E-3	5.79E-3	7.72E-3	7.72E-3	7.72E-3
Baffle length (m)	2.73E-1	2.76E-1	2.83E-1	**9.74E-1**	9.74E-1	1.46E0	2.13E0	2.25E0	2.48E0
Baffle ratio	67%	66%	69%	**83%**	83%	88%	82%	75%	82%
Number of baffles	5	6	5	**2**	2	2	2	2	2

The selected contours in Figures 4.44 - 4.47 describing the CFD generated flow pattern of feacal coliform transport are presented to demonstrate the uniqueness of the results provided in Table 4.9 as well as to give more insight about the cost and performance data that were generated.

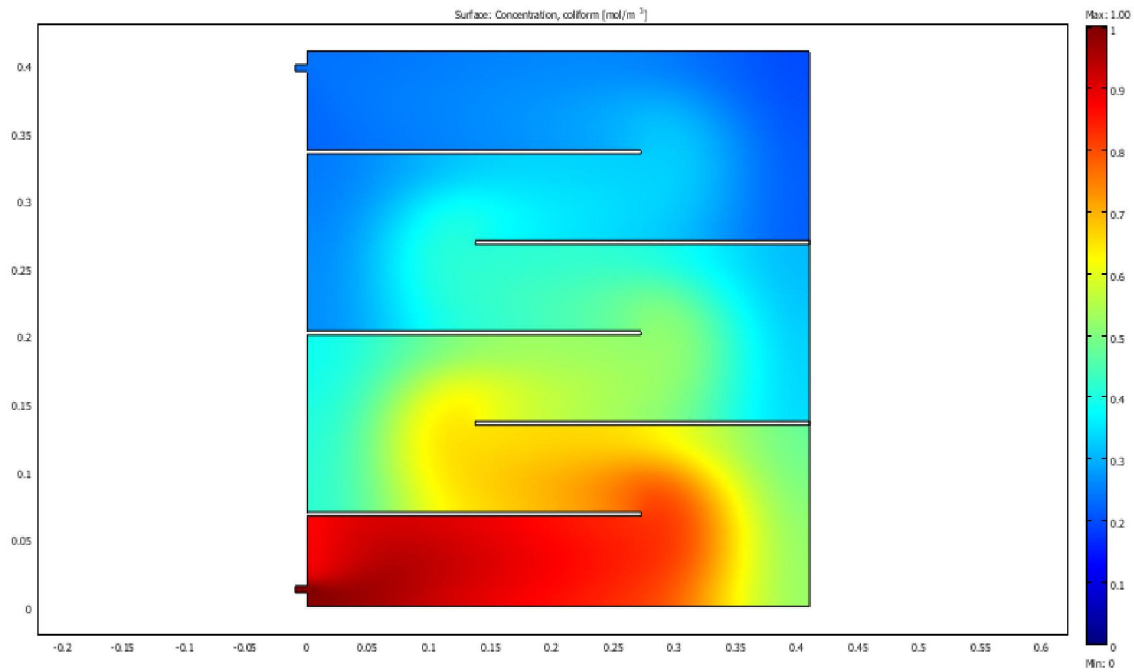

Figure 4.44 SIMPLEX optimal faecal coliform removal design with least cost for combination of even and odd longitudinal baffle arrangement in anaerobic reactor

The five-baffle anaerobic reactor in Figure 4.44 shows a surface area ratio 1:1 of the simplex design flow pattern on a surface plane at a depth 0.118 m below the water surface for the optimal FC removal. The reactor baffle lengths are 0.273 m (67% pond width) spread evenly at a flow channel width of 0.066 m in the baffle compartment and a baffle opening of 0.137 m. With this configuration, a cummulatie log removal of 0.63 was achieved at the outlet. It can be seen that there are significant visible difference of flow pattern as the wastewater travels from one baffle compartment to the other. With the positioning of the 5-baffles, the colour in the fifth baffle compartment is almost blue. The gradual flow pattern in the first baffle compartment allows wastewater to transit steadily and makes mixing to occur. The inlet and the outlet are on the same side and this type of reactors can be placed side by side to form waste stabilization pond in series by connecting one outlet with another inlet with a u-shaped connector.

The researcher`s finding reveals that the inclusion of longituidnal baffle arrangment in waste stabilzation pond design is limited. This may be attibuted to cost of placing a long baffles along the length of a pond. Though Abbas et al. (2006) investigated the effect of even baffle longitudinal arrangement, the author could not find further literature that documents the performance and treatment efficiency of an odd longituidal baffle configuration in waste stabilization pond. This has been one of the question and hypothesis possed by the researcher to investigate into the hydraulic performance of odd transverse and longitudinal baffle arrangement. However, it was expressed by Watters et al. (1973) in Shilton and Harrison (2003a) that for a comparable length of baffling, essentially the same result was achieved as for transverse baffling across the pond width.

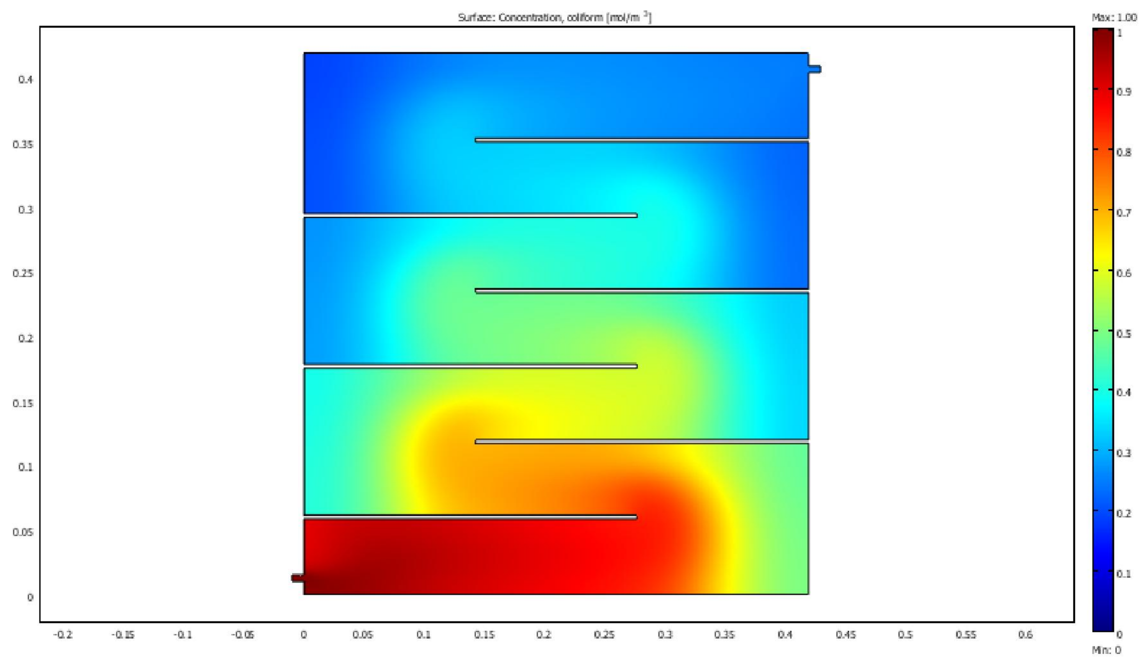

Figure 4.45 SIMPLEX minimum faecal coliform removal design for combination of even and odd longitudinal baffle arrangement in anaerobic reactor

Figure 4.45 shows the Simplex design flow pattern on a ratio 1:1 surface plane at a depth 0.113 m below the anaerobic reactor surface. It is interesting to know that inspite of the increase in the baffles to 6 baffles, there was no improvement in the efficiency as compared to the optimal design. This may support the observation of Pedahzur *et al.* (1993) that baffles succeeded in channeling the influent flow but failed to increase considerably the retention time and the treatment efficiency of the pond. The baffle lengths of 0.276 m (66% pond width) with a flow channel width of 0.596 m in the baffle compartment and a baffle opening of 0.143 m. With this configuration, a cummulative log removal of 0.61 was achieved at the outlet. There is significant visible difference of flow pattern in the first and last two baffle compartments which shows that there is improved mixing of wastewater in each baffle compartment.

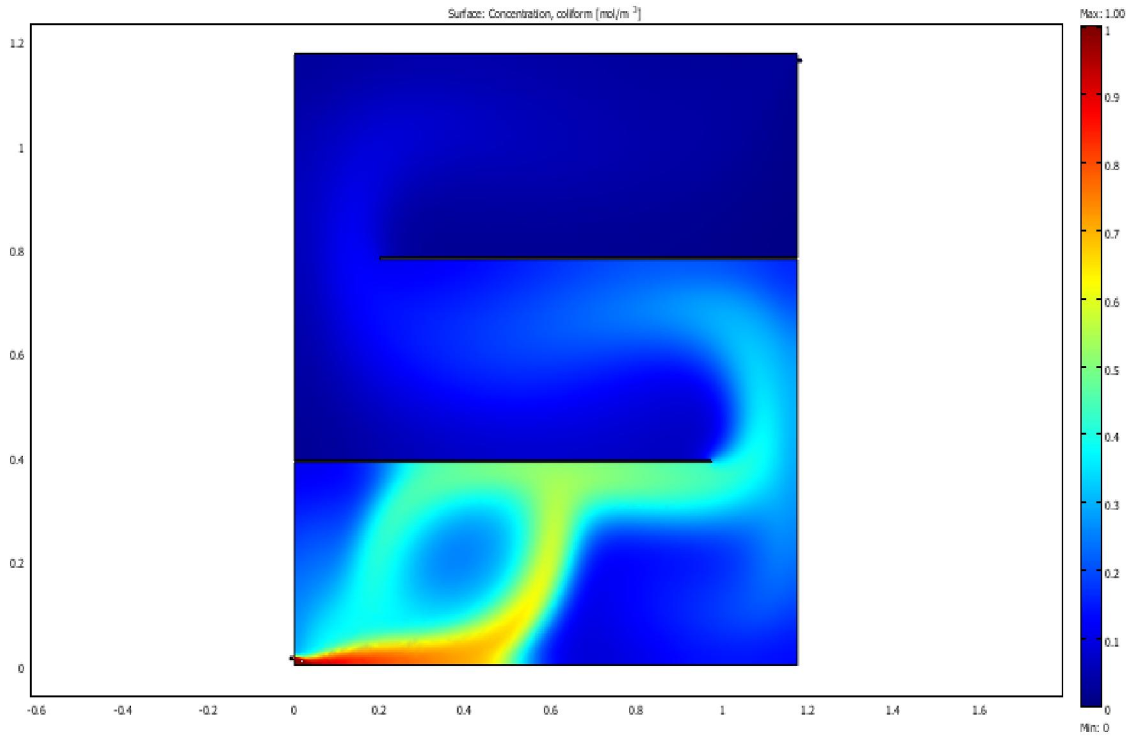

Figure 4.46 SIMPLEX optimal faecal coliform removal design for combination of even and odd longitudinal baffle arrangement in facultative reactor

Figure 4.46 shows the Simplex design flow pattern and faecal coliform distribution on a ratio 1:1 surface plane at a depth 0.048 m below the facultative reactor surface. It consist of 2 baffles with baffle lengths of 0.974 m (83% pond width) with a flow channel width of 0.388 m in the baffle compartment and a baffle opening of 0.190 m. With this configuration, a cummulative log removal of 1.62 was achieved at the outlet. The velocity achieved at the inlet has a value of 5.79×10^{-3} m/s. There is significant visible difference of flow pattern in the first baffle compartment. The circulating flow pattern in the first baffle opening compartment shows that there is circulating region that allows mixing to occur.

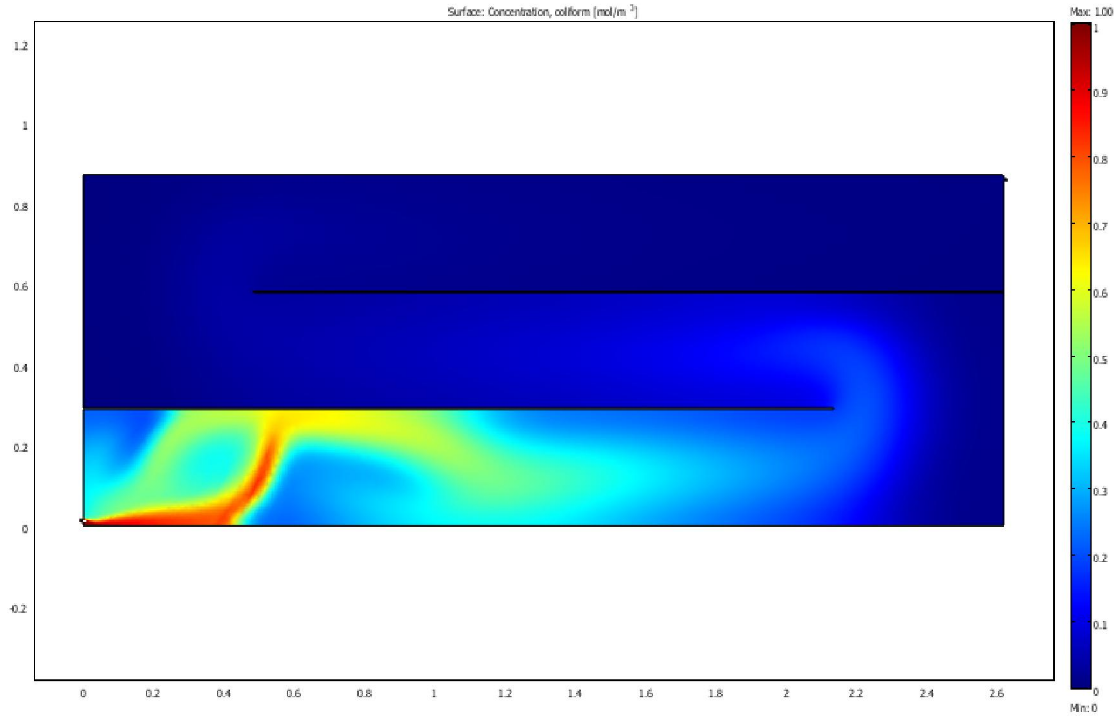

Figure 4.47 SIMPLEX optimal faecal coliform removal design for combination of even and odd longitudinal baffle arrangement in maturation reactor

Figure 4.47 describes the Simplex design flow pattern and faecal coliform distribution on a ratio 3:1 surface plane at a depth 0.036 m below the maturation reactor surface. It consist of 2 baffles with baffle lengths of 2.13 m (82% pond width) with a flow channel width of 0.288 m in the baffle compartment and a baffle opening of 0.48 m. With this configuration, a cummulative log removal of 2.42 was achieved at the outlet. The velocity achieved at the inlet has a value of 7.72×10^{-3} m/s. There is significant visible difference of flow pattern in the first and third baffle compartments.

In the overall consideration in both the transverse and longitudinal configurations from Tables 4.2, 4.3, 4.8 and 4.9 with cost objective in mind, one could conclude that the designs represented in Table 4.10 are representative of the overall optimal design results for the three reactors (anaerobic and maturation transverse arrangement) while one design was chosen for the longitudinal facultative reactor (Table 4.9) for pollutant removal in WSP. This is due to the fact that it achieved the optimal cost objective in the simplex optimization exercise.

Table 4.10 Simplex Optimal design results

	Anaerobic Transverse			Facultative Longitudinal			Maturation Transverse		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 297**	1, 387	1, 419	**5, 091**	5, 091	5, 270	**7,221**	8, 095	8, 051
Log removal	**0.61**	0.61	0.64	**1.62**	1.62	2.06	**1.84**	1.68	2.51
Area ratio	**3:1**	4:1	3:1	**1:1**	1:1	2:1	**3:1**	4:1	4:1
Depth (m)	**1.15E-1**	9.12E-2	1.20E-1	**4.80E-2**	4.80E-2	4.80E-2	**4.00E-2**	3.49E-2	3.60E-2
Baffle ratio	**49%**	40%	50%	**83%**	83%	88%	**70%**	61%	71%
Number of baffles	**3**	4	6	**2**	2	2	**4**	5	7

When investigating the anaerobic pond cost, the same log removal was achieved with a substantial reduction in cost (approximately N300 less) over the transverse design results in Table 4.2. This lower cost was achieved by reducing the baffle length to 49%, having one less baffle, and using a deeper reactor. A similar cost reduction was also predicted for the facultative pond with an improved log reduction (1.62 vs 1.49). In the facultative reactor design, the cost reduction was achieved by utilizing a rectangular reactor (i.e., reactor area ratio was 1:1), with two fewer baffles that were each longer than 80%. However, the maturation result is similar to the result predicted in Table 4.2 suggesting that the 70 % baffle width could be an optimal configuration given the maturation design constraints.

Figures 4.39, 4.42 and 4.46 shows that the presence of the baffles clearly reduce the extent of mixing and short circuiting that may occur in these ponds. This observation is not new and is well recognized in the literature. It does, however, explain how optimizing around cost with appropriate constraints to meet effluent microbial log reduction can achieve non-intuitive results that deviate from previously reported WSP design configurations in the literature (Banda 2007; Shilton and Harrison, 2003a; Sperling et al. 2002). The question that still remains is whether the optimal results predicted using the SIMPLEX optimization was the global optimum WSP design or a local optimum due to the complexity of the search space.

4.5.2 The Multi-objective MOGA II optimization configuration results

MOGA II algorithm, an efficient multi-objective genetic algorithm uses a smart multi-search elitism which is able to preserve excellent solutions at the global optimal without bringing premature convergence to local-optimal frontiers. Figures 4.48-4.50 show the history cost on designs for the combination of even and odd transverse baffle arrangement for the anaerobic, facultative and maturation reactors respectively.

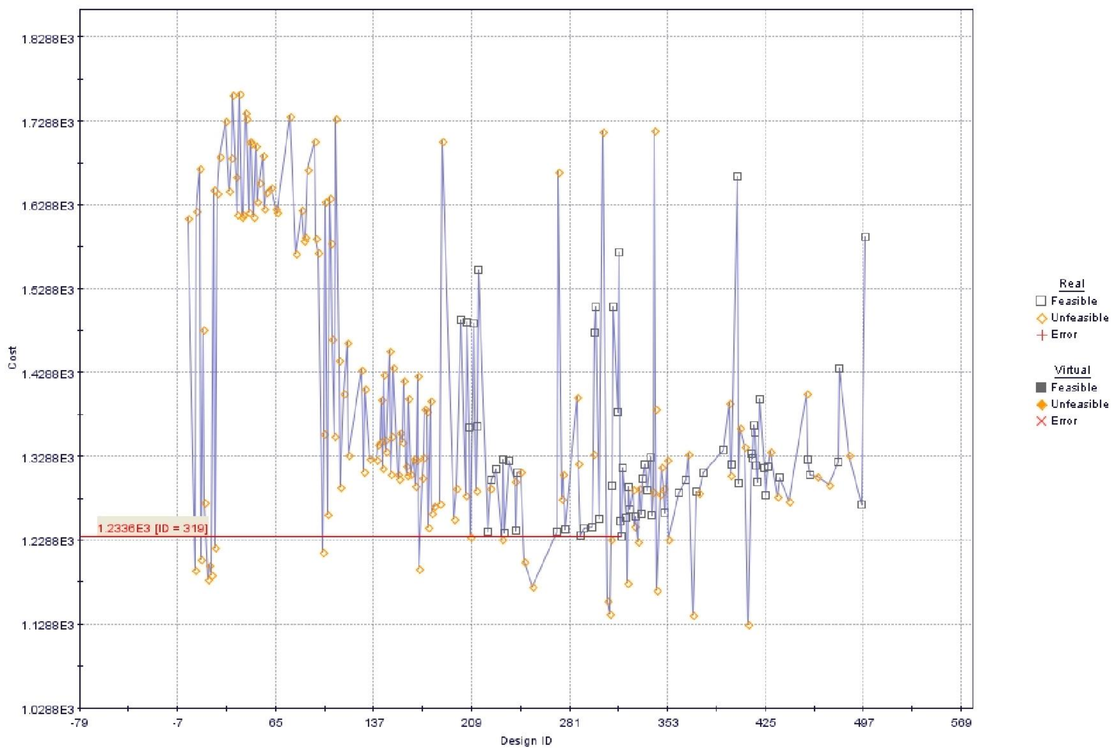

Figure 4.48 MOGA-II history cost on design for transverse baffle arrangement in anaerobic reactor

The algorithm shows that design ID 319 gives the optimized construction material cost of **N** 1,234.00 for a faecal coliform log removal of 0.60 with 2 baffles at 58% pond width and an area ratio of 2:1 (Figure 4.48). This optimal design configuration happens to be the optimal FC removal which is an indication that the best scenario for cost objective function has been achieved. The other properties of the design are presented in Table 4.11.

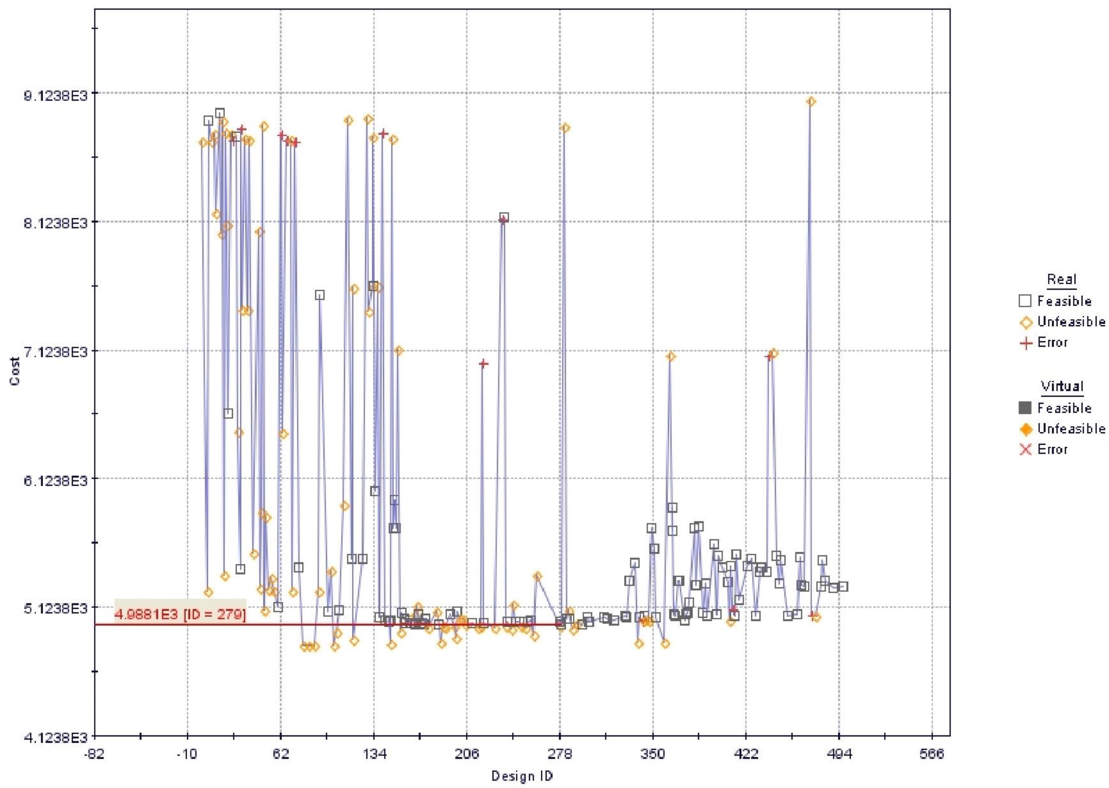

Figure 4.49 MOGA-II history cost on design for transverse baffle arrangement in facultative reactor

The MOGA II algorithm in Figure 4.49 shows that the optimal design gives the optimized construction material cost of **N** 4,988.00 for a faecal coliform log removal of 1.51 with 2 baffles at 53% pond width and an area ratio of 1:1. This design configuration happens to be the overall optimal design when comparing both transverse and longitudinal baffle arrangement in MOGA II algorithm design. It is an indication that this is best design in considering two objective functions (minimizing cost and FC output). The other properties of the design are presented in Table 4.11.

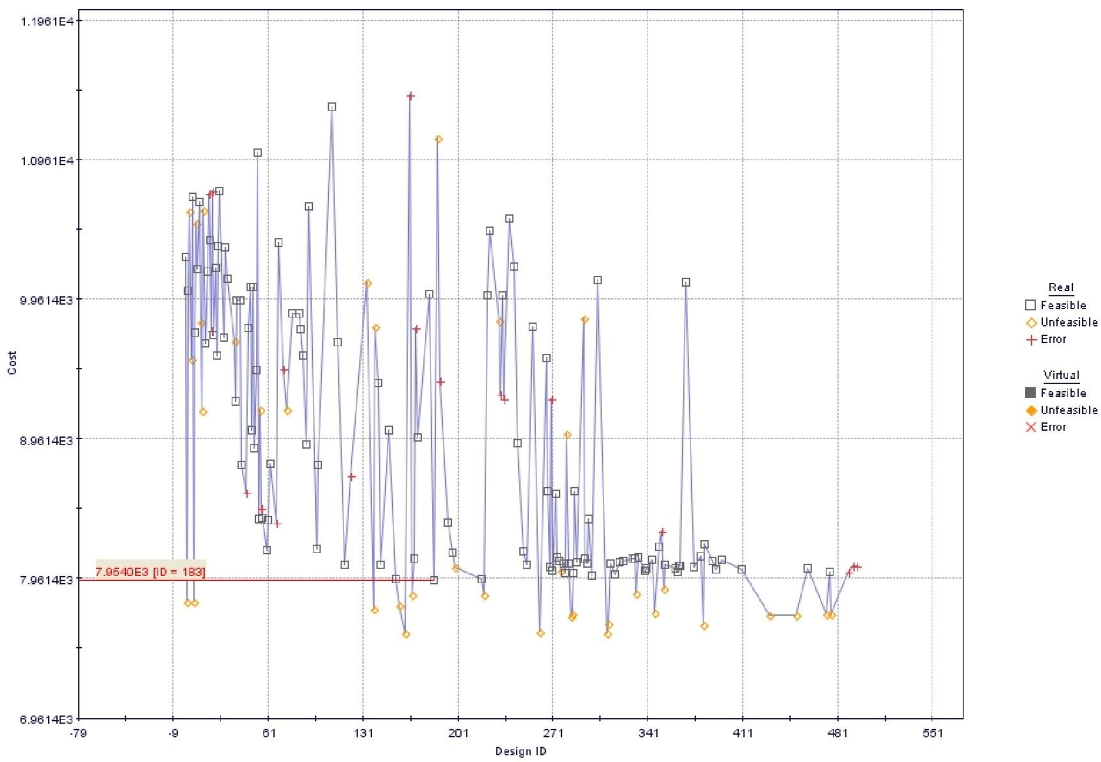

Figure 4.50 MOGA-II history cost on design for transverse baffle arrangement in maturation reactor

Figure 4.50 shows the history cost on designs for the combination of even and odd transverse baffle arrangement for the maturation reactor. The MOGA II algorithm shows that design ID 183 gives the optimized construction material cost of **N** 7,954.00 for a faecal coliform log removal of 1.74 with 4 baffles at 93% pond width and an area ratio of 2:1. The other properties of the design are presented in Table 4.11.

Figures 4.51-4.53 present a trade off plot/pareto front to display the contradiction when trying to maximize the reactors effluent feacal log reduction in an attempt to minimizing the construction cost. Though the cost and the fecal reduction depend on other model parameters (length, width, depth, baffle length and numbers), the plot shows the range of reasonable possibilities in the design space for the three reactors. In the fronts, feasible region contains solutions that satisfy the constraints for both log reduction and the cost objectives. One may not be able to plot a curve because there is no solution that improves at the same time both of the objectives. In this front, the selection of any design solution is based on the designers' preference. However, the optimal, minimum and the maximum design solution has been selected and presented in Table 4.11. The optimal design is the best compromise between the 2 objectives.

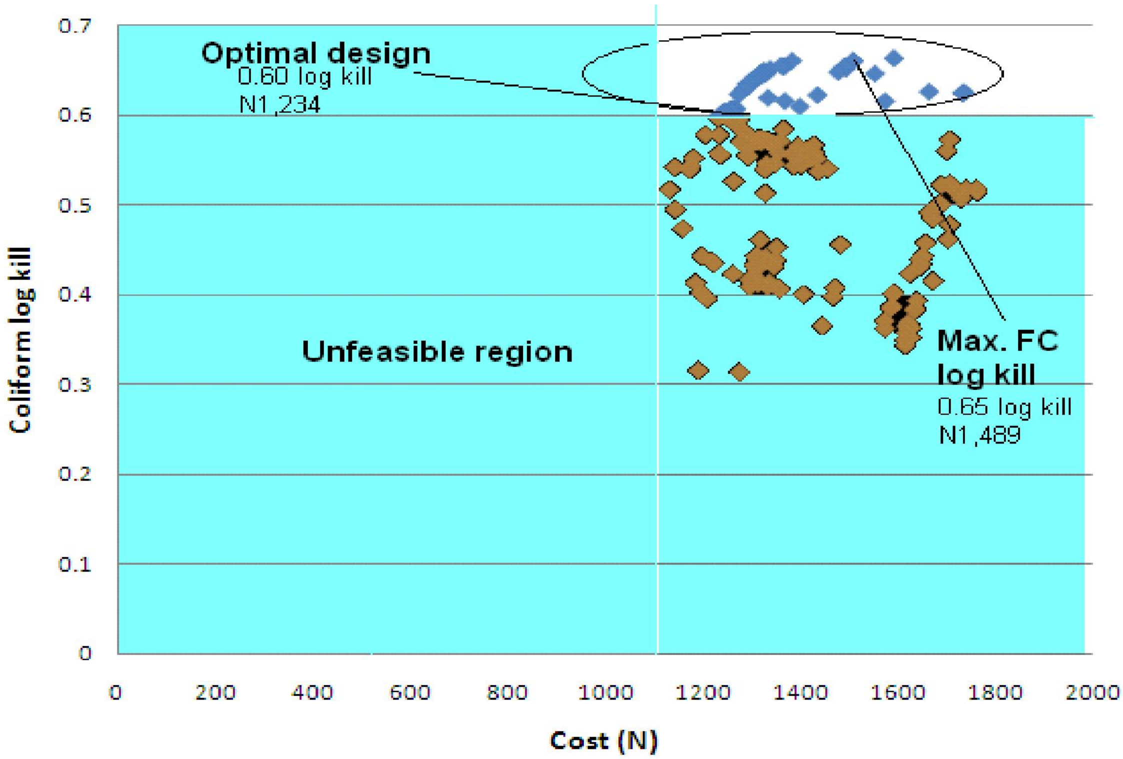

Figure 4.51 Trade off plot for Anaerobic Transverse design space.

Figure 4.51 presents feasible region containing solution that satisfy the constraint of log kill greater than or equal to 0.6 (68 designs of a total of 216) in the optimization process. The optimal design produced a feacal log kill of 0.60 with an associated cost of N1, 234 while the maximum FC log kill attracts an extra cost of N255 to achieve an extra log kill of 0.05. It is interesting to note that the same solution of 0.60 log kill was achieved for both the optimal and minimum feacal reduction as presented in Table 4.11. The cost is a function of the reactor geometry and the number/placement of baffles in the reactor. The unfeasible region represents solutions of feacal log kill that are below the minimum constraint of 0.60 that was set in the optimization algorithm to achieve a target feacal reduction. The optimal, minimum and maximum design solutions has been selected and presented in Table 4.11.

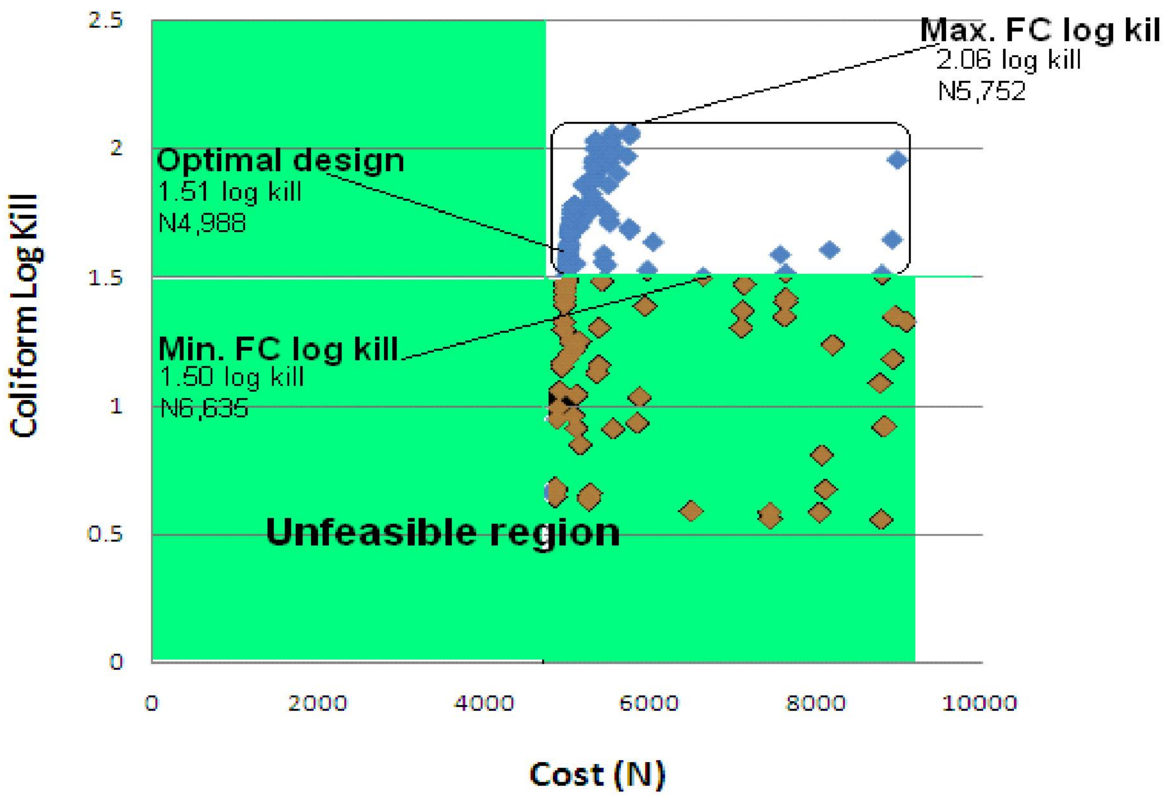

Figure 4.52 Trade off plot for Facultative Transverse design space.

Figure 4.52 presents feasible region containing the optimal, minimum and maximum solutions that satisfy the constraint of log kill greater than or equal to 1.50 (105 of a total of 175 feasible designs generated) in the optimization process. The optimal design produced a feacal log kill of 1.51 with a cost of N4, 988 while the increase in cost achieves a better performance of feacal reduction for the maximum log kill. However, further increase in cost as to N6, 635 gave the minimum FC log kill. This shows that the log kill is not strictly a function of the cost per say but that of the reactor geometry and other parameter that is processing the contaminant. The unfeasible region represents solutions of feacal log kill that are below the minimum constraint of 1.50 that was set in the optimization algorithm to achieve a target feacal reduction.

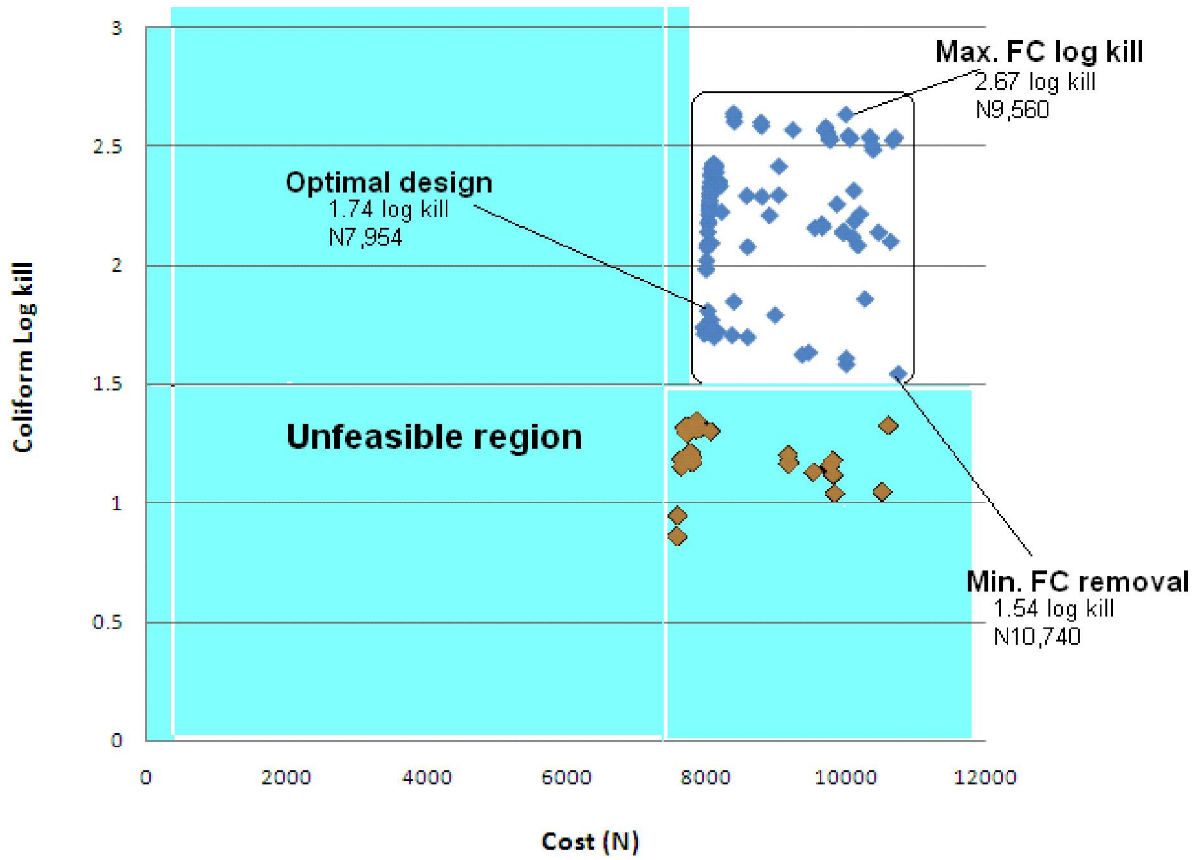

Figure 4.53 Trade off plot for Maturation Transverse design space.

In Figure 4.53 a maximum log kill attracts a cost of construction that is cheaper than the minimum pollutant log removal in the maturation reactor. Feasible region contains solution that satisfy the constraint of log kill greater or equal to 1.50 (98 designs of a total of 129) in the optimization process. The optimal design achieved a better log reduction than the minimum FC removal design with an appreciable cost to the tune of N2, 786. Inside the front, user can select a design according to his preference. In this study, the optimal, minimum and the maximum design solution has been selected and presented in Table 4.11.

Table 4.11 summarizes all the properties of the design configuration for the combination of even and odd transverse baffle arrangement using MOGA II algorithm in achieving the optimization of both the cost and FC minimization objectives. The algorithm solved and generated the optimal trade-offs among the two objectives. Although several other alternatives were generated, the final sets of optimal solutions were selected from the output file. The algorithm solved for a total number of evaluations that is equal to the initial population multiplied by the number of generations. The optimal design solutions were based on the fact that there is no solution that satisfies the two conditions at the same time but could reach a compromise.

Table 4.11 MOGA II designs for transverse baffle arrangement

	Anaerobic Transverse			**Facultative Transverse**			**Maturation Transverse**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**2.51E-1**	2.51E-1	2.22E-1	**3.11E-2**	3.15E-2	8.66E-3	1.84E-2	2.86E-2	2.10E-3
Cost (N)	**1, 234**	1, 234	1, 489	**4, 988**	6, 635	5, 752	7, 954	10, 740	9, 560
Log removal	**0.60**	0.60	0.65	**1.51**	1.50	2.06	1.74	1.54	2.67
Area ratio	**2:1**	2:1	2:1	**1:1**	1:1	1:1	2:1	2:1	2:1
Area (m^2)	**1.65E-1**	1.65E-1	1.65E-1	**1.36E0**	1.96E0	1.53E0	2.28E0	3.28E0	2.78E0
Depth (m)	**1.20E-1**	1.20E-1	1.20E-1	**4.80E-2**	3.36E-2	4.32E-2	3.60E-2	2.51E-2	2.95E-2
Length (m)	**5.74E-1**	5.74E-1	5.74E-1	**1.17E0**	1.40E0	1.24E0	2.13E0	2.56E0	2.36E0
Width (m)	**2.87E-1**	2.87E-1	2.87E-1	**1.17E0**	1.40E0	1.24E0	2.13E0	2.56E0	2.36E0
Velocity (m/s)	**2.32E-3**	2.32E-3	2.32E-3	**5.79E-3**	8.27E-3	6.44E-3	7.72E-3	1.11E-2	9.44E-3
Baffle length (m)	**1.66E-1**	1.66E-1	1.74E-1	**6.16E-1**	8.07E-1	9.96E-1	9.93E-1	1.19E0	1.10E0
Baffle ratio	**58%**	58%	61%	**53%**	58%	81%	93%	93%	93%
Number of baffles	**2**	2	6	**2**	2	4	4	4	6

The properties of the optimized designs for faecal coliform inactivation and flow pattern/contour within the reactors for even and odd, transverse baffle arrangements based on MOGA II algorithm are presented (Figure 4.54-4.58) while Figure 4.48, 4.49 and 4.52 present the history cost on design for the anaerobic and facultative reactors respectively.

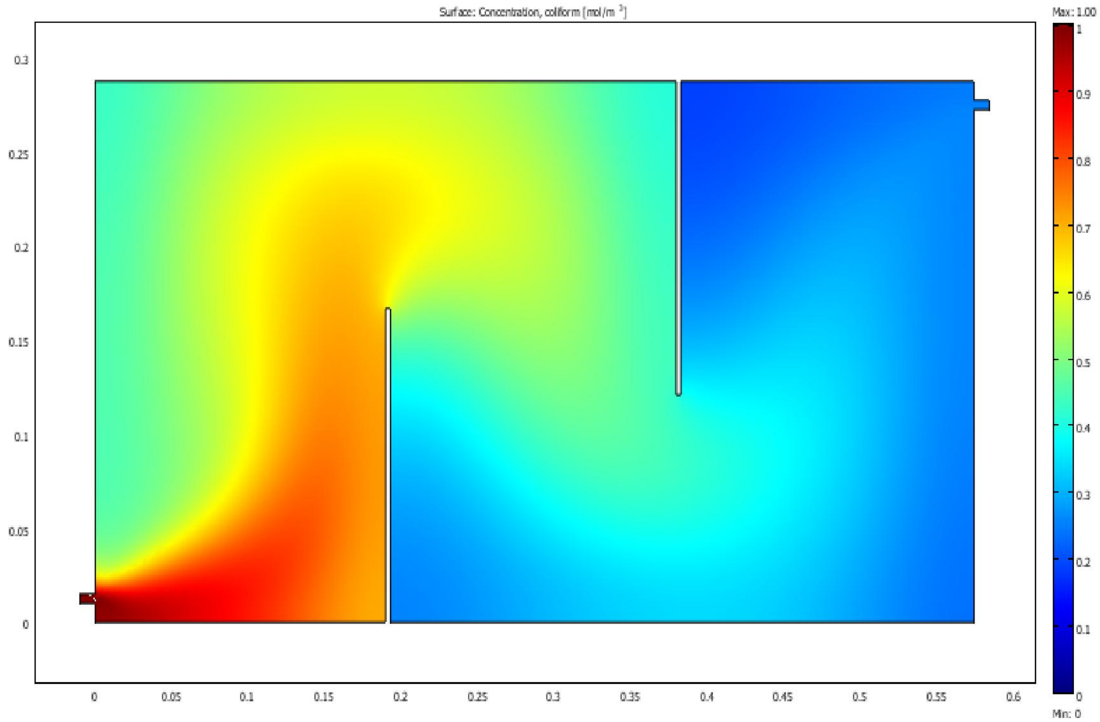

Figure 4.54 MOGA-II optimal faecal coliform removal design with least cost for even baffle transverse arrangement in anaerobic reactor

Figure 4.54 shows the MOGA II design flow pattern on a ratio 2:1 surface plane at a depth 0.12m below the anaerobic reactor surface. It consist of 2 baffles at baffle length ratio of 58% pond width with a flow channel width of 0.189 m in the baffle compartment and a baffle opening of 0.121m. With this configuration, a cummulatie log removal of 0.60 was achieved at the outlet. The velocity achieved at the inlet has a value of 2.32×10^{-3} m/s. It can be seen that there is a mixture of red and yellow colours at the inlet area and as the flow traverses towards the second baffle compartment, the colour gradually changes through yellow to light blue at the outlet. With the positioning of this baffles, the effect of short circuiting has been avoided knowing that a small amout of short-circuiting could result in a large reduction in the discharge quality. Banda (2007) pointed out that the width of baffle spacing and baffle opening is a key factor that may improve the treatment and hydraulic performance of baffled waste stabilization ponds.

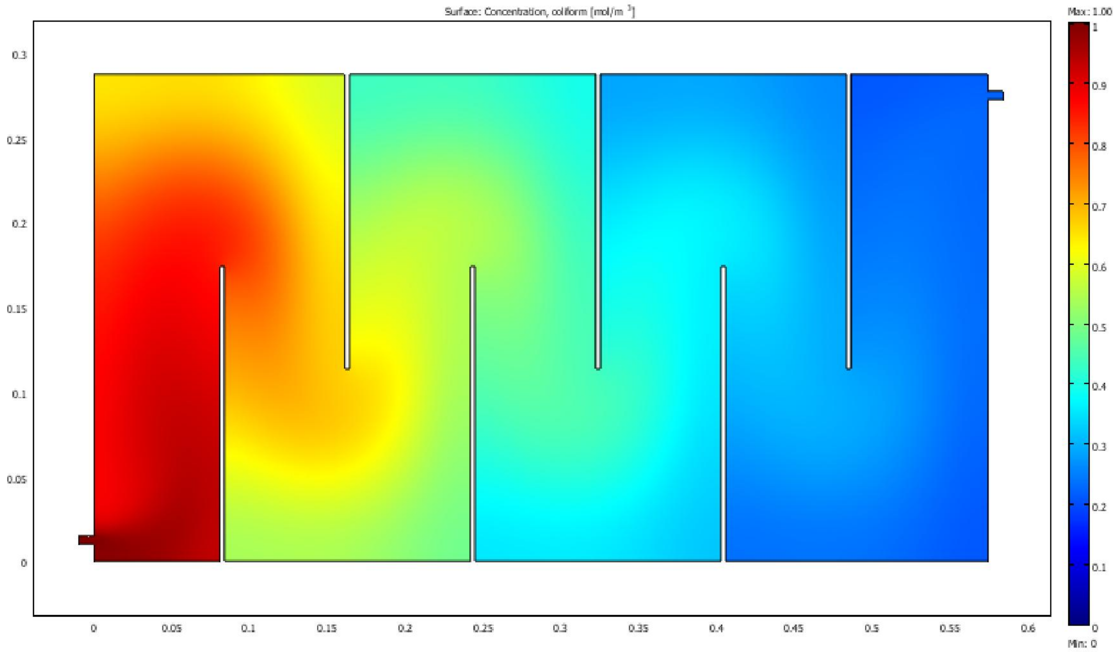

Figure 4.55 MOGA-II maximum faecal coliform removal design for even baffle transverse arrangement in anaerobic reactor

Figure 4.55 describes the maximum FC removal and design flow pattern in anaerobic reactor of ratio 2:1 surface plane at a depth 0.12m below the reactor surface. It consist of 6 baffles at baffle length ratio of 61% pond width with a flow channel width of 0.79 m in the baffle compartment and a baffle opening of 0.113m. With this configuration, a log removal of 0.65 was achieved at the outlet. The velocity at the inlet has a value of 2.32×10^{-3} m/s. It can be seen that the first baffle compartment is full of red colours at the inlet area and as the flow traverses towards the second baffle compartment, the colour gradually changes through yellow to light blue at the outlet. This describes the inactivation of feacal coliform in the reator. A clear distinction between the inlet and the outlet area. It can be seen that the baffle placed close to the inlet has forced the wastewater in the first compartment to circulate around before transiting to other compartments.

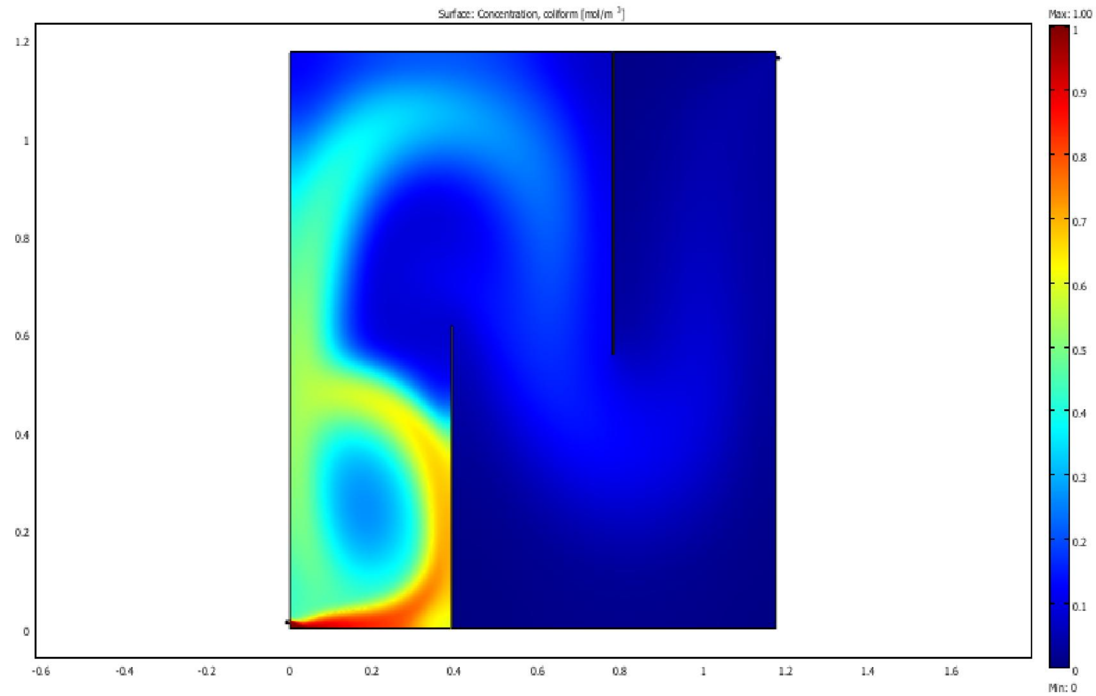

Figure 4.56 MOGA-II optimal faecal coliform removal design with least cost for even baffle transverse arrangement in facultative reactor

The MOGA II optimal design as shown in Figure 4.56 describes the design flow pattern in transverse facultative reactor of area ratio 1:1 surface plane at a depth 0.048m below the reactor surface. It consist of 2 baffles at baffle length ratio of 53% pond width with a flow channel width of 0.388m in the baffle compartment and a baffle opening of 0.554m. With this configuration, a log removal of 1.51 was achieved at the outlet. The velocity at the inlet has a value of 5.79×10^{-3} m/s. The circulation flow pattern in the first baffle compartment allows mixing to occur. It can be seen that the first baffle compartment is a blend of red, yellow and light blue colours and as the flow traverses towards the second baffle compartment, the colour changed to blue at the outlet. A clear distinction between the inlet and the outlet area.

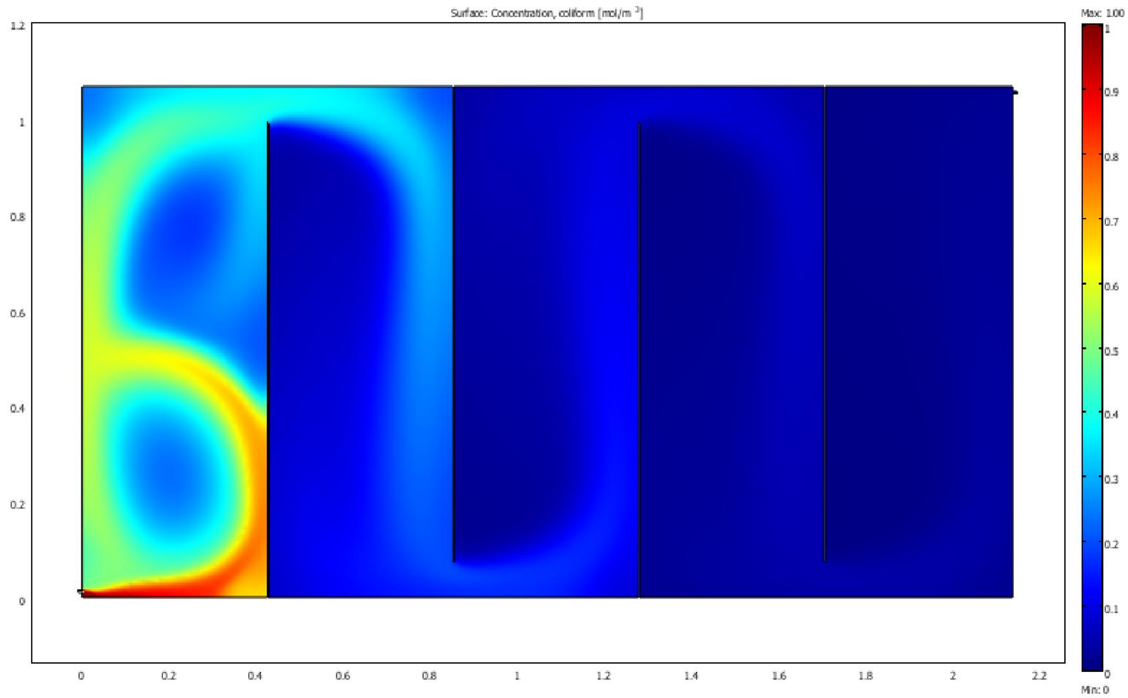

Figure 4.57 MOGA-II optimal faecal coliform removal design with least cost for even baffle transverse arrangement in maturation reactor

Figure 4.57 describes the optimal FC removal and design flow pattern in maturatin reactor of area ratio 2:1 surface plane at a depth 0.036m below the reactor surface. It consist of 4 baffles at baffle length ratio of 93% pond width with a flow channel width of 0.424 m in the baffle compartment and a small baffle opening size of 0.077m. With this configuration, a log removal of 1.74 was achieved at the outlet. The velocity at the inlet has a value of $7.72{\times}10^{-3}$ m/s. It can be seen that the first baffle compartment is also a mixture of red, yellow and blue colours at the inlet area indicating the mixing of wastewater as it enters the first baffle compartments. As the flow traverses towards the second baffle compartment, the colour gradually changes through light blue to deep blue at the outlet. A clear distinction between the inlet and the outlet area. The placement of towards the inlet has forced the wastewater to circulate and mix in the first compartment.

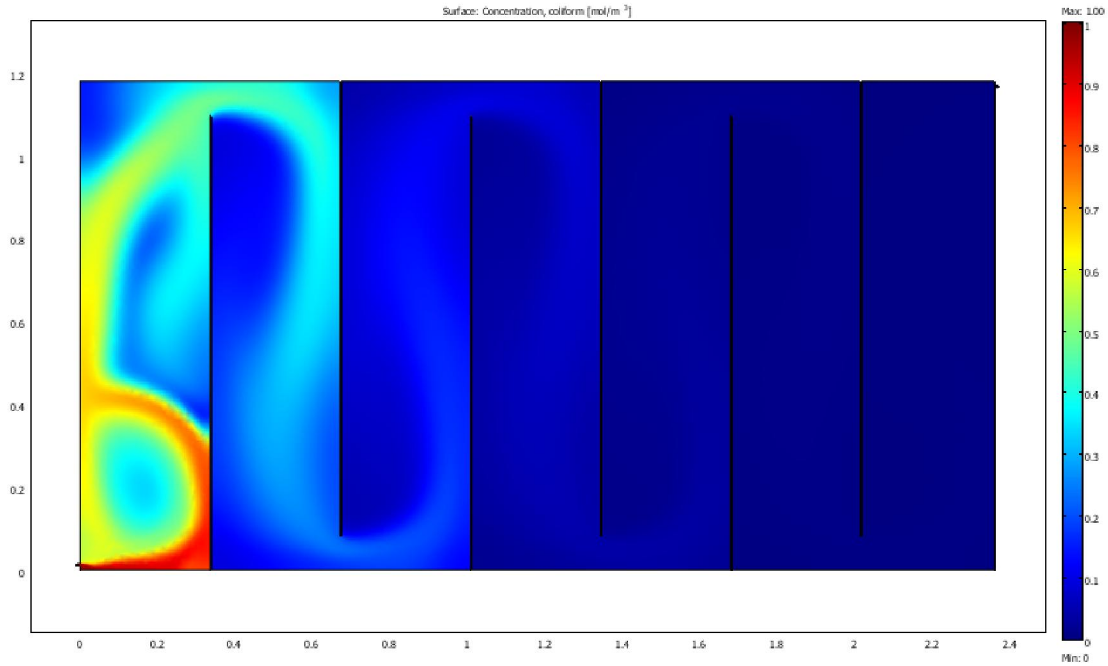

Figure 4.58 MOGA-II maximum faecal coliform removal design for even baffle transverse arrangement in maturation reactor

Figure 4.58 describes the maximum FC removal and design flow pattern in maturatin reactor of area ratio 2:1 surface plane at a depth 0.0295m below the reactor surface. It consist of 6 baffles at baffle length ratio of 93% pond width with a flow channel width of 0.335m in the baffle compartment and a small baffle opening size of 0.08m. With this configuration, a log removal of 2.67 was achieved at the outlet. This give the overall maximum FC removal in all the reactors considered. The velocity at the inlet has a value of 9.44×10^{-3} m/s. It can be seen that the first baffle compartment is also a mixture of red, yellow and blue colours at the inlet area indicating the mixing of wastewater as it enters the first baffle compartments. As the flow traverses towards the second baffle compartment, the colour gradually changes through light blue to deep blue at the outlet indicating the extent of microbial inactivation in the reator.

Figures 4.59-4.61 present a trade off plot/pareto front showing the range of reasonable possibilities in the design space for the longitudinal baffle arrangement in the three reactors. In the fronts, feasible region contains solutions that satisfy the constraints for both log reduction and the cost objectives as previously described for the transverse baffle arrangement. One may not be able to plot a curve because there is no solution that improves at the same time both of the objectives. In this front, the selection of any design solution is based on the designers' preference. However, the optimal, minimum and the maximum design solution has been selected and presented in Table 4.12. The optimal design is the best compromise between the 2 objectives.

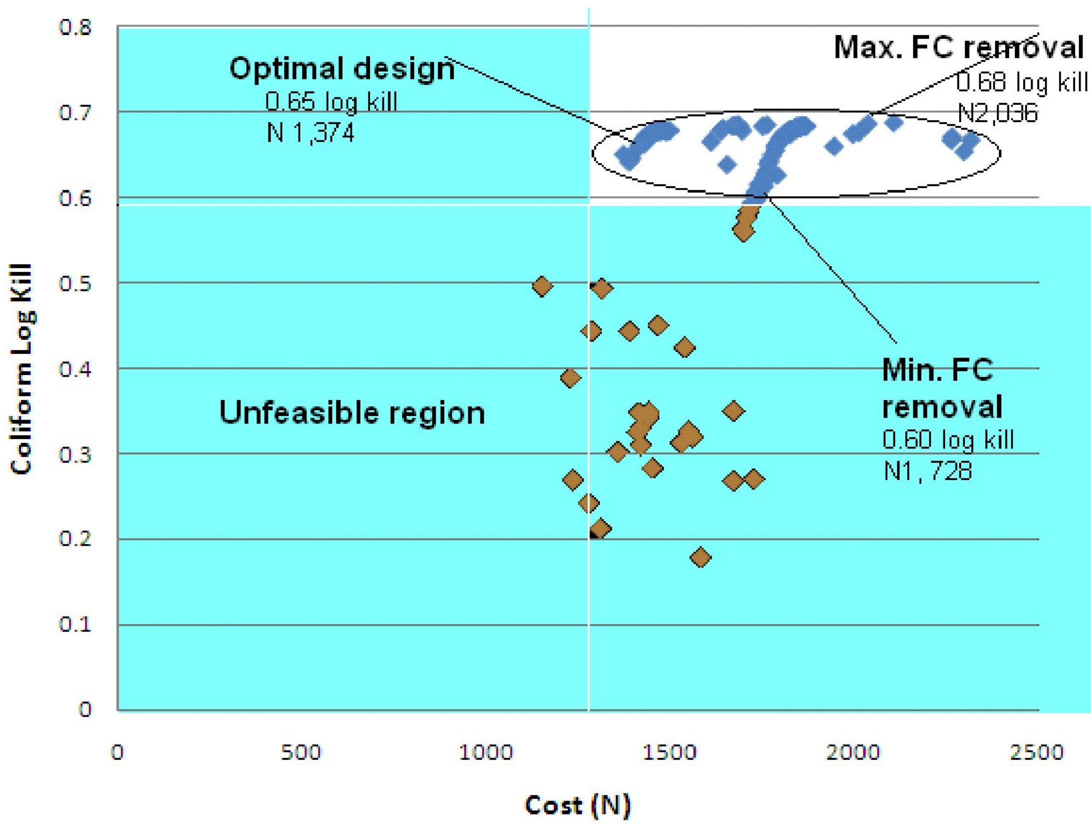

Figure 4.59 Trade off plot for anaerobic longitudinal design space.

Figure 4.59 presents feasible region that contains solution which satisfy the constraint of log kill greater than or equal to 0.65 (99 designs of a total of 132) in the optimization process. The optimal design gives a better performance producing a feacal log kill of 0.65 with an associated cost of N1, 374 as against a log kill of 0.60 and N1, 234 in the transverse arrangement while the maximum FC log kill attracts an extra cost of N662 to achieve a log kill of 0.68. It is interesting to note that with higher cost as shown in the front, a reduced feacal coliform was achieved with a cost of N1, 728. The unfeasible region represents solutions of feacal log kill that are below the minimum constraint of 0.60 that was set in the optimization algorithm to achieve a target feacal reduction.

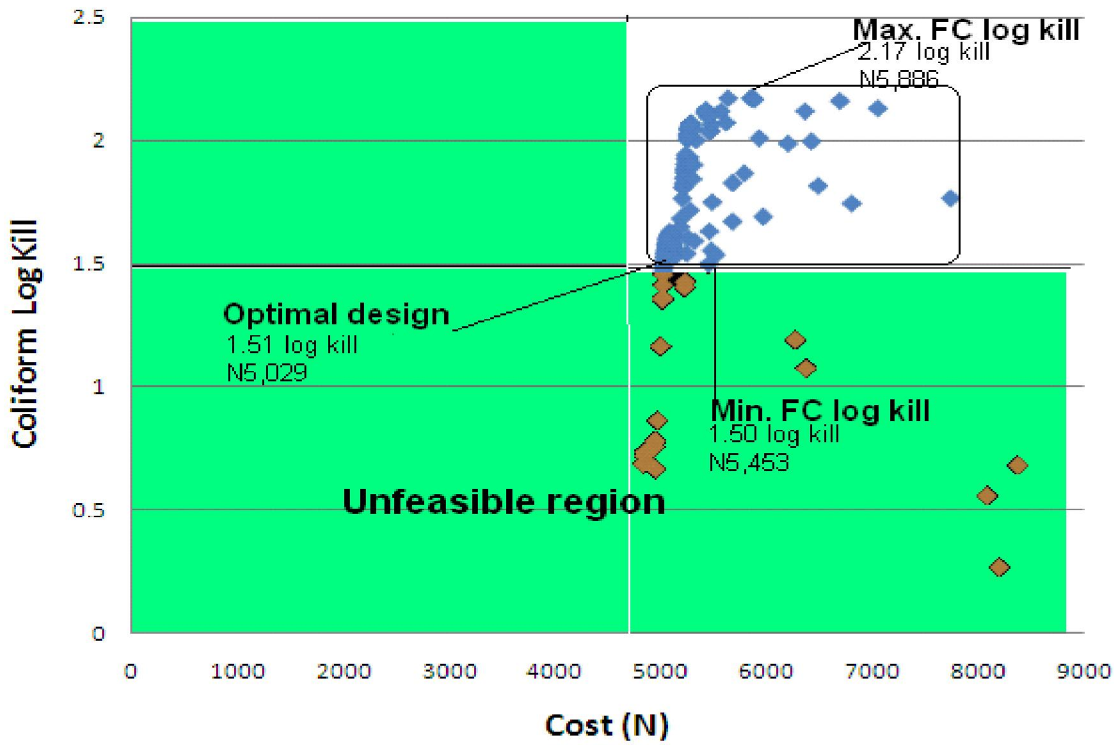

Figure 4.60 Trade off plot for facultative longitudinal design space.

Figure 4.60 presents feasible region containing the optimal, minimum and maximum solutions that satisfy the constraint of log kill greater than or equal to 1.50 (112 of a total of 139 feasible designs generated) in the optimization process. The optimal design produced the same feacal log kill of 1.51 as in the transverse arrangement but with an higher cost of N5, 029 which makes the transverse be a better choice for anaerobic reactor design. The increase in cost to N5, 886 achieves a better performance of feacal reduction for the maximum log kill while the minimum of 1.50 log kill was achieved with a cost of N5, 453. The unfeasible region represents solutions of feacal log kill that are below the minimum constraint of 1.50 that was set in the optimization algorithm to achieve a target feacal reduction and there was no cost lesser than N5, 000 to achieve any feacal reduction.

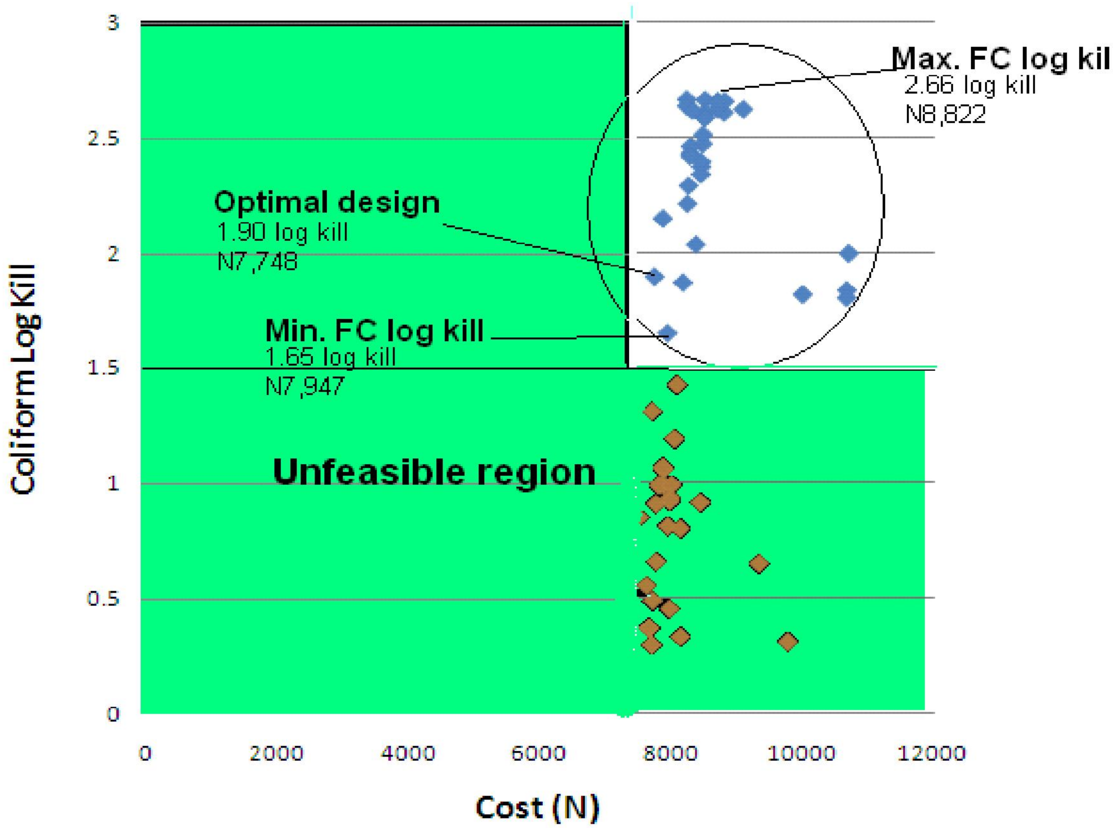

Figure 4.61 Trade off plot for maturation longitudinal design space.

In Figure 4.61 a maximum log kill attracts a cost of construction that is higher than the optimal pollutant log removal in the maturation reactor (N8, 822 vs N7, 748). Feasible region contains solution that satisfy the constraint of log kill greater or equal to 1.50 (34 designs of a total of 60) in the optimization process. The optimal design achieved a better log reduction than the minimum FC removal design with a reduced cost to the tune of N199. Inside the front, user can select a design according to preference. In this study, the optimal, minimum and the maximum design solution has been selected and presented in Table 4.12.

Table 4.12 summarizes all the properties of the design configuration for the combination of even and odd longitudinal baffle arrangement using MOGA II algorithm in achieving the optimization of both the cost and FC minimization objectives. The algorithm solved and generated the optimal trade-offs among the two objectives. Although several other alternatives were generated, the final sets of optimal solutions were selected from the output file. Table 4.12 shows that maturation longitudinal design has a better comparative cost to the Transverse baffle configuration in Table 4.11. Except for the anaerobic optimal design that has 68% baffle length ratio, all the remaining designs presented have all the baffle length greater than 70% which shows that the 70% pond width reported in literature may not consistently be the best configuration.

Table 4.12 MOGA II designs for longitudinal baffle arrangement

	Anaerobic Longitudinal			**Facultative Longitudinal**			**Maturation Longitudinal**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	2.23E-1	2.50E-1	2.06E-1	3.07E-2	3.15E-2	6.80E-3	**1.27E-2**	2.23E-2	2.19E-3
Cost (N)	1, 374	1, 728	2, 036	5, 029	5, 453	5, 886	**7, 748**	7, 947	8, 822
Log removal	0.65	0.60	0.68	1.51	1.50	2.17	**1.90**	1.65	2.66
Area ratio	1:1	3:1	3:1	1:1	1:1	2:1	**1:1**	1:1	4:1
Area (m^2)	1.65E-1	1.96E-1	3.74E-1	1.38E0	1.53E0	1.38E0	**2.28E0**	2.34E0	2.28E0
Depth (m)	1.20E-1	1.01E-1	5.28E-2	4.80E-2	4.32E-2	4.80E-2	**3.60E-2**	3.49E-2	3.60E-2
Length (m)	4.06E-1	7.67E-1	1.06E-1	1.17E0	1.24E0	1.66E0	**1.51E0**	1.53E0	3.02E0
Width (m)	4.06E-1	2.56E-1	3.53E-1	1.17E0	1.24E0	8.30E-1	**1.51E0**	1.53E0	7.55E-1
Velocity (m/s)	2.32E-3	2.76E-3	5.27E-3	5.79E-3	6.44E-3	5.79E-3	**7.72E-3**	7.96E-3	7.72E-3
Baffle length (m)	2.74E-1	5.75E-1	9.80E-1	7.57E-1	8.41E-1	1.44E0	**1.22E0**	1.23E0	2.72E0
Baffle ratio	68%	75%	93%	65%	68%	87%	**81%**	81%	90%
Number of baffles	3	3	3	2	2	5	**2**	2	4

The properties of the optimized designs for faecal coliform inactivation and flow pattern/contour within the reactors for even and odd, longitudinal baffle arrangements based on MOGA II algorithm are presented (Figure 4.62-4.65) for the anaerobic, facultative and maturation reactors respectively.

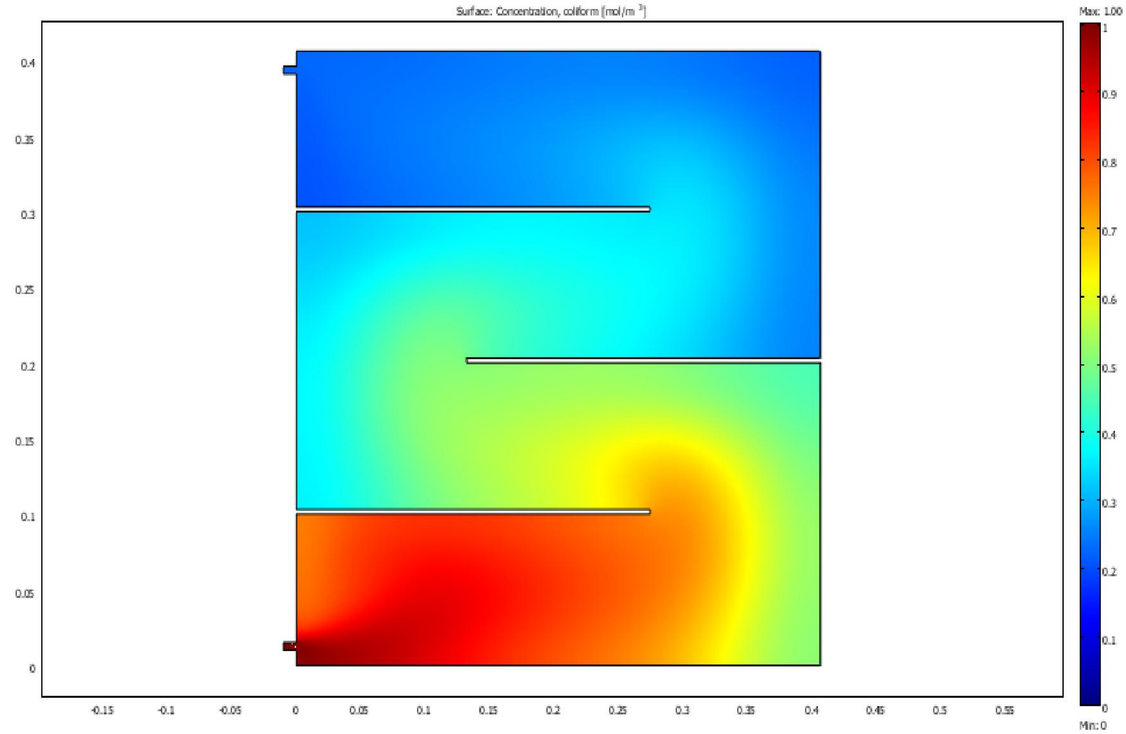

Figure 4.62 MOGA-II optimal faecal coliform removal design with least cost for even baffle longitudinal arrangement in anaerobic reactor

The three-baffle anaerobic reactor in Figure 4.62 shows a surface area ratio 1:1 of the MOGA II design flow pattern on a surface plane at a depth 0.120 m below the water surface for the optimal FC removal. The reactor baffle lengths are 0.274 m (68% pond width) spread evenly at a flow channel width of 0.099 m in the baffle compartment and a baffle opening of 0.132 m. With this configuration, a cummulatie log removal of 0.65 was achieved at the outlet. The velocity at the inlet has a value of 2.32×10^{-3} m/s. With the positioning of the 3-baffles, the flow flow pattern shows that there is improved mixing of wastewater in each comartment and there is no short-circuiting visible at this level as there are no contours directly linking the inlet to the outlet.

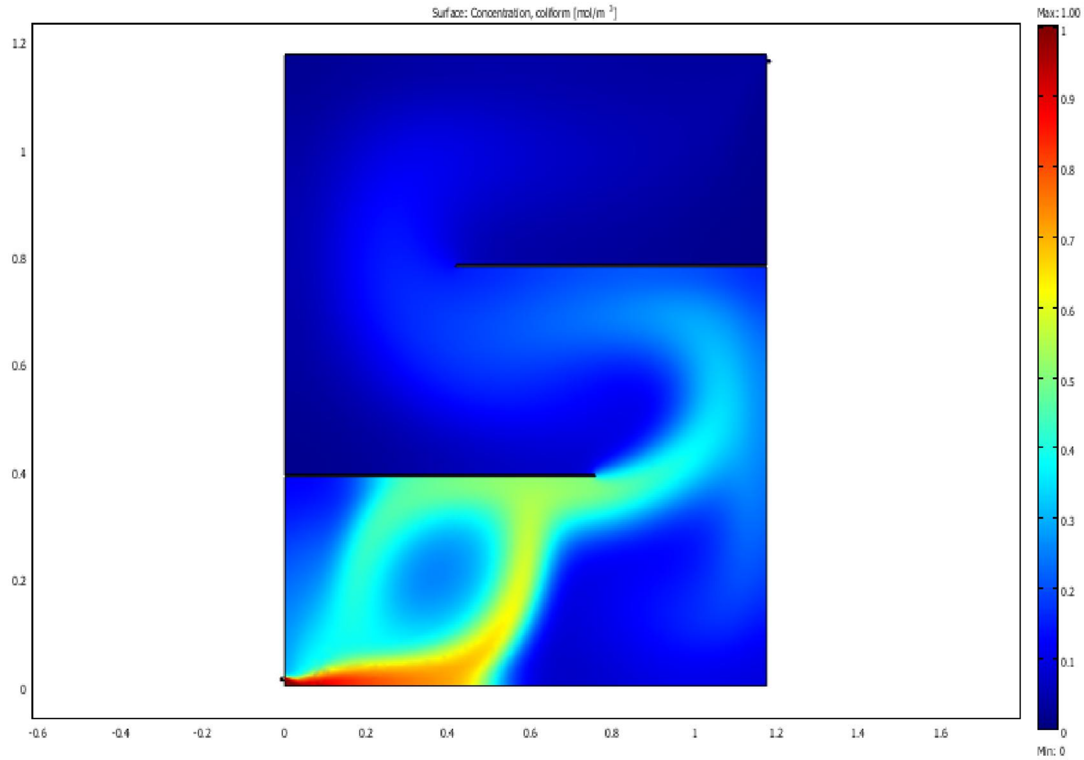

Figure 4.63 MOGA-II optimal faecal coliform removal design with least cost for even baffle longitudinal arrangement in facultative reactor

Figure 4.63 describes the MOGA II design flow pattern and faecal coliform distribution on a ratio 1:1 surface plane at a depth 0.048 m below the facultative reactor surface. It consist of 2 baffles with baffle lengths of 0.757 m (65% pond width) with a flow channel width of 0.388 m in the baffle compartment and a baffle opening of 0.413 m. With this configuration, a cummulative log removal of 1.51 was achieved at the outlet. The velocity achieved at the inlet has a value of 5.79×10^{-3} m/s. There is significant visible difference of flow pattern in the first and third baffle compartments. The circulating flow pattern close to inlet in the first baffle opening compartment allows mixing to occur. Table 4.12 describes other properties associated with this design configuration.

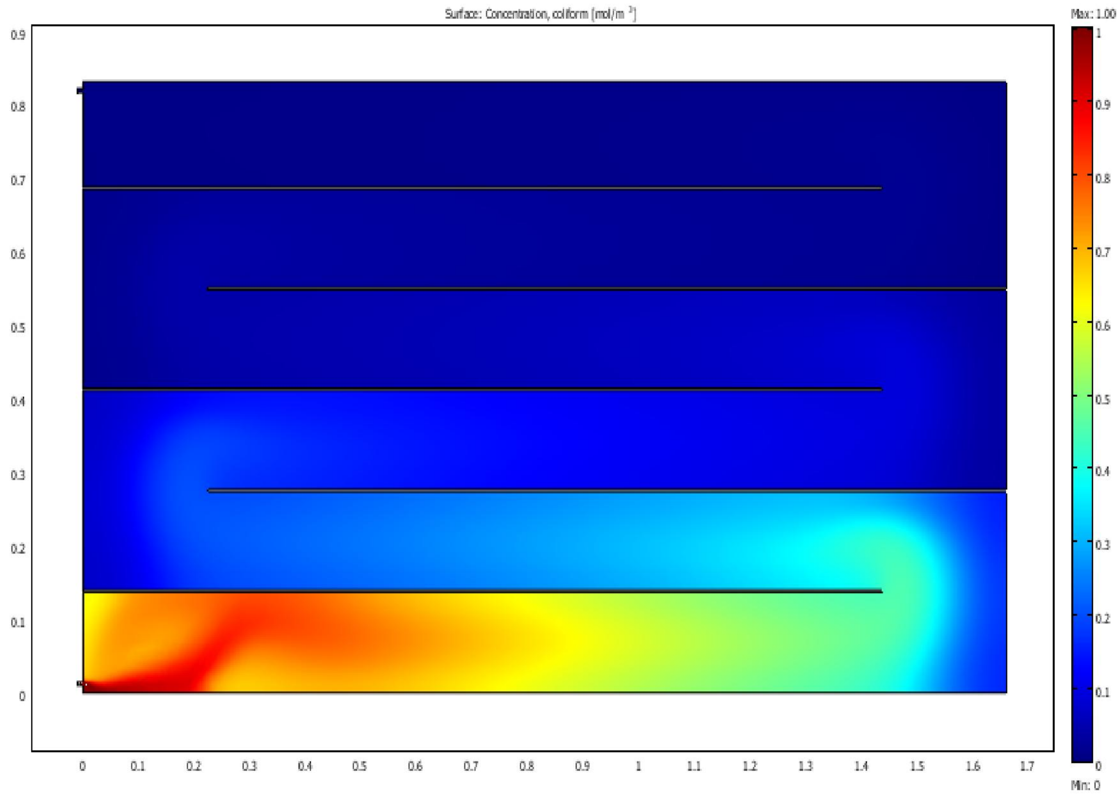

Figure 4.64 MOGA-II maximum faecal coliform removal design for even baffle longitudinal arrangement in facultative reactor

Figure 4.64 shows the maximum FC removal design flow pattern on an area ratio 2:1 surface plane at a depth of 0.048 m below the facultative reactor surface. It consist of 5 baffles with baffle lengths of 1.44 m (87% pond width) with a flow channel width of 0.274 m in the baffle compartment and a baffle opening of 0.399 m. With this configuration, a cummulative log removal of 2.17 was achieved at the outlet. The red color runs across the length of the baffle in the first compartment before changing to yellow towards end of the reactor length. The velocity achieved at the inlet has a value of 5.79×10^{-3} m/s and there is significant visible difference of flow pattern in the first and last baffle compartments which shows that there is improved treatment of wastewater along each baffle compartment. This type of arrangement is expensive as compared to optimal two baffle facultative reactor when comparing their log removal of 1.51 versus 2.17.

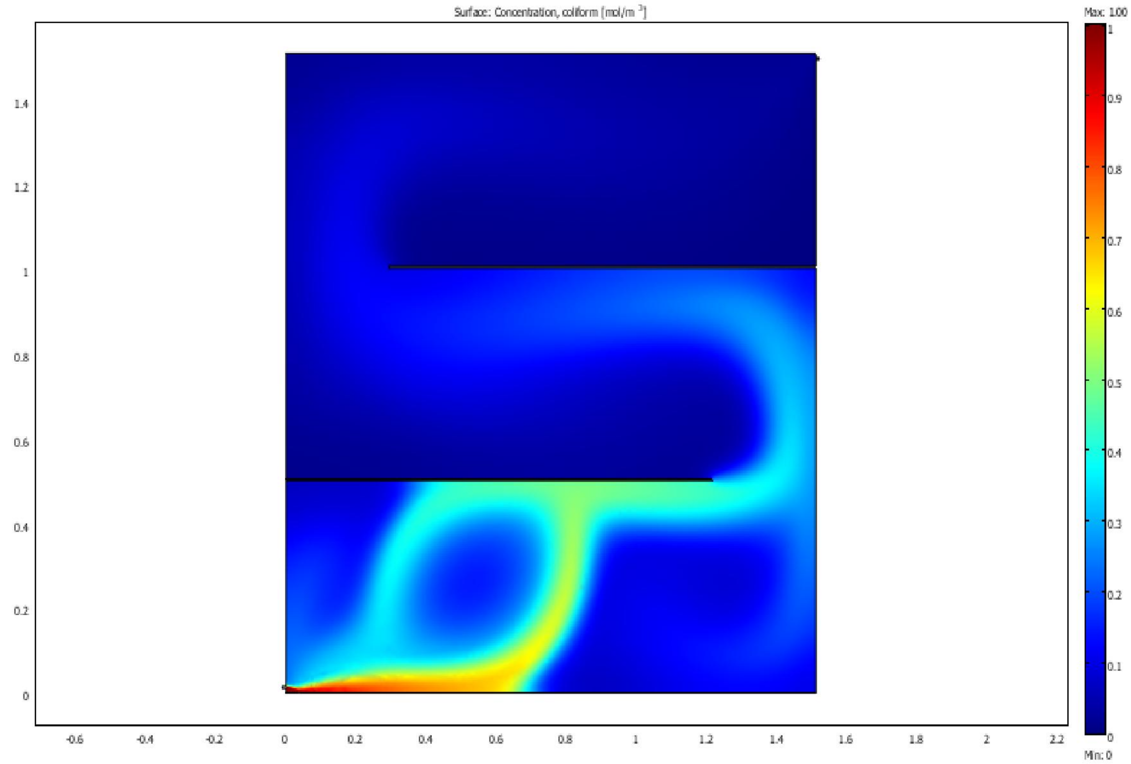

Figure 4.65 MOGA-II optimal faecal coliform removal design with least cost for even baffle longitudinal arrangement in maturation reactor

The two-baffle maturation reactor in Figure 4.65 shows a surface area ratio 1:1 of the MOGA II design flow pattern on a surface plane at a depth 0.036 m below the water surface for the optimal FC removal. The reactor baffle lengths are 1.22 m (81% pond width) spread evenly at a flow channel width of 0.501 m in the baffle compartment and a baffle opening of 0.29 m. With this configuration, a cummulative log removal of 1.90 was achieved at the outlet. The velocity at the inlet has a value of 7.72×10^{-3} m/s. It can be seen that there are significant visible difference of flow pattern as the wastewater travels from one baffle compartment to the other and there is no short-circuiting visible at this level.

In the overall consideration in both the transverse and longitudinal configurations from Tables 4.2, 4.3, 4.11 and 4.12 with cost objective in mind, transverse design configurations were chosen for the anaerobic, facultative and maturation reactors. This is due to the fact that they achieve the most optimal cost objective and one could conclude that the designs presented in Table 4.13 are good representation of the overall optimal design results for the three reactors (anaerobic-, facultative- and maturation transverse arrangement).

Table 4.13 MOGA-II Optimal design results

	Anaerobic Transverse			**Facultative Transverse**			**Maturation Transverse**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 234**	1, 234	1, 489	**4, 988**	6, 635	5, 752	**7, 221**	8, 095	8, 051
Log removal	**0.60**	0.60	0.65	**1.51**	1.50	2.06	**1.84**	1.68	2.51
Reactor L/W ratio (r)	**2:1**	2:1	2:1	**1:1**	1:1	1:1	**3:1**	4:1	4:1
Depth (m)	**1.20E-1**	1.20E-1	1.20E-1	**4.80E-2**	3.36E-2	4.32E-2	**4.00E-2**	3.49E-2	3.60E-2
Baffle ratio	**58%**	58%	61%	**53%**	58%	81%	**70%**	61%	71%
Number of baffles	**2**	2	6	**2**	2	4	**4**	5	7

In Table 4.13, the MOGA-II optimization predicted that a transverse baffle arrangement for anaerobic, facultative, and maturation ponds was the global optimum configuration for the WSP system. The most significant difference between the MOGA-II and SIMPLEX results occurred with the anaerobic pond design. MOGA-II predicted a least cost anaerobic design that utilized one less baffle, a lower reactor area ratio, longer baffle lengths, and a slightly deeper water depth compared to the SIMPLEX results. In terms of the facultative pond, the MOGA-II result was similar to the SIMPLEX predicted result with only changes in the log reduction (1.51 for MOGA-II and 1.62 for SIMPLEX) and baffle length (53% for MOGA-II and 83% for SIMPLEX). The longer SIMPLEX baffle length prediction is likely due to SIMPLEX using a longitudinal baffle arrangement. MOGA-II also predicted the same pond configuration for the facultative pond.

The MOGA-II results in Table 4.13 further confirms that the strictly 70% baffle length configuration may not consistently be the best pond configuration for WSP as previously reported. Although the MOGA-II optimization algorithm included an additional objective function (i.e., maximizing the pond log reduction), the final overall log reduction for the WSP system was not significantly different compared to the SIMPLEX result (4.0 vs 4.1) and only slightly higher than the results in Tables 4.2 and 4.3 (4.0. vs 3.94 and 3.82).

The scaling-up of design laboratory-scale model to a field scale prototype is provided in section 4.5.3 following the Froude number principle that was used in chapters 3. It is again important to state that the selection of any design could be based on the engineers' discretion if at all the optimal design which gives the minimum cost is not to be chosen for a specific reason. However, it should be born in mind that when other cost (labor, construction, maintenance and other expenses) are included, the designer may be forced to consider the optimal design specification. It was observed in general that the costs of optimized designs are lower in MOGA II algorithm as compared to the Simplex. This could be attributed to the fact that it is a global multi-objective search tool.

4.5.3 Scaling up of Optimized design configuration

Three designs were selected for scaling up to full scale prototype (Anaerobic-, Facultative- and Maturation Transverse arrangement). These were selected as the optimal designs from MOGA II design algorithm.

4.5.3.1 Scaling up of Anaerobic Transverse baffle arrangement (MOGA II Design).

This has the configuration as:

Length = 0.574 m

Width = 0.287 m

Depth of water = 0.12 m,

Area = $0.165 m^2$

Velocity = 2.32×10^{-3} m/s

Area ratio 2:1

Number of baffles = 2

Pond-width = 58%

Log removal rate = 0.60.

The flow rate into a single reactor, Q = AV

= depth × width of flow × velocity

$= 0.12 \times 0.005 \times 2.32 \times 10^{-3} = 1.392 \times 10^{-6}\ m^3/s$

$= 0.12\ m^3/d$.

Flow rate into the model reactor = $0.12\ m^3/day$

Flow rate in the prototype, $Q_p = 1196.8\ m^3/day$

Length of model = 0.574 m

Width of model = 0.287 m

Depth of wastewater in the model = 0.12 m

Surface area of model = $0.165\ m^2$

Applying the scale ratio of 45.5 for the Horizontal dimension as expressed in chapter 3;

$$\frac{L_p}{L_m} = 45.5$$

$L_p = 0.574 \times 45.5 = 26.12m$ as the length of prototype

$$\frac{B_p}{B_m} = 45.5$$

$B_p = 0.287 \times 45.5 = 13.06m$ as the width of prototype

Applying the scale ratio of 38.5 for the vertical dimension;

$$\frac{D_p}{D_m} = 38.5$$

$D_p = 0.12 \times 38.5 = 4.62m$ as the water depth in prototype

Depth of prototype = 4.62 m which is within the range specified in literature (2.5 - 5 m)

Area ratio of 2:1 gives the length and width = 26.12 m × 13.06 m

Volume of each pond = $1576\ m^3$

The anaerobic pond has 2 baffles

The length of baffle = 58% of reactor width (7.57 m) and with a log removal of 0.60.

4.5.3.2 Scaling up of Facultative Transverse baffle arrangement (MOGA II Design).

This has the configuration as: Length = 1.17 m, width = 1.17 m, depth of water = 0.048 m, area = 1.36m^2, velocity = 5.79×10^{-3} m/s, area ratio 1:1, 2 number of baffles at a length of 53% of the pond width with a log removal of 1.51.

The flow rate into a single reactor, Q = AV

= depth × width of flow × velocity

= $0.048 \times 0.005 \times 5.79 \times 10^{-3} = 1.390 \times 10^{-6}$ m^3/s

= 0.12 m^3/d.

Flow rate into the model reactor = 0.12 m^3/day

Flow rate in the prototype, Q_p = 1196.8 m^3/day

Length of model = 1.17 m

Width of model = 1.17 m

Depth of wastewater in the model = 0.048 m

Total area of model = 1.37 m^2

Applying the scale ratio of 55.8 for the Horizontal dimension as expressed in chapter 3;

$$\frac{L_p}{L_m} = 55.8$$

$L_p = 1.17 \times 55.8 = 65.29$m as the length of prototype

$$\frac{B_p}{B_m} = 55.8$$

$B_p = 1.17 \times 55.8 = 65.29$m as the width of prototype

Applying the scale ratio of 38.9 for the vertical dimension;

$$\frac{D_p}{D_m} = 38.9$$

$D_p = 0.048 \times 38.9 = 1.87$m as the water depth in prototype

Depth of prototype = 1.87 m which is within the range specified in literature (1.0 – 2.0 m)

Area ratio of 1:1 gives the length and width = 65.29 m × 65.29m = 4262.78 m^2

Volume of each pond = 65.29 × 65.29 × 1.87 = 7971.41m^3

The facultative pond has 2 baffles

The length of baffle = 53% of reactor width (34.60 m) and with a log removal of 1.51

4.5.3.3 Scaling up of Maturation Transverse baffle arrangement (MOGA II Design).

This has the configuration as: Length = 2.10 m, width = 0.83 m, depth of water = 0.04 m, area = 1.74 m^2, velocity = 6.94×10^{-3} m/s, area ratio 3:1, 4 number of baffles at a length of 70% of the pond width with a log removal of 1.84.

The flow rate into a single reactor, Q = AV

= depth × width of flow × velocity

= $0.04 \times 0.005 \times 6.94 \times 10^{-3} = 1.390 \times 10^{-6}$ m^3/s

= 0.12 m^3/d.

Flow rate into the model reactor = 0.12 m^3/day

Flow rate in the prototype, Q_p = 1196.8 m^3/day

Length of model = 2.10 m

Width of model = 0.83 m

Depth of wastewater in the model = 0.04 m

Total area of model = 1.74 m^2

Applying the scale ratio of 56 for the Horizontal dimension as expressed in chapter 3;

$$\frac{L_p}{L_m} = 56$$

$L_p = 2.47 \times 56 = 138.37$ m as the length of prototype

$$\frac{B_p}{B_m} = 56$$

$B_p = 0.83 \times 56 = 46.13$ m as the width of prototype

Applying the scale ratio of 37.5 for the vertical dimension;

$$\frac{D_p}{D_m} = 37.5$$

$D_p = 0.04 \times 37.5 = 1.50$m as the water depth in prototype

Depth of prototype = 1.50 m which is in the range specified in literature (1.0 – 1.5 m)

Area ratio of 3:1 gives the length and width = 138.37m × 46.13 m = 6383.01 m^2

Volume of each pond = 138.37 × 46.13 × 1.50 = 9574.51m^3

The maturation pond has 4 baffles

The length of baffle = 70% of reactor width (32.54 m) and with a log removal of 1.84

4.5.3.4 Summary of results of Scaling up of Design Configuration

Table 4.14 summarizes the results of the scaling up for the ponds**.** The ponds in series maintain a constant flow rate with varying size and depth. The area ratios are 2:1, 1:1 and 3:1 anaerobic, facultative and maturation ponds respectively.

Table 4.14 Summary of results of Scaling up of Design Configuration

Design Parameters	AP	FP	MP
Flow rate (m^3/day)	1196.8	1196.8	1196.8
Length of Pond (m)	26.12	65.29	138.37
Width of Pond (m)	13.06	65.29	46.13
Depth of Pond (m)	4.62	1.87	1.50
Area of Pond (m^2)	341.13	4262.78	6383.11
Volume of Ponds (m^3)	1576.00	7971.41	9574.55
Number of baffle	2	2	4
Area ratio	2:1	1:1	3:1
Length of baffle (m)	58%	53%	70%

4.5.4 Results of sensitivity analysis for Simplex design at upper and lower boundary.

Sensitivity analysis have been used in determining how sensitive a model structure changes as a result of a change in two model parameters. The results obtained from the optimization algorithms show that changing the first order decay constant, k and temperature, T has effect on the effluent faecal coliform and the entire ponds configuration. There are differences in values of the pond size, baffle length, baffle numbers and other parameters.

Table 4.15 shows the summary of the Simplex sensitivity on Transverse baffle arrangement. The same order of optimal solutions was observed as when the base value of 9.124 was used for k. The overall optimal solutions are: Anaerobic and maturation designs from the transverse arrangement. The optimized design flow patterns for the three reactors in Table 4.15 are presented in Figures 4.59 - 4.61.

Table 4.15 Sensitivity Analysis Results for Transverse baffle arrangement (k=13.686)

	Anaerobic Transverse SA1			**Facultative Transverse SA1**			**Maturation Transverse SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**1.52E-1**	1.58E-1	1.04E-1	2.02E-3	9.40E-2	1.27E-3	**4.54E-3**	9.83E-3	1.69E-3
Cost (N)	**1, 200**	1, 211	1, 767	5, 343	5, 672	5, 649	**7, 891**	7, 993	7, 992
Log removal	**0.82**	0.80	0.98	2.69	2.03	2.89	**2.34**	2.01	2.77
Area ratio	**3:1**	3:1	2:1	2:1	2:1	3:1	**2:1**	4:1	2:1
Area (m^2)	**1.75E-1**	1.75E-1	3.17E-1	1.38E0	1.53E0	1.36E0	**2.28E0**	2.35E0	2.28E0
Depth (m)	**1.13E-1**	1.13E-1	6.24E-2	4.80E-2	4.32E-2	4.80E-2	**3.60E-2**	3.49E-2	3.60E-2
Length (m)	**7.25E-1**	7.25E-1	7.96E-1	1.66E0	1.75E0	2.03E0	**2.13E0**	3.07E0	2.13E0
Width (m)	**2.42E-1**	2.42E-1	3.98E-1	8.30E-1	8.75E-1	6.68E-1	**1.07E0**	7.66E-1	1.07E0
Velocity (m/s)	**2.46E-3**	2.46E-3	4.46E-3	5.79E-3	6.44E-3	5.79E-3	**7.72E-3**	7.96E-3	7.72E-3
Baffle length (m)	**1.93E-2**	2.30E-2	2.83E-1	5.69E-1	5.12E-1	6.37E-1	**6.78E-1**	3.41E-1	7.20E-1
Baffle ratio	**8%**	10%	71%	69%	59%	94%	**64%**	45%	68%
Number of baffles	**3**	4	7	6	6	8	**5**	4	6

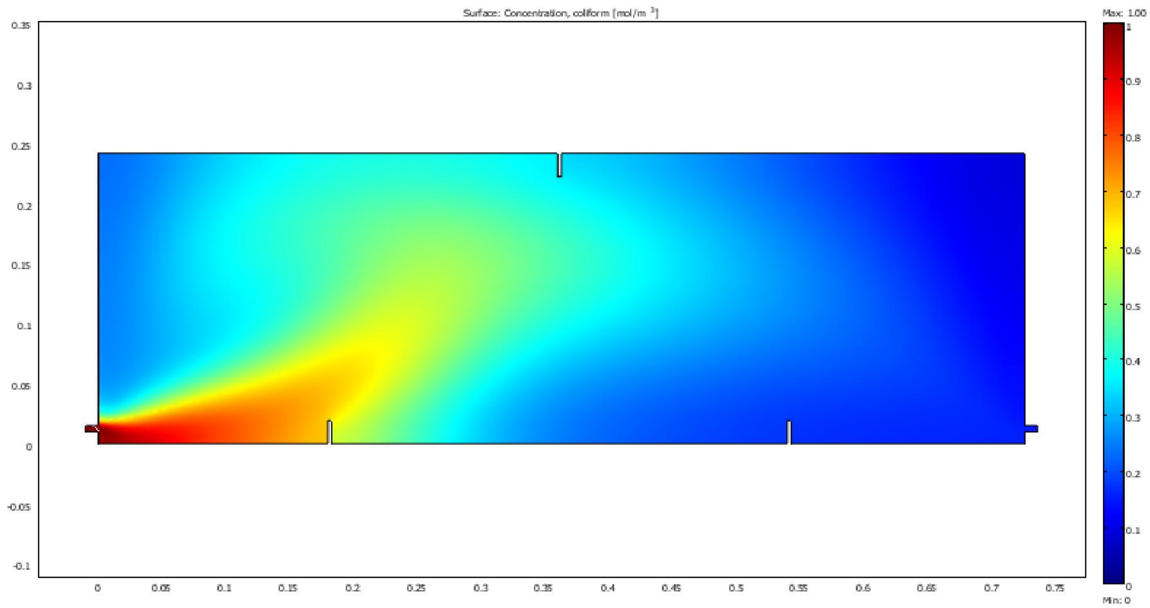

Figure 4.66 SIMPLEX optimal faecal coliform removal design with least cost for transverse baffle arrangement in anaerobic reactor (k = 13.686)

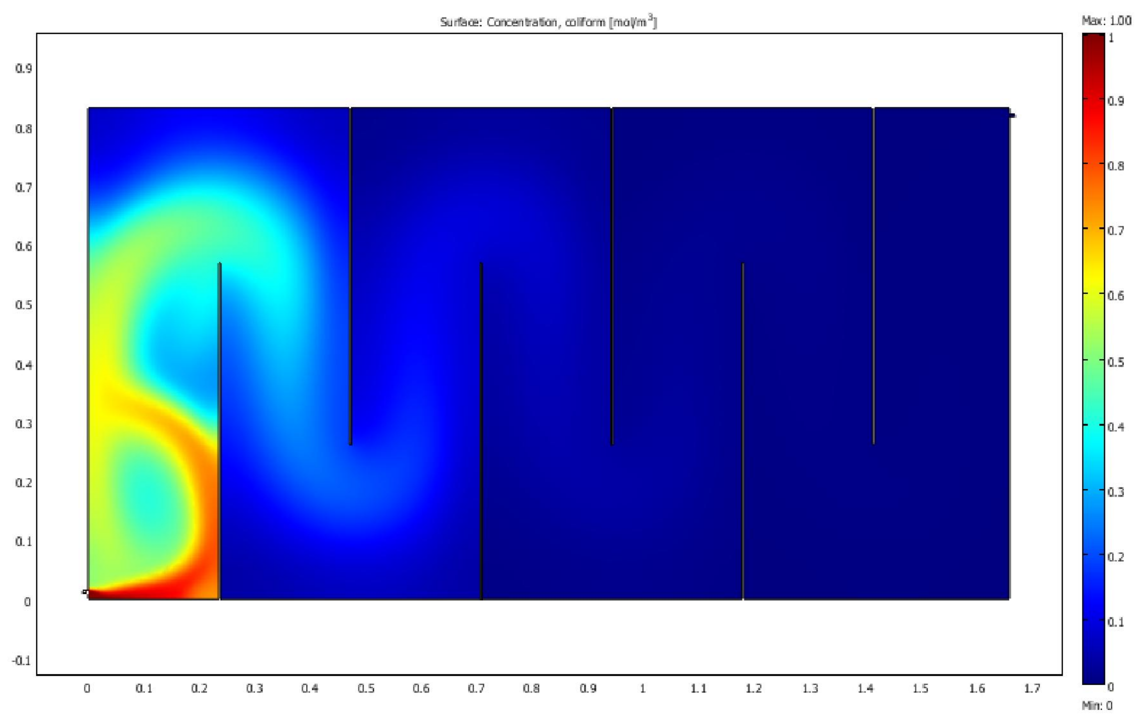

Figure 4.67 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for transverse baffle arrangement in facultative reactor (k = 13.686)

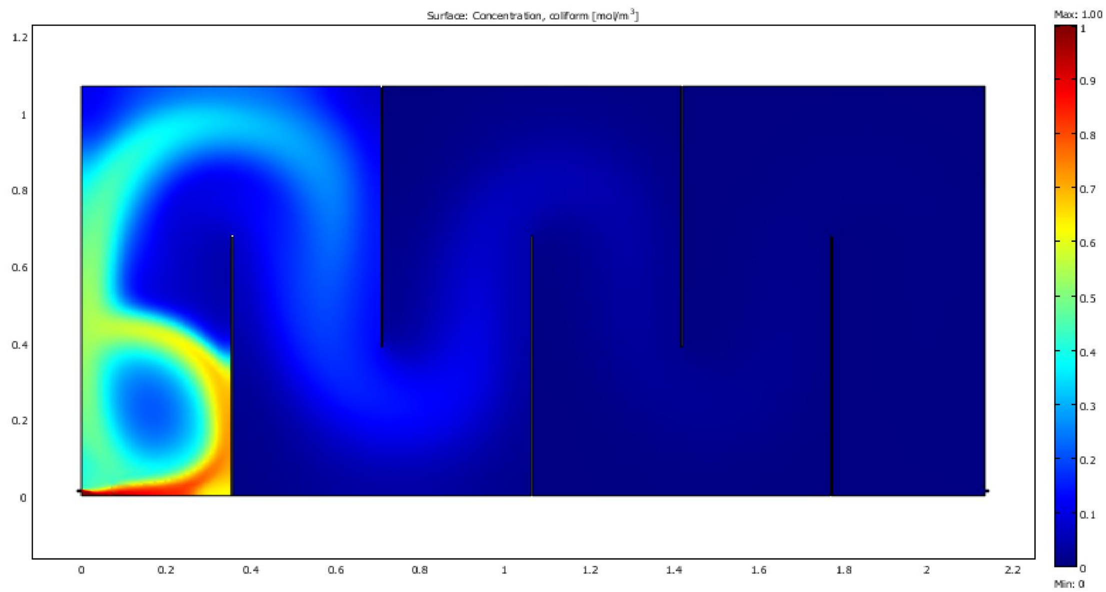

Figure 4.68 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for transverse baffle arrangement in maturation reactor (K= 13.686)

The short baffle in Figure 4.66 indicated that the optimal configuration may not require the inclusion of baffles. A model test was run with the same and physical properties without the inclusion of baffles to investigate the assumptions and the result of the faecal coliform log removal of 0.82 confirms that the optimized solution for anaerobic reactor at this value of k would achieve the same result without the inclusion of baffles. It is interesting to note that short-baffle in the reactor could reduce the influent momentum such that there is no direct path of wastewater flow that directly links the inlet and outlet. Short baffles are cost effective and could perform more satisfactorily when fitted near the inlet and outlet of a pond (Shilton and Harrison, 2003a). One could probably see a bit of stagnation region and short-circuiting effect in the first half of the reactor.

Figures 4.67 and 4.68 present the optimal design for the facultative and the maturation reactors. The optimal solution gives quite excellent performance with regards to the cost when compared with the minimum and maximum FC removal designs that are presented in Table 4.15. The result suggests that the effect of the k value has great significance in affecting the entire pond configurations which could vary from one location to another because the first order kinetics is a function of temperature and microbial activities within the reactors.

Table 4.16 shows all the properties of the model associated with the anaerobic, facultative and maturation longitudinal baffle arrangements at the upper boundary of 13.686 for k value using the Simplex algorithm. Overall result for both transverse and longitudinal indicated facultative to be the choice with a cost of **N** 5, 184.00. The optimized design flow patterns for the anaerobic, facultative and maturation reactors in Table 4.16 are presented in Figures 4.69-4.71.

Table 4.16 Sensitivity Analysis Results for Longitudinal baffle arrangement (k=13.686)

	Anaerobic Transverse SA1			**Facultative Transverse SA1**			**Maturation Transverse SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	1.33E-1	1.33E-1	1.17E-1	**4.82E-3**	4.82E-3	1.19E-3	1.85E-3	2.83E-3	1.15E-3
Cost (N)	1, 545	1, 545	1, 582	**5, 184**	5, 184	5, 265	8, 108	8, 609	8, 134
Log removal	0.88	0.88	0.93	**2.32**	2.32	2.92	2.73	2.55	2.94
Area ratio	1:1	1:1	1:1	**2:1**	2:1	2:1	4:1	3:1	4:1
Area (m^2)	1.75E-1	1.75E-1	1.68E-1	**1.38E0**	1.38E0	1.38E0	2.28E0	2.50E0	2.28E0
Depth (m)	1.13E-1	1.13E-1	1.18E-1	**4.80E-2**	4.80E-2	4.80E-2	3.60E-2	3.27E-2	3.60E-2
Length (m)	4.19E-1	4.19E-1	4.10E-1	**8.30E-1**	8.30E-1	8.30E-1	7.55E-1	9.14E-1	7.55E-3
Width (m)	4.19E-1	4.19E-1	4.10E-1	**8.30E-1**	8.30E-1	8.30E-1	7.55E-1	9.14E-1	7.55E-1
Velocity (m/s)	2.46E-3	2.46E-3	2.83E-1	**1.15E0**	1.15E0	1.44E0	2.13E0	1.91E0	2.25E0
Baffle length (m)	2.68E-1	2.68E-1	2.83E-1	**1.15E0**	1.15E0	1.44E0	2.13E0	1.91E0	2.25E0
Baffle ratio	64%	64%	69%	**70%**	70%	87%	71%	70%	75%
Number of baffles	5	5	5	**2**	2	2	2	2	2

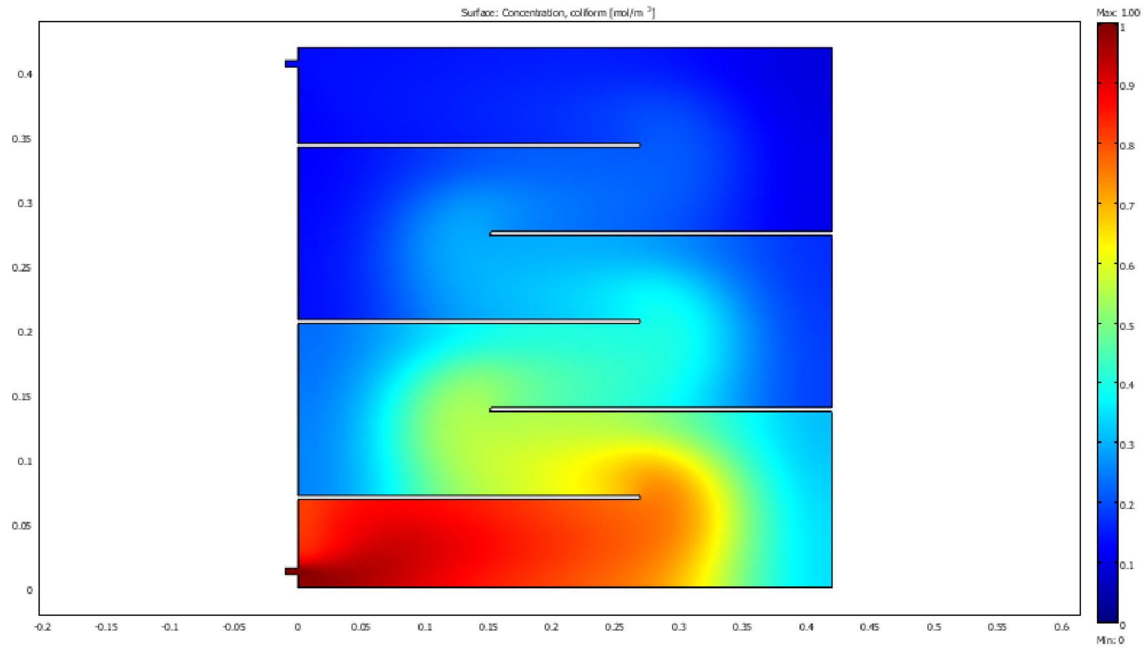

Figure 4.69 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in anaerobic reactor (k = 13.686)

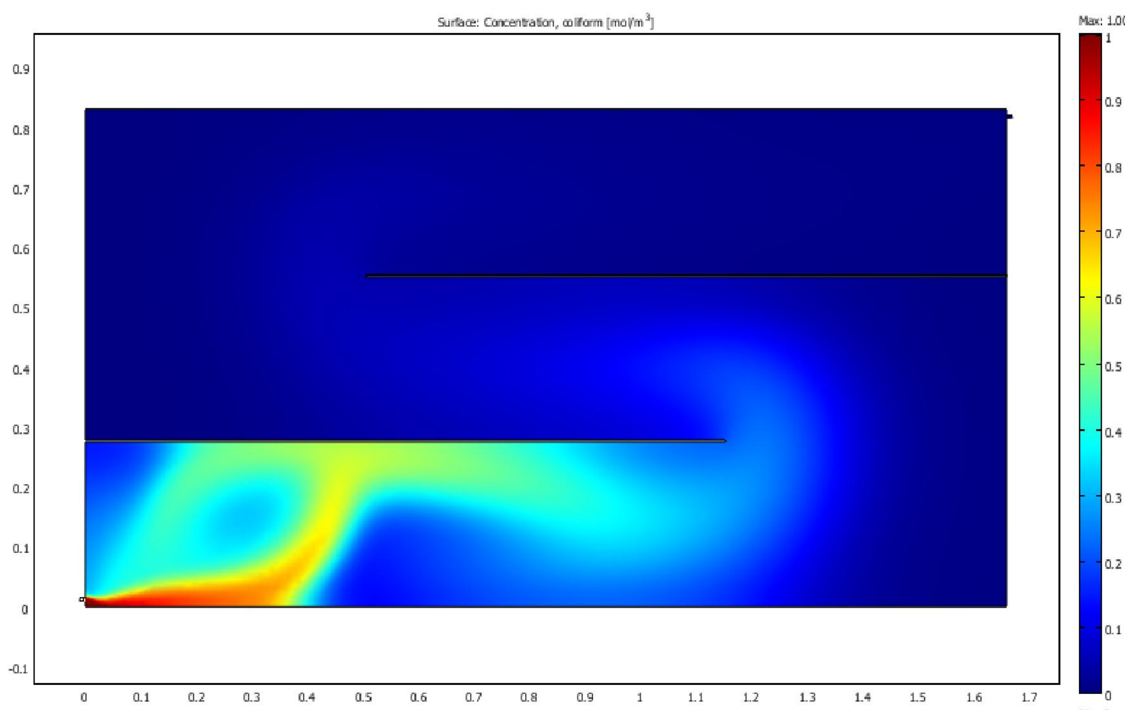

Figure 4.70 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in facultative reactor (k = 13.686)

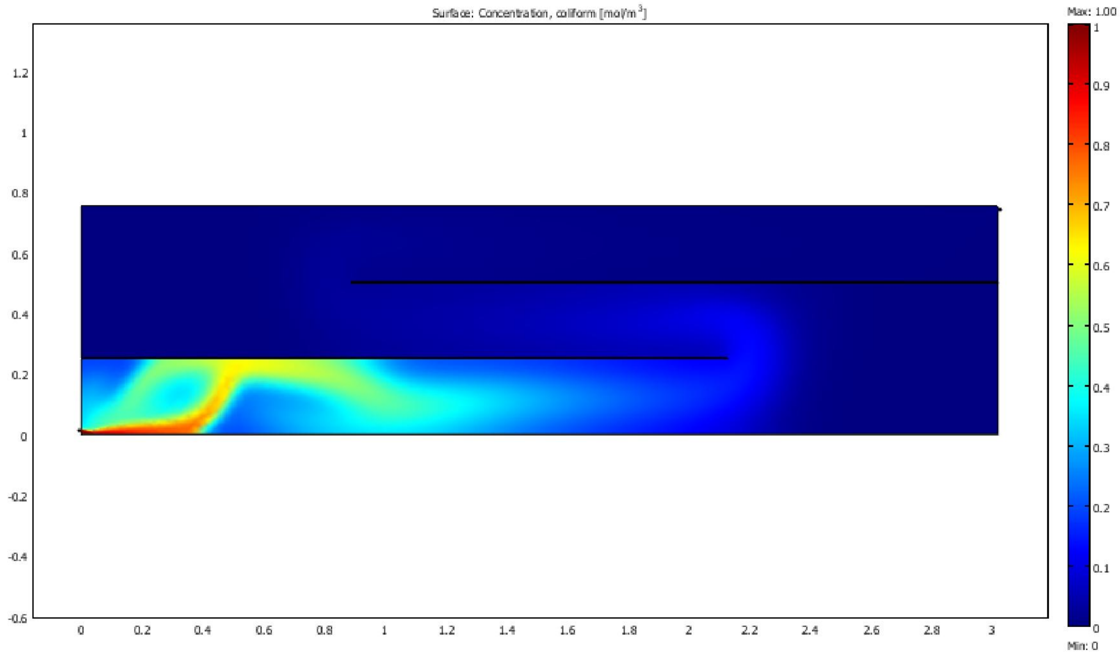

Figure 4.71 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in maturation reactor (k = 13.686)

The five-baffle anaerobic reactor in Figure 4.69 shows a similar design flow pattern as Figure 4.44 with the first order k value of 9.124. It has the same surface area ratio of 1:1 with baffle lengths of 0.268 m (64% pond width) spread evenly at a flow channel width of 0.067 m in the baffle compartment and a baffle opening of 0.151 m. With this configuration, a cummulatie log removal of 0.88 was achieved at the outlet. It can be seen also that there is a significant visible difference of flow pattern as the wastewater travels from one baffle compartment to the other.

Figures 4.70 and 4.71 describe the flow pattern in the facultative and maturation reactor with k value of 13.686 at the upper boundary for the sensitivity test. The number of baffles in both cases are maintained as two baffles but with a reduction in baffle lengths from 83% to 70 % in facultative and 82% to 71% in maturation reactors. A log removal of 2.32 and 2.73 was achieved with an assoicated cost of **N** 5,184.00 and **N** 8, 108.00 respectively. Tables 4.17 display the combination of SIMPLEX sensitivity analysis design results at upper disinfectant rate.

Table 4.17 SIMPLEX sensitivity analysis optimal design results for k = 13.686

	Anaerobic Transverse SA1			**Facultative Longitudinal SA1**			**Maturation Transverse SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 200**	1, 211	1, 767	**5, 184**	5, 184	5, 265	**7, 891**	7, 993	7, 992
Log removal	**0.82**	0.80	0.98	**2.32**	2.32	2.92	**2.34**	2.01	2.77
Reactor L/W ratio (r)	**3:1**	3:1	2:1	**2:1**	2:1	2:1	**2:1**	4:1	2:1
Depth (m)	**1.13E-1**	1.13E-1	6.24E-2	**4.80E-2**	4.80E-2	4.80E-2	**3.60E-2**	3.49E-2	3.60E-2
Baffle ratio	**8%**	10%	71%	**70%**	70%	87%	**64%**	45%	68%
Number of baffles	**3**	4	7	**2**	2	2	**5**	4	6

The results in Table 4.17 of the sensitivity analysis performed using the single object SIMPLEX program at the higher k value show a significant change in feacal coliform log removal as expected, however, with an appreciable difference in the associated cost at the higher disinfection rate constant (Table 4.17: **N**14, 275 as compared to Table 4.10: **N**13, 609).

Table 4.18 details the summary of the Simplex sensitivity test carried out on Transverse baffle arrangement. The same order of optimal solutions was observed as when the base value of 9.124 was used for k. The overall optimal solutions are: anaerobic and maturation for the transverse arrangement and one facultative from the longitudinal baffle arrangement. The optimized design flow patterns for the anaerobic, facultative and maturation reactors in Table 4.18 are presented in Figures 4.72-4.74.

Table 4.18 Sensitivity Analysis Results for Transverse baffle arrangement (k = 4.56)

	Anaerobic Transverse SA2			**Facultative Transverse SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**4.83E-1**	4.96E-1	4.64E-1	9.86E-2	1.92E-1	8.86E-2	**8.34E-2**	9.53E-2	4.62E-2
Cost (N)	**1, 192**	1, 237	1, 579	5, 412	5, 980	5, 480	**7, 991**	8, 698	8, 071
Log removal	**0.32**	0.30	0.33	1.01	0.72	1.05	**1.08**	1.02	1.30
Area ratio	**3:1**	3:1	4:1	4:1	4:1	4:1	**4:1**	3:1	4:1
Area (m^2)	**1.65E-1**	2.01E-1	2.66E-1	1.38E0	1.73E0	1.34E0	**2.28E0**	2.59E0	2.28E0
Depth (m)	**1.20E-1**	9.84E-2	7.44E-2	4.80E-2	3.84E-2	4.80E-2	**3.60E-2**	3.16E-2	3.60E-2
Length (m)	**7.03E-1**	7.76E-1	1.03E0	2.35E0	2.63E0	2.35E0	**3.02E0**	2.79E0	3.02E0
Width (m)	**2.34E-1**	2.59E-1	2.58E-1	5.87E-1	6.56E-1	5.87E-1	**7.55E-1**	9.30E-1	7.55E-1
Velocity (m/s)	**2.32E-3**	2.83E-3	3.74E-3	5.79E-3	7.24E-3	5.79E-3	**7.72E-3**	8.79E-3	7.72E-3
Baffle length (m)	**3.16E-2**	4.01E-2	1.33E-1	4.28E-1	2.43E-1	4.34E-1	**5.28E-1**	4.55E-1	5.58E-1
Baffle ratio	**14%**	16%	52%	73%	37%	74%	**70%**	49%	74%
Number of baffles	**2**	2	7	7	2	8	**6**	5	7

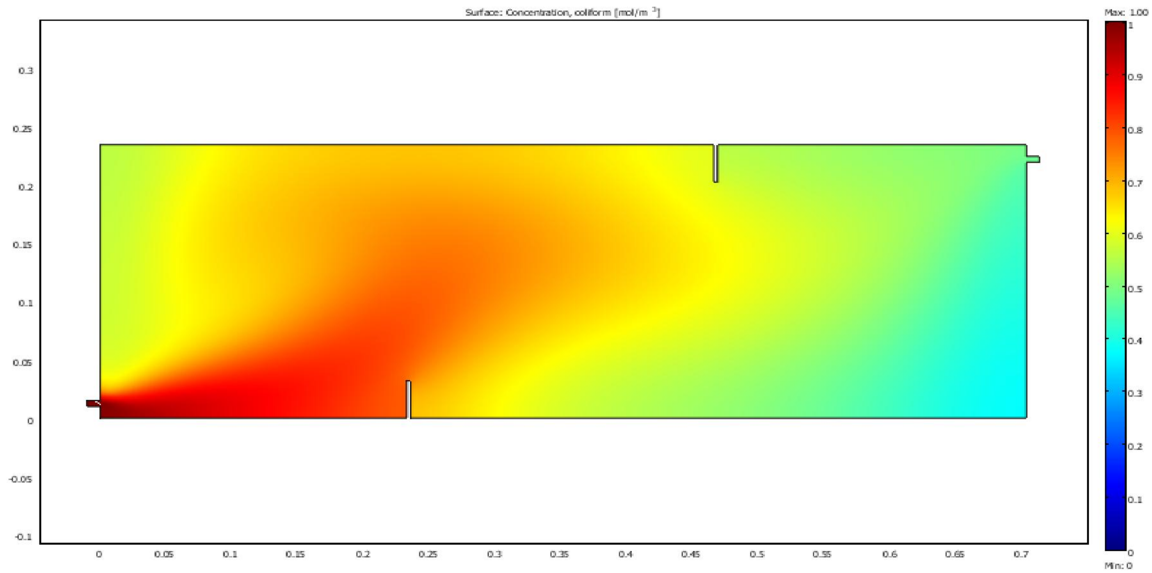

Figure 4.72 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for transverse baffle arrangement in anaerobic reactor (k = 4.562)

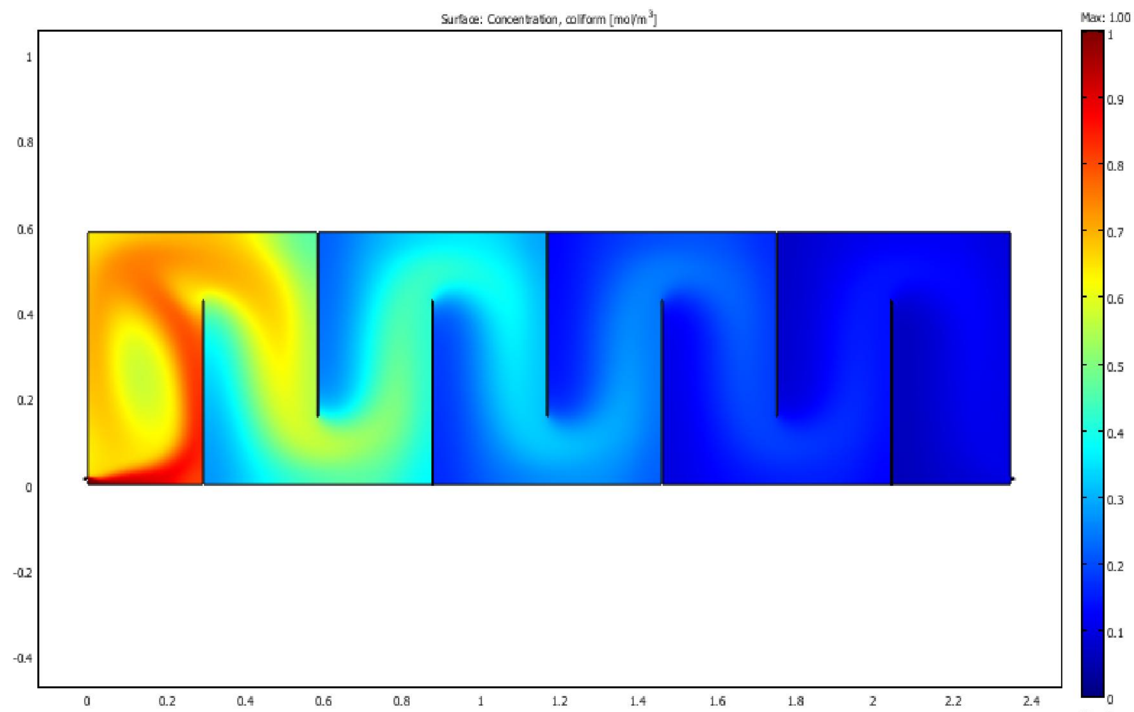

Figure 4.73 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for transverse baffle arrangement in facultative reactor (k = 4.562)

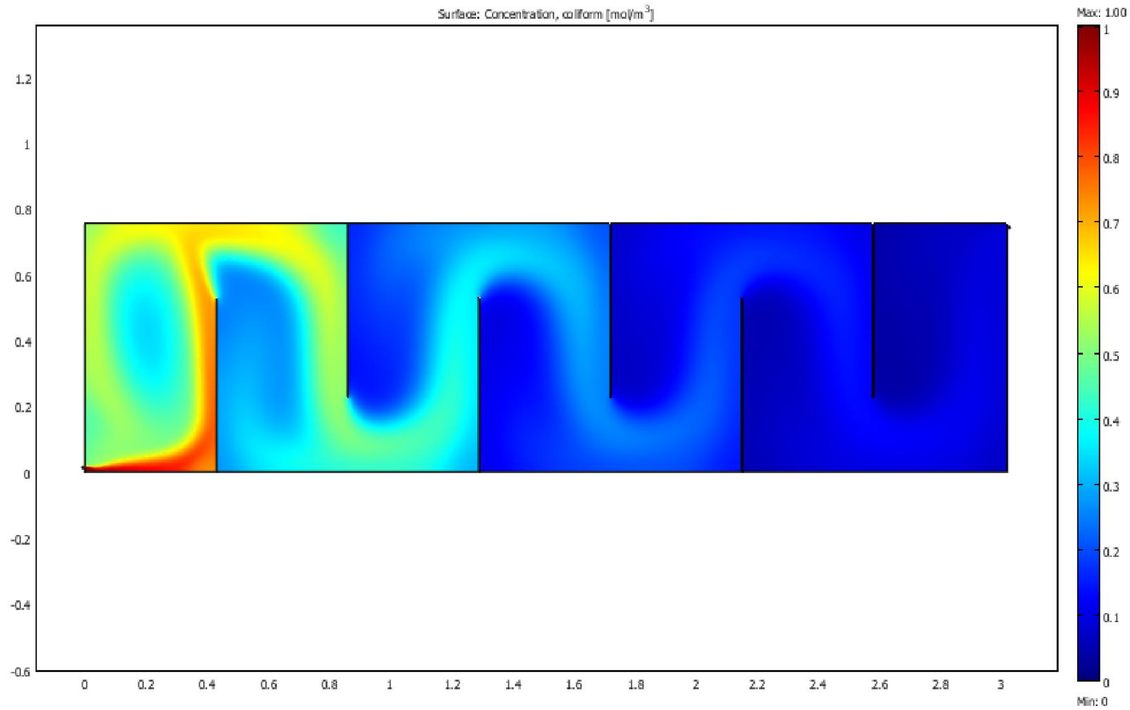

Figure 4.74 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for transverse baffle arrangement in maturation reactor (k = 4.562)

Figure 4.72 shows a short baffle anaerobic reactor indicating that the optimal configuration may not require the inclusion of baffles. A model test was also run with the same physical properties without the inclusion of baffles to investigate the assumptions and the result of the faecal coliform log removal of 0.32 confirms that the optimized solution for anaerobic reactor at this value of k would achieve the same result without the inclusion of baffles. It can be seen from the diagram that the effect of stagnation region and short-circuiting is well pronounced through the entire reactor space. This indicates degradation in the wastewater effluent quality as against when baffles of appreciable lengths are included.

Figures 4.73 and 4.74 present the optimal design for the facultative and the maturation reactors. The optimal solution gives quite excellent performance with regards to the cost when compared with the minimum and maximum FC removal designs that are presented in Table 4.11. A log removal of 1.01 and 1.083 was achieved with an assoicated cost of **N**5,412.00 and **N**7, 991.00 respectively.

Table 4.19 shows all the properties of the model associated with the anaerobic, facultative and maturation longitudinal baffle arrangements at the lower boundary of 4.562 for k value using the Simplex algorithm. Overall result for both transverse and longitudinal indicated facultative design to be the choice with a cost of **N** 5,220.00. The optimized design flow patterns for the anaerobic, facultative and maturation reactors in Table 4.19 are presented in Figures 4.75-4.77.

Table 4.19 Sensitivity Analysis Results for Longitudinal baffle arrangement (k= 4.562)

	Anaerobic Transverse SA2			**Facultative Transverse SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	4.87E-1	4.97E-1	4.65E-1	9.66E-2	9.66E-2	6.88E-2	3.97E-2	7.06E-2	3.94E-2
Cost (N)	1, 412	1, 492	1, 582	5, 220	5, 220	5, 750	8, 100	9, 322	8, 258
Log removal	0.31	0.30	0.33	1.01	1.01	1.16	1.40	0.97	1.40
Area ratio	1:1	1:1	1:1	2:1	2:1	3:1	3:1	4:1	4:1
Area (m^2)	1.75E-1	1.75E-1	1.68E-1	1.38E0	1.38E0	1.38E0	2.27E0	2.79E0	2.28E0
Depth (m)	1.13E-1	1.13E-1	1.18E-1	4.80E-2	4.80E-2	4.80E-2	3.60E-2	2.95E-2	3.60E-2
Length (m)	4.19E-1	4.19E-1	4.10E-1	1.66E0	1.66E0	2.03E0	2.61E0	3.34E0	3.02E0
Width (m)	4.19E-1	4.19E-1	4.10E-1	8.30E-1	8.30E-1	6.78E-1	8.71E-1	8.34E-1	7.55E-3
Velocity (m/s)	2.46E-3	2.46E-3	2.36E-3	5.79E-3	5.79E-3	5.79E-3	7.72E-3	9.44E-3	7.72E-3
Baffle length (m)	2.36E-1	2.36E-1	2.83E-1	1.28E0	1.28E0	1.93E0	2.38E0	1.32E0	2.82E0
Baffle ratio	57%	57%	69%	77%	77%	95%	91%	40%	94%
Number of baffles	4	5	5	2	2	3	2	2	2

The SIMPLEX optimal prediction for the maturation pond required longer baffles than both the anaerobic and facultative regardless of the microbial disinfection kinetics. Reactor area ratios were not sensitive to the disinfection rate constant for the anaerobic pond. However, for the facultative and maturation ponds, changes in area ratio were necessary to meet the target log reduction. Pond depth did not vary significantly for the facultative and maturation ponds but showed that with increasing disinfection rate constant, the pond depth decreased for the anaerobic pond.

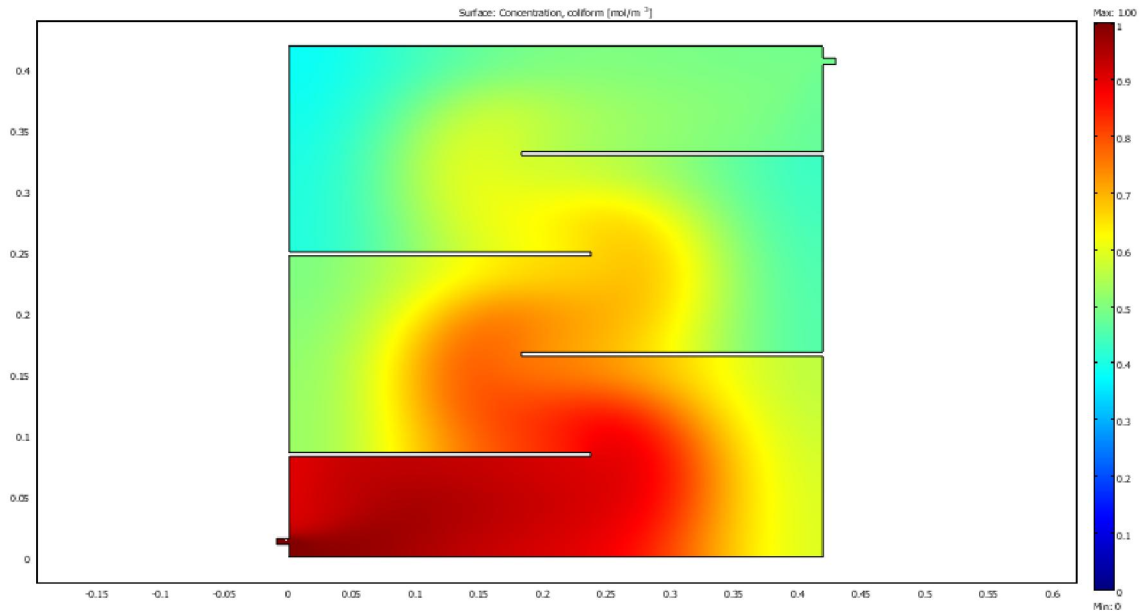

Figure 4.75 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in anaerobic reactor (k = 4.562)

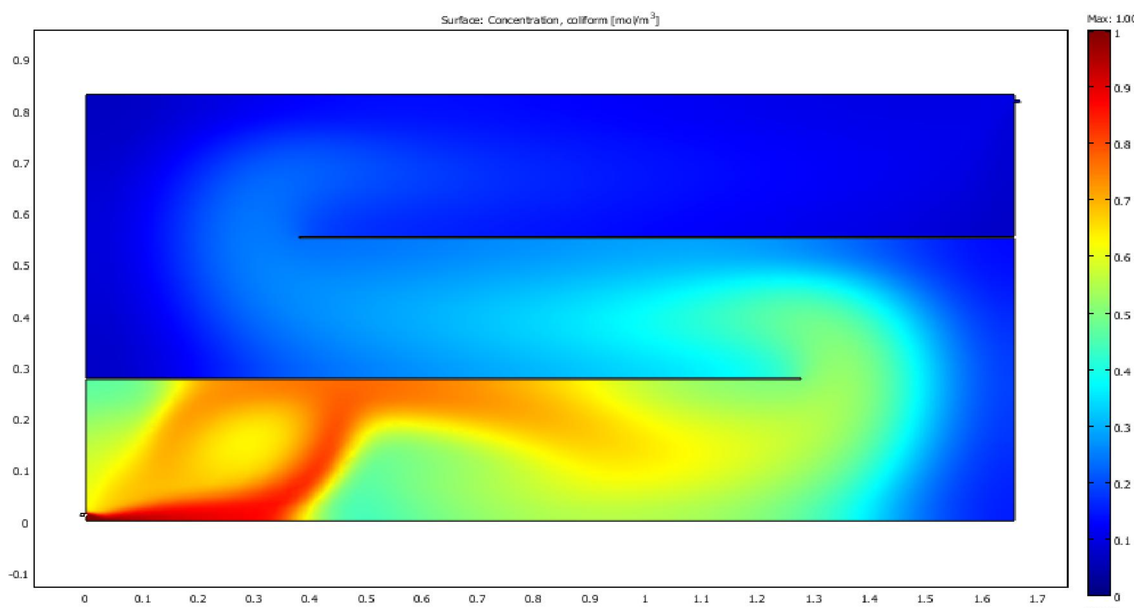

Figure 4.76 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in facultative reactor (k = 4.562)

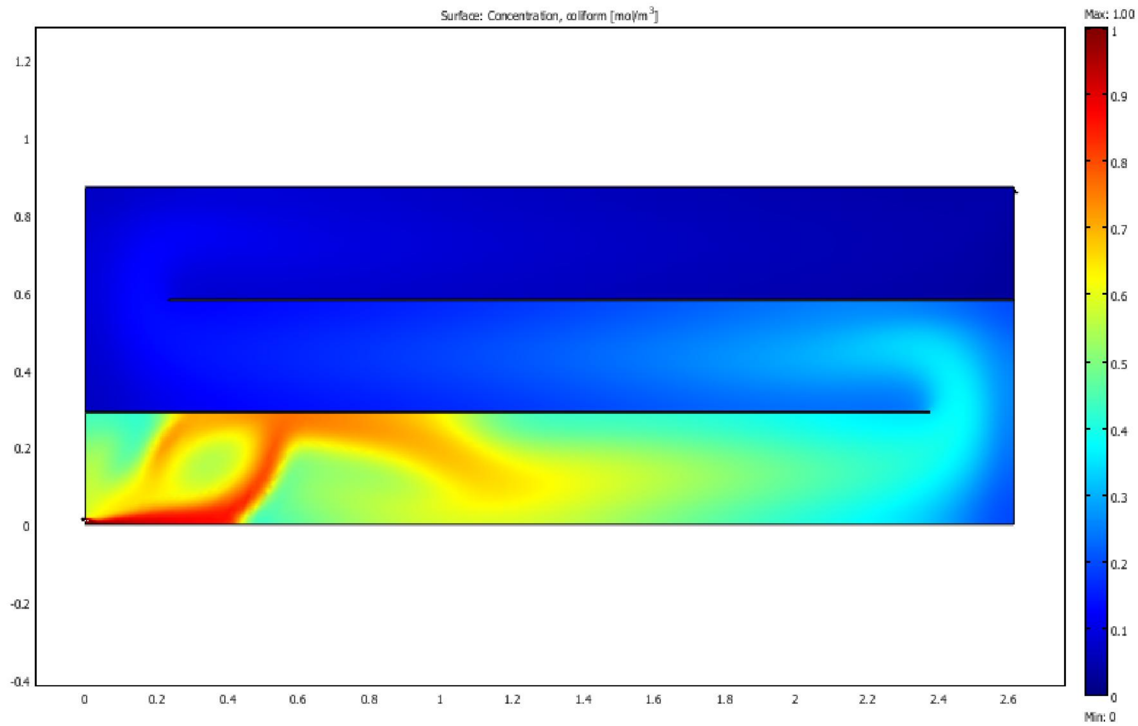

Figure 4.77 SIMPLEX sensitivity optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in maturation reactor (k = 4.562)

Figure 4.75 shows the design flow pattern with a four-baffle 57% pond-length anaerobic reactor. The baffle opening is quite wider than the baffle channel compartment which could be attributed to the rapid change from red to yellow color. This indicates the initiation of short circuitng as the yellow color moves along the baffle openings. With this configuration, a cummulative log removal of 0.31 was achieved at the outlet with a cost of **N**1,412.00.

Figures 4.76 show the two baffled facultative reactor that has a surface area ratio of 2:1 at depth of 0.048m to achieve an FC log-removal of 1.01 at a cost of **N**5, 220.00 while Figure 4.77 describes the design flow pattern in a 2-baffle longitudinal reactor having a depth of 0.036 with a 3:1 area ratio that cost **N**8,100.00. It can be seen that the design flow patterns are similar due to the fitting of long baffles that encourage the mixing of wastwater and also increases the length of the flow path from the inlet to the outlet in the two reactors.

Table 4.20 displays the combination of Tables 4.18 and 4.19 and it can be concluded that a small cost difference at the lower disinfection rate constant is achieved (Table 4.20: N14, 403). The SIMPLEX model predicted similar baffle lengths for disinfection rate constants below the base case of 9.12 for the maturation pond.

Table 4.20 SIMPLEX sensitivity analysis optimal design results for k = 4.562

	Anaerobic Transverse SA2			**Facultative Longitudinal SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 192**	1, 237	1, 579	**5, 220**	5, 220	5, 750	**7, 991**	8, 698	8, 071
Log removal	**0.32**	0.30	0.33	**1.01**	1.01	1.16	**1.08**	1.02	1.30
Reactor L/W ratio (r)	**3:1**	3:1	4:1	**2:1**	2:1	3:1	**4:1**	3:1	4:1
Depth (m)	**1.20E-1**	9.84E-2	7.44E-2	**4.80E-2**	4.80E-2	4.80E-2	**3.60E-2**	3.16E-2	3.60E-2
Baffle ratio	**14%**	16%	52%	**77%**	77%	95%	**70%**	49%	74%
Number of baffles	**2**	2	7	**2**	2	3	**6**	5	7

Comparing Tables 4.17 and 4.20 at higher and lower disinfectant rate, it can be concluded that at higher rate constants, a slightly lower baffle length was predicted as the optimal solution. Lower baffle length ratios were predicted for higher and lower rate constants for the facultative pond compared to the base case.

Very low baffle lengths were predicted for the anaerobic pond at both the higher and lower disinfection rate constants (8% pond- baffle length in Table 4.17 or the 14% pond baffle length in Table 4.20). A separate simulation was performed to verify the same anaerobic configuration without the use of baffles and the results were quite similar. These additional unbaffled simulations suggest that baffles may not be required to achieve a target log inactivation for a cost effective anaerobic pond design for some microbial disinfection kinetic rates.

The SIMPLEX prediction for the maturation pond required more baffles than both the anaerobic and facultative regardless of the microbial disinfection kinetics. Reactor area ratios were not sensitive to the disinfection rate constant for the anaerobic pond. However, for the facultative and maturation ponds, changes in area ratio were necessary to meet the target log reduction. Pond depth did not vary significantly for the facultative and maturation ponds but showed that with increasing disinfection rate constant, the pond depth decreased for the anaerobic pond.

4.5.5 Results of sensitivity analysis for MOGA II at upper and lower boundary.

The result of the sensitivity analysis obtained from the MOGA II optimization algorithms show that changing the rate constant resulting from either a change in the microbial disinfection kinetics or from ambient temperature changes has an impact on the optimal results. Table 4.21 details the summary of the MOGA II algorithm sensitivity test carried out on the transverse baffle arrangement.

Table 4.21 MOGA-II Sensitivity Analysis Results for Transverse baffle (k= 13.686)

	Anaerobic Transverse SA1			**Facultative Transverse SA1**			**Maturation Transverse SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**1.54E-1**	1.57E-1	9.93E-2	**1.25E-2**	1.25E-2	3.33E-3	2.60E-3	9.63E-3	1.17E-3
Cost (N)	**1, 219**	1, 369	1, 539	**4, 966**	4, 966	5, 235	8, 815	9, 154	9, 557
Log removal	**0.81**	0.80	1.00	**1.90**	1.90	2.48	2.59	2.02	2.93
Area ratio	**1:1**	1:1	1:1	**1:1**	1:1	1:1	1:1	1:1	2:1
Area (m^2)	**1.72E-1**	1.68E-1	1.72E-1	**1.37E0**	1.37E0	1.44E0	2.69E0	2.79E0	2.88E0
Depth (m)	**1.15E-1**	1.17E-1	1.15E-1	**4.80E-2**	4.80E-2	4.56E-2	3.05E-2	2.95E-2	2.84E-2
Length (m)	**4.14E-1**	4.10E-1	4.14E-1	**1.17E0**	1.17E0	1.20E0	1.64E0	1.67E0	2.40E0
Width (m)	**4.14E-1**	4.10E-1	4.14E-1	**1.17E0**	1.17E0	1.20E0	1.64E0	1.67E0	1.20E0
Velocity (m/s)	**2.41E-3**	2.36E-3	2.41E-3	**5.79E-3**	5.79E-3	6.10E-3	9.10E-3	9.44E-3	9.80E-3
Baffle length (m)	**1.91E-1**	2.71E-1	3.27E-1	**5.40E-1**	5.40E-1	8.19E-1	8.77E-1	1.20E0	7.94E-1
Baffle ratio	**46%**	66%	79%	**46%**	46%	68%	54%	72%	60%
Number of baffles	**2**	3	4	**2**	2	2	2	2	4

The same order of optimal solutions was observed as when the base value of 9.124 was used for k. The overall optimal solutions are: anaerobic and facultative designs from the transverse arrangement and one maturation reactor from the longitudinal baffle arrangement. The optimized design flow patterns for the three reactors in Table 4.21 are presented in Figures 4.78- 4.80.

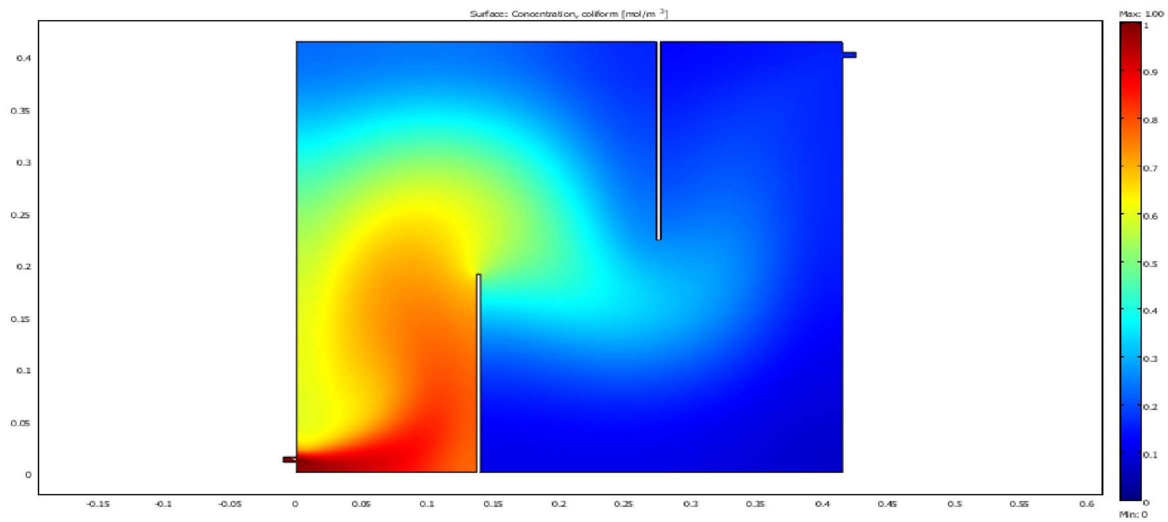

Figure 4.78 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in anaerobic reactor (k = 13.686)

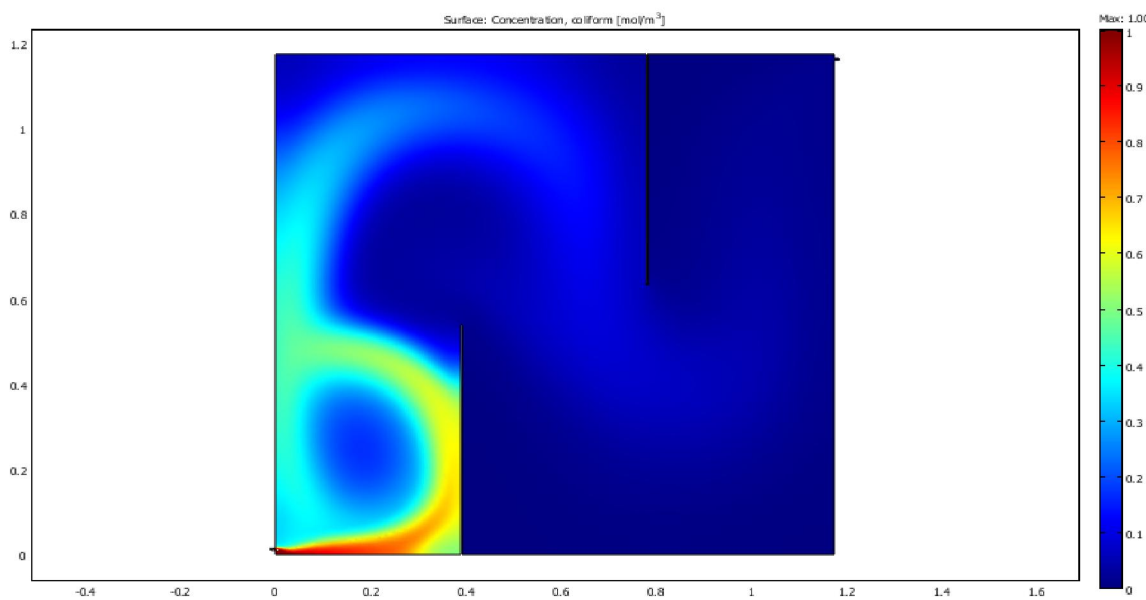

Figure 4.79 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in facultative reactor (k = 13.686)

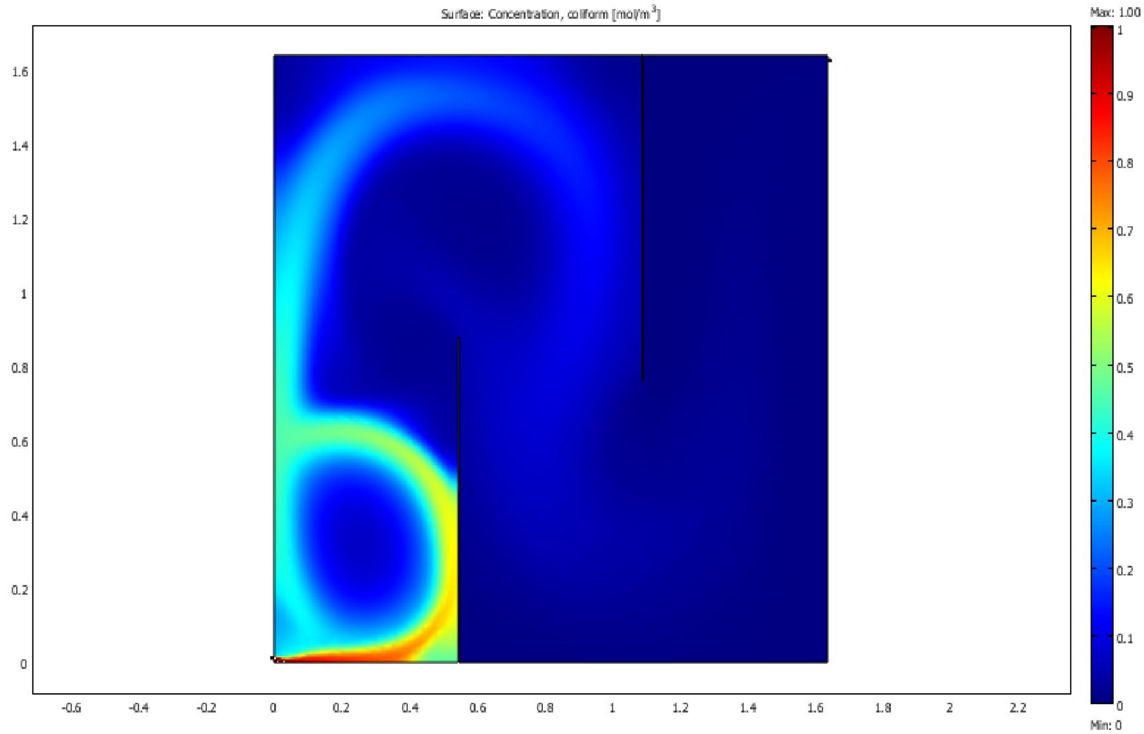

Figure 4.80 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in maturation reactor (k = 13.686)

Figures 4.78 and 4.79 share a similar property with 2-baffle 46% pond-width spread evenly across the length of the reactors while the maturation pond has a baffle length of 0.877 m at 54% pond width. The design flow pattern in the maturation reactor is similar to that in the facultative. It is shown that the baffle openings are wider than the baffle channel compartment in the facultative reactor which initiated the effect of short-circuiting and consequently reduce the quality of the effluent. The three optimized reactors in series have a log-removal of 0.81, 1.90 and 2.59 respectively. The circulating flow pattern close to inlet in the first baffle opening compartment which allows mixing to occur in the facultative and the maturation reactors before the wastewater was released to flow through the channels. The accrued costs of construction material for these three reactors are **N**1, 219.00, **N**4, 966.00 and **N**8, 815.00 respectively.

Table 4.22 shows all the properties of the model associated with the anaerobic, facultative and maturation longitudinal baffle arrangements at the upper boundary of 13.686 for K value using the MOGA II algorithm. Overall result for both transverse and longitudinal, indicated maturation design to be the choice with a cost of **N**7, 727.00. The optimized design flow for the anaerobic, facultative and maturation reactors in Table 4.22 are presented in Figures 4.81- 4.83.

Table 4.22 MOGA-II Sensitivity Analysis Results for Longitudinal arrangement (k = 13.686)

	Anaerobic Transverse SA1			**Facultative Transverse SA1**			**Maturation Transverse SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	9.67E-2	1.08E-1	9.47E-2	8.47E-3	1.23E-2	1.02E-3	**5.21E-3**	9.85E-3	1.06E-3
Cost (N)	1, 608	1, 784	1, 762	5, 032	5, 872	5, 836	**7, 727**	8, 457	7, 873
Log removal	1.01	0.97	1.02	2.07	1.91	2.99	**2.28**	2.01	2.98
Area ratio	1:1	3:1	4:1	1:1	4:1	3:1	**1:1**	4:1	2:1
Area (m^2)	2.17E-1	2.17E-1	1.68E-1	1.38E0	1.38E0	1.45E0	**2.28E0**	2.28E0	2.29E0
Depth (m)	9.12E-2	9.12E-2	1.18E-1	4.80E-2	4.80E-2	4.56E-2	**3.60E-2**	3.60E-2	3.60E-2
Length (m)	4.65E-1	8.06E-1	8.20E-1	1.17E0	2.35E0	2.09E0	**1.51E0**	3.02E0	2.14E0
Width (m)	4.65E-1	2.69E-1	2.05E-1	1.17E0	5.87E-1	6.95E-1	**1.51E0**	7.55E-3	7.72E3
Velocity (m/s)	3.05E-3	3.05E-3	2.36E-3	5.79E-3	5.79E-3	6.10E-3	**7.72E-3**	7.72E-3	7.72E-3
Baffle length (m)	4.10E-1	6.65E-1	7.58E-1	7.69E-1	1.55E0	1.76E0	**1.12E0**	1.87E0	1.61E0
Baffle ratio	88%	83%	93%	66%	66%	85%	**74%**	62%	76%
Number of baffles	4	3	2	2	4	3	**2**	4	2

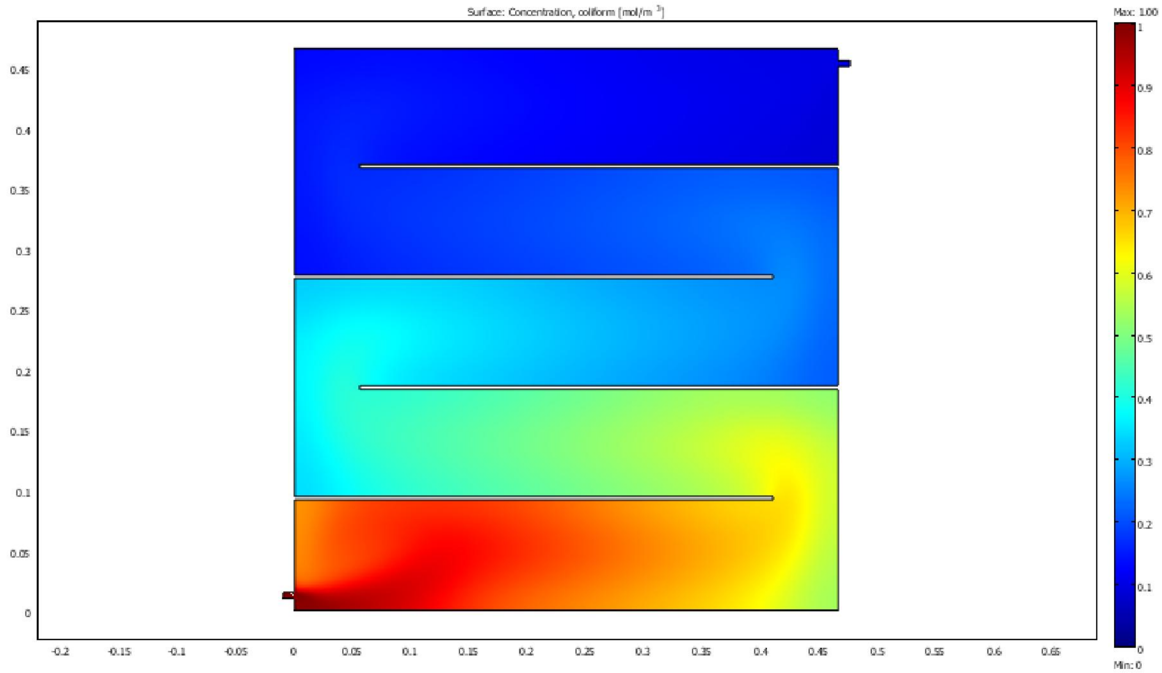

Figure 4.81 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in anaerobic reactor (k = 13.686)

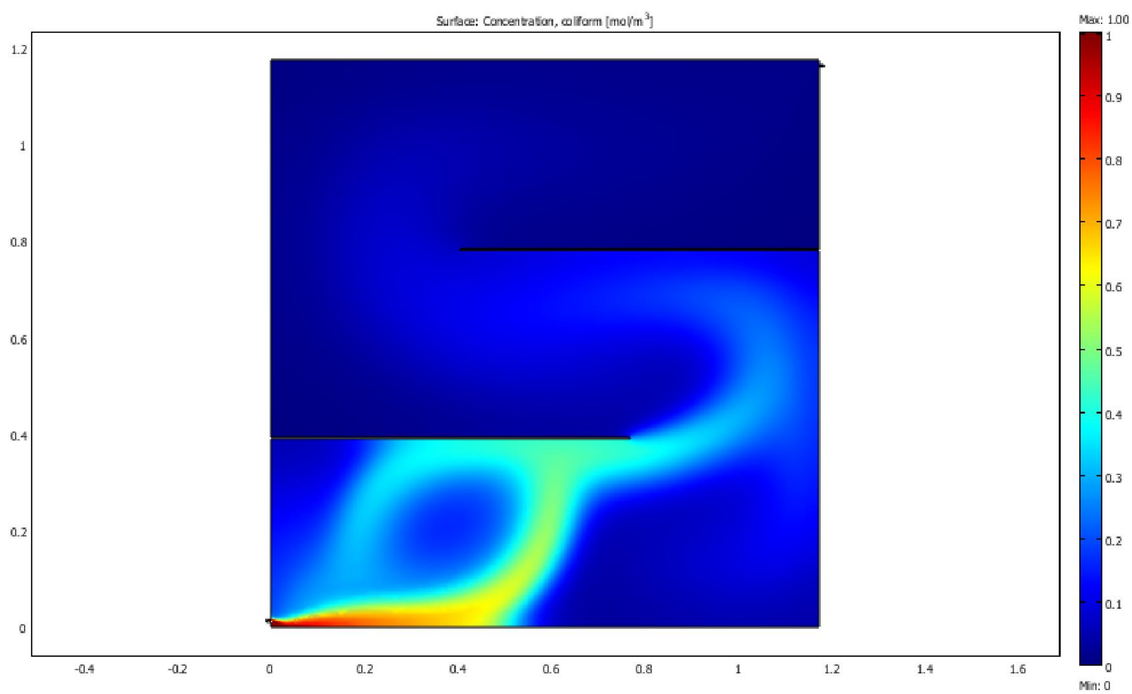

Figure 4.82 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in facultative reactor (k = 13.686)

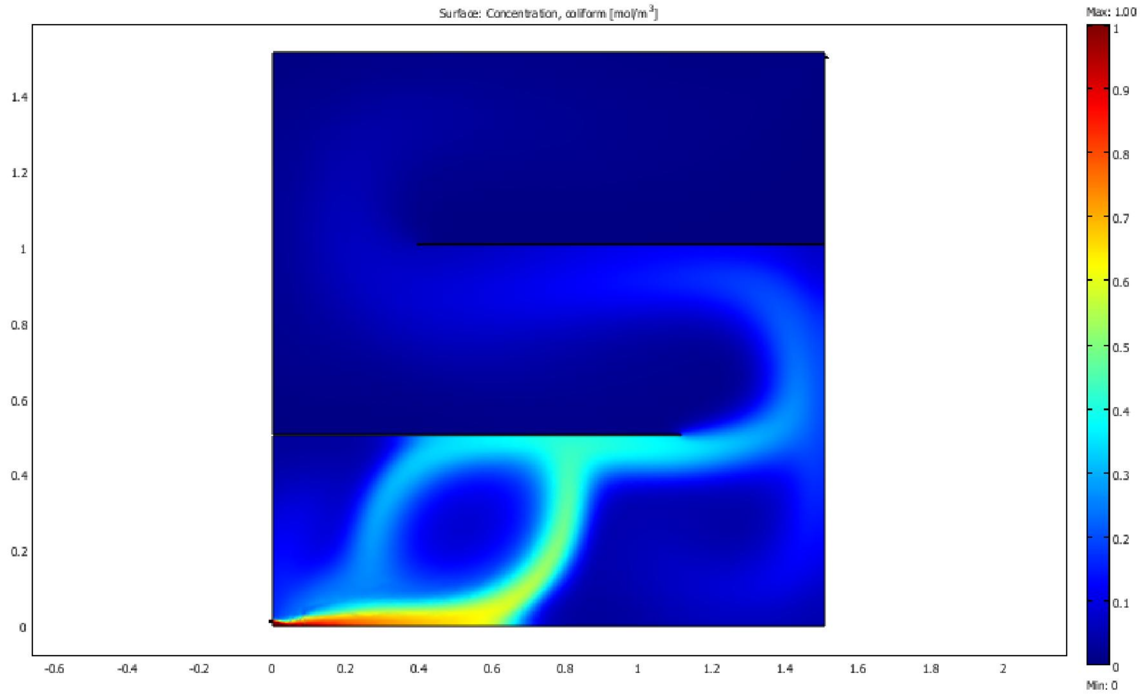

Figure 4.83 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in maturation reactor (k = 13.686)

Figure 4.81 shows the design flow pattern on a ratio 1:1 surface plane at a depth 0.091 m below the anaerobic reactor surface. It consist of 4 baffles with baffle lengths of 0.410 m (88% pond width) with a flow channel width of 0.091 m in the baffle compartment and a baffle opening of 0.055 m. With this configuration, a cummulative log removal of 1.01 was achieved at the outlet. The red color runs almost through the length of the baffle in the first compartment before chanign to yellow. Good mixing characterise this type of flow pattern where as in Figures 4.82 and 4.83, the flow pattern in the facultative and maturation reactor are similar. The two number of baffles in both cases are maintained as in the case of the 9.124 k value but with an increase in baffle length from 65% to 66 % in facultaive and a reduction from 81% to 74% in maturation reactors. A log removal of 2.07 and 2.28 was achieved with an assoicated cost of **N**5,032.00 and **N**7, 727.00 respectively.

Table 4.23 display significant differences in optimal design configurations compared to the SIMPLEX results in Table 4.17. These differences include baffle length ratios of 46%, 46% and 74% for the MOGA-II three ponds as compared to 8%, 70% and 64 % for the SIMPLEX's three ponds.

Table 4.23 MOGA-II Sensitivity Analysis Optimal Design Results k = 13.686

	Anaerobic Transverse SA1			**Facultative Transverse SA1**			**Maturation Longitudinal SA1**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 219**	1, 369	1, 539	**4, 966**	4, 966	5, 235	**7, 727**	8, 457	7, 873
Log removal	**0.81**	0.80	1.00	**1.90**	1.90	2.48	**2.28**	2.01	2.98
Area ratio	**1:1**	1:1	1:1	**1:1**	1:1	1:1	**1:1**	4:1	2:1
Depth (m)	**1.15E-1**	1.17E-1	1.15E-1	**4.80E-2**	4.80E-2	4.56E-2	**3.60E-2**	3.60E-2	3.60E-2
Baffle ratio	**46%**	66%	79%	**46%**	46%	68%	**74%**	62%	76%
Number of baffles	**2**	3	4	**2**	2	2	**2**	4	2

The reactor area ratio (1:1) for the optimal configurations and the number of baffles (2) were the same for all three ponds in the MOGA-II analysis as compared (3:1, 2:1 and 2:1) and (3, 2 and 5), respectively, in the SIMPLEX sensitivity optimal design. The overall MOGA-II feacal log removal was a half log lower (4.99 vs 5.48) with a lower cost (i.e., a difference of **N**363).

Table 4.24 shows all the properties of the model associated with the anaerobic, facultative and maturation transverse baffle arrangements at the lower boundary of 4.562 for k value using the MOGA II algorithm. Overall result for both transverse and longitudinal configurations indicated anaerobic, facultative and maturation design in transverse arrangement to be the optimal designs. The optimized design flow patterns for the anaerobic, facultative and maturation reactors in Table 4.24 are presented in Figures 4.84 - 4.86.

Table 4.24 MOGA-II Sensitivity Analysis Results for Transverse arrangement (k = 4.562)

	Anaerobic Transverse SA2			**Facultative Transverse SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
FC out	**4.99E-1**	4.99E-1	4.50E-1	**1.94E-1**	1.94E-1	7.51E-2	**1.53E-1**	1.58E-1	3.53E-2
Cost (N)	**1, 188**	1, 476	1, 538	**4, 931**	4, 974	5, 385	**7, 623**	9, 566	9, 626
Log removal	**0.30**	0.30	0.35	**0.71**	0.71	1.1	**0.81**	0.80	1.42
Area ratio	**2:1**	3:1	1:1	**2:1**	3:1	1:1	**1:1**	3:1	1:1
Area (m^2)	**1.79E-1**	3.17E-1	2.66E-1	**1.38E0**	1.38E0	1.37E0	**2.28E0**	2.90E0	2.89E0
Depth (m)	**1.10E-1**	6.24E-2	7.44E-2	**4.80E-2**	4.80E-2	4.80E-2	**3.60E-2**	2.84E-2	2.84E-2
Length (m)	**5.98E-1**	9.75E-1	5.15E-1	**1.66E0**	2.03E0	1.17E0	**1.51E0**	2.95E0	1.70E0
Width (m)	**2.99E-1**	3.25E-1	5.15E-1	**8.30E-1**	6.78E-1	1.17E0	**1.51E0**	9.82E-1	1.70E0
Velocity (m/s)	**2.52E-3**	4.46E-3	3.74E-3	**5.79E-3**	5.79E-3	5.79E-3	**7.72E-3**	9.80E-3	9.80E-3
Baffle length (m)	**8.53E-2**	1.06E-1	4.20E-1	**9.13E-2**	6.78E-2	9.98E-1	**6.34E-1**	8.79E-1	1.46E0
Baffle ratio	**29%**	33%	82%	**11%**	10%	85%	**42%**	90%	86%
Number of baffles	**2**	2	3	**6**	6	4	**2**	3	3

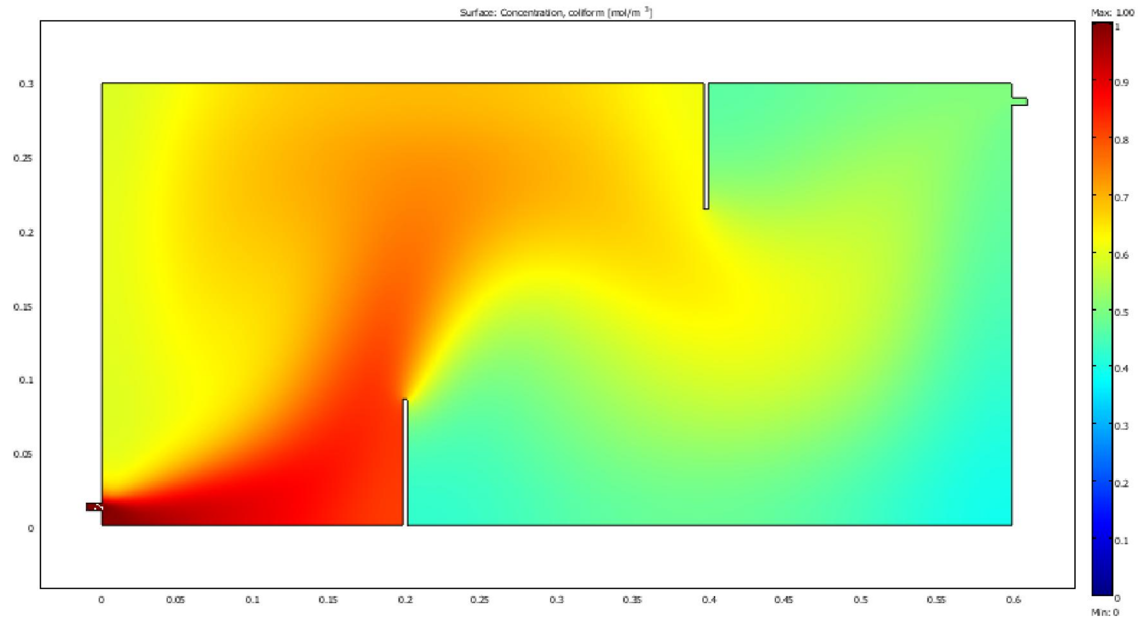

Figure 4.84 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in anaerobic reactor (k = 4.562)

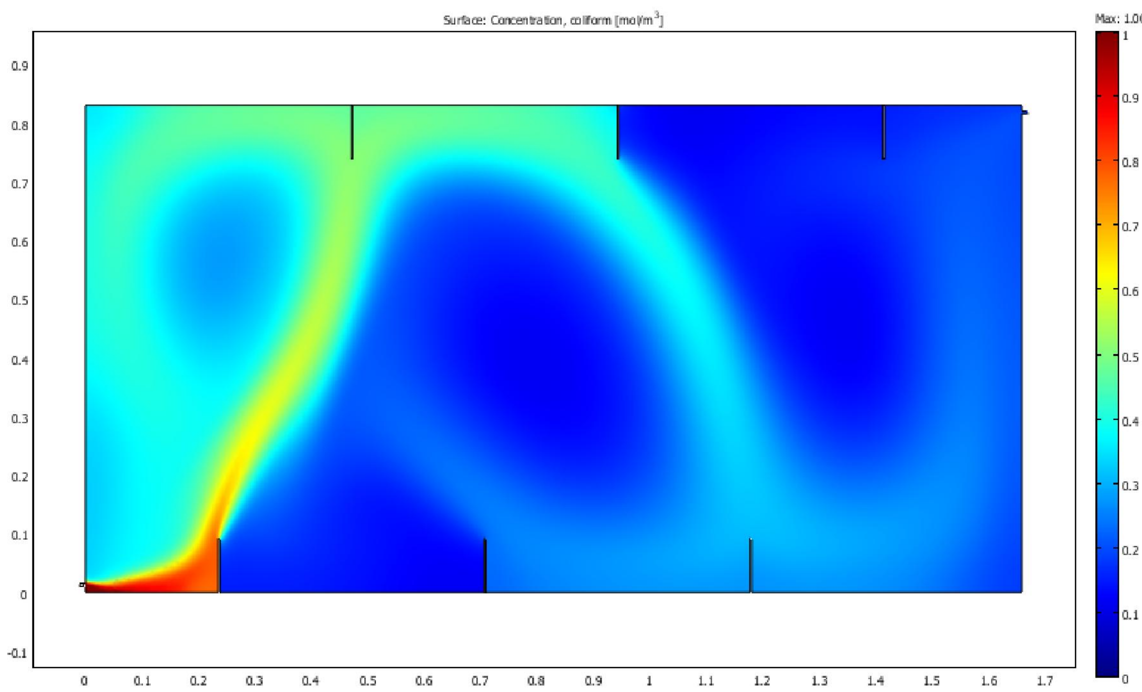

Figure 4.85 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in facultative reactor (k = 4.562)

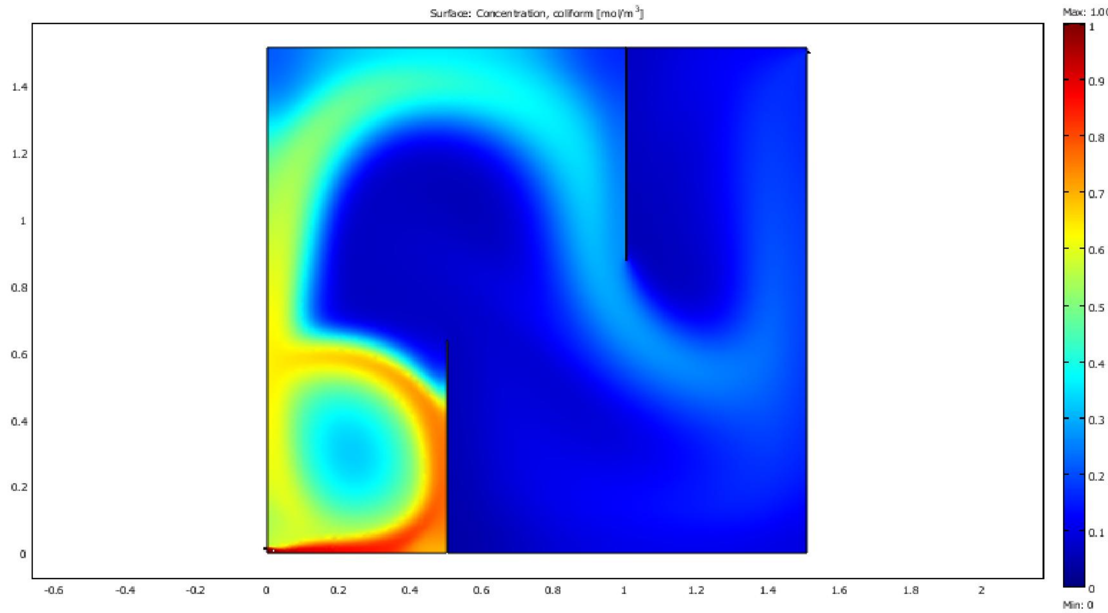

Figure 4.86 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for transverse baffle arrangement in maturation reactor (k = 4.562)

Figure 4.84 shows the design flow pattern with a four-baffle 29% pond-length anaerobic reactor. The baffle opening is quite wider than the baffle channel compartment which could be attributed to the rapid change from red to yellow color. This indicates the initiation of short circuitng as the yellow color moves along the baffle openings. With this configuration, a cummulative log removal of 0.30 was achieved at the outlet with a cost of **N**1,188.00.

The short 6-baffle facultative reactor in Figure 4.85 shows a short and unique flow pattern. The circulation of wastewater towards the middle of the reactor shows the effect of short baffle of the hydraulic performance of WSP. With this arrangemnt, a log removal of 0.71 was achieved with a cost of 4, 931.00 while Figure 4.86 has only two baffles to achieve the optimal design for lower boundary k value in the maturation reactor. In comparing these reactors with the longitudinal arrangment at lower k value for MOGA-II algortihm, the three design configuration of the transverse arrangements gave the optimal designs with a cost of **N**1,188.00, **N**4,931.00, and **N**7,623.00 respectively.

Table 4.25 shows all the properties of the model associated with the anaerobic, facultative and maturation longitudinal baffle arrangements at the lower boundary of 4.562 for k value using the MOGA II algorithm. Overall result for both transverse and longitudinal indicated transverse configuration as the optimal solution for all the reactors. The optimized design flow patterns for the anaerobic, facultative and maturation reactors in Table 4.25 are presented in Figures 4.87- 4.89.

Table 4.25 MOGA-II Sensitivity Analysis Results for Longitudinal arrangement (k= 4.56)

	Anaerobic Transverse SA2			**Facultative Transverse SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Design ID	93	125	35	44	317	331	214	214	35
FC out	4.87E-1	4.98E-1	4.51E-1	1.65E-1	1.81E-1	6.64E-2	1.52E-1	1.52E-1	3.89E-2
Cost ( )	1, 194	1, 246	1, 636	5, 034	5, 553	6, 380	7, 663	7, 663	8, 538
Log removal	0.31	0.30	0.34	0.78	0.74	1.18	0.82	0.82	1.41
Area ratio	3:1	3:1	3:1	4:1	2:1	2:1	1:1	1:1	4:1
Area (m^2)	1.65E-1	2.06E-1	2.11E-1	1.38E0	1.53E0	1.72E0	2.28E0	2.28E0	2.28E0
Depth (m)	1.20E-1	9.60E-2	9.36E-2	4.80E-2	4.32E-2	3.84E-2	3.60E-2	3.60E-2	3.60E-2
Length (m)	7.03E-1	7.86E-1	7.96E-1	2.35E0	1.75E0	1.86E0	1.51E0	1.51E0	3.02E0
Width (m)	2.34E-1	2.62E-1	2.65E-1	5.87E-1	8.75E-1	9.28E-1	1.51E0	1.51E0	7.55E-1
Velocity (m/s)	2.32E-3	2.90E-3	2.97E-3	5.79E-3	6.44E-3	7.24E-3	7.72E-3	7.72E-3	7.72E-3
Baffle length (m)	3.51E-2	4.32E-2	7.24E-2	1.88E-1	1.08E0	1.65E0	8.23E-1	8.23E-1	2.75E0
Baffle ratio	5%	6%	91%	8%	62%	89%	55%	55%	91%
Number of baffles	2	2	2	2	2	3	2	2	3

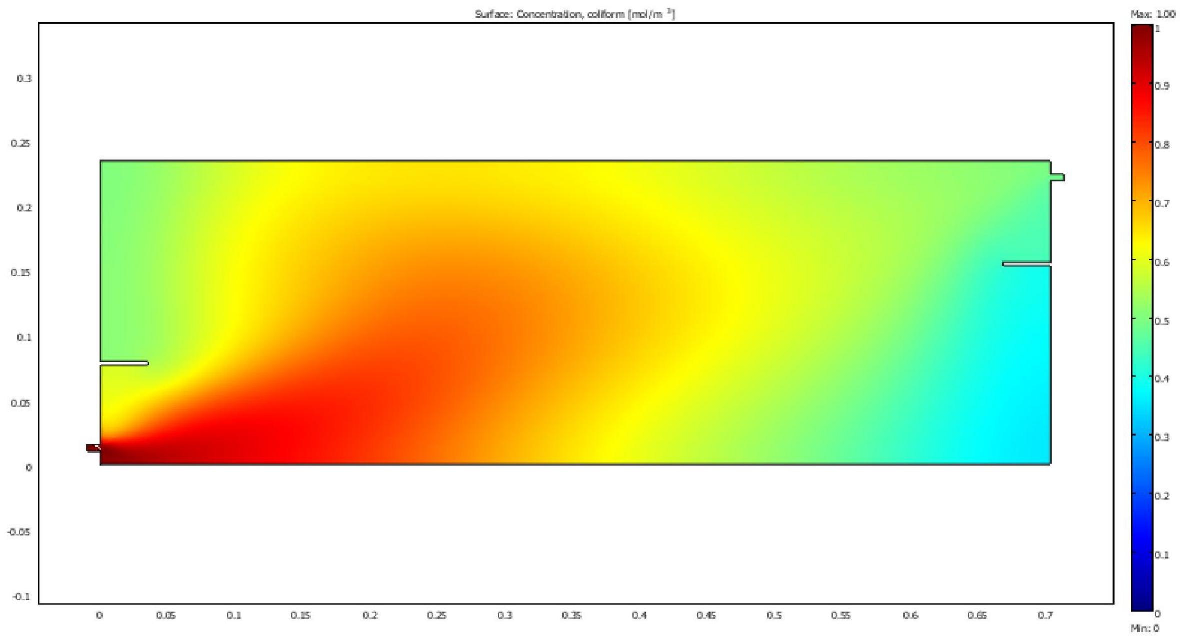

Figure 4.87 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in anaerobic reactor (k= 4.562)

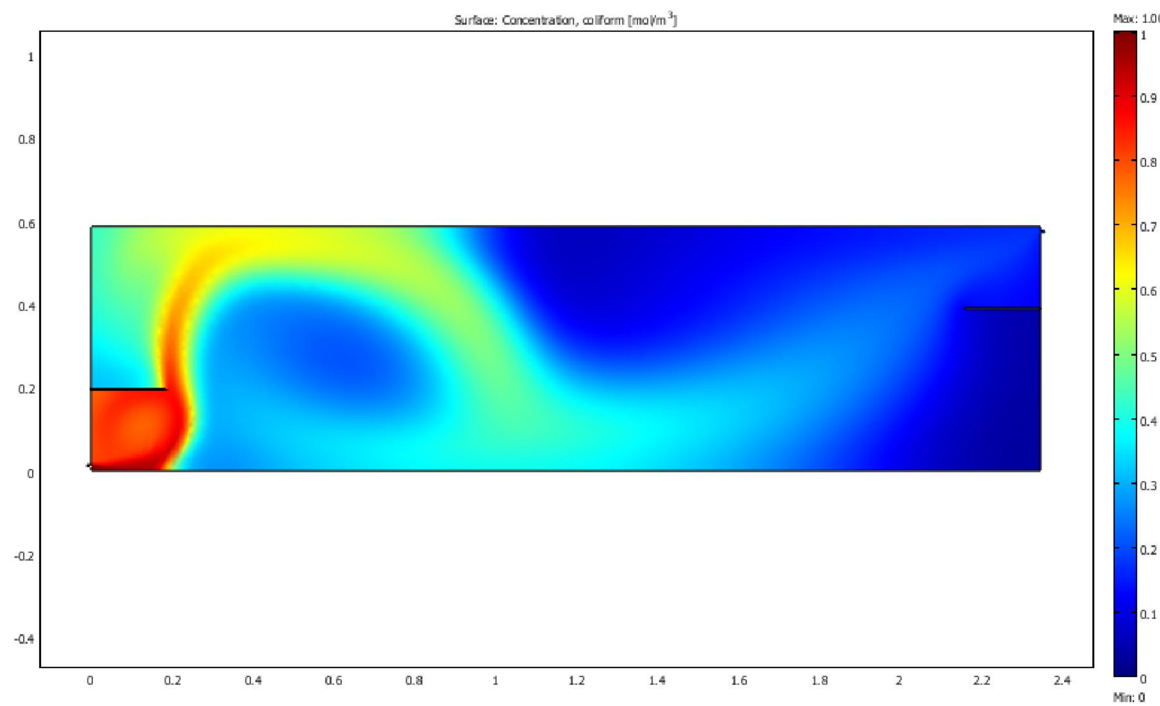

Figure 4.88 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in facultative reactor (k = 4.562)

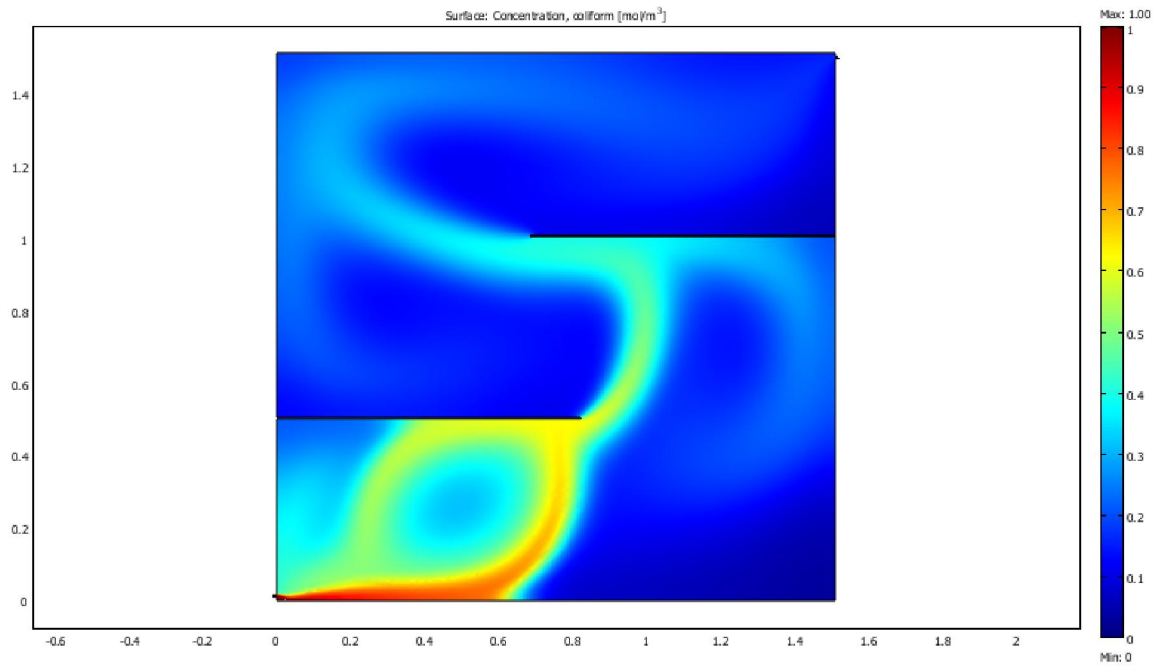

Figure 4.89 MOGA II sensitivity analysis optimal faecal coliform removal design with least cost for longitudinal baffle arrangement in maturation reactor (k = 4.562)

Figure 4.87 shows a short baffle anaerobic reactor indicating that the optimal configuration may not require the inclusion of baffles. A model test was also run with the same physical properties without the inclusion of baffles to investigate the assumptions short-circuiting and the result of the faecal coliform log removal of 0.31 confirms that the optimized solution for anaerobic reactor at this value of k would achieve the same result without the inclusion of baffles. It can be seen from the diagram that the effect of stagnation region and short-circuiting is well pronounced through the entire reactor space. This indicates degradation in the wastewater effluent quality as against when baffles of appreciable lengths are included.

However, it is interesting to see in Figure 4.88 a short baffle of 8% in the facultative reactor exhibit the flow pattern as compared to the anaerobic reactor. This could be due to the length of the reactor and the area ratio of 4:1 which allows a circulatory pattern in the first part and then flows sinusoidally to the outlet to achieve a log removal of 0.78 at an accrued cost of **N**5,034.00.

Figures 4.89 shows the two baffled maturation reactor that has a surface area ratio of 1:1 at depth of 0.036m to achieve an FC log-removal of 0.82 at a cost of **N**7, 663.00. Another unique flow pattern can be seen as the wastewater flows from the first baffle compartment to the second. The circulatory yellowish-red color indicates the effect of stagnation close to the inlet of the first baffle compartment. Also, evident is mixing of the wastewater in the first baffle compartment before it travels through other baffle compartments. Parameters that describe these reactors are presented in Table 4.26.

Table 4.26 displays combination of the optimal results of the sensitivity analysis performed using the MOGA-II multi-objective program at the lower k value. The MOGA-II results at the lower disinfection rate constant also produced a cheaper design at the expense of losing a half log reduction compared to the SIMPLEX design. Differences were also noted in the baffle arrangement, lengths, and number with no change in pond depths.

Table 4.26 MOGA-II Sensitivity Analysis Optimal Design Results k = 4.562

	Anaerobic Transverse SA2			**Facultative Transverse SA2**			**Maturation Transverse SA2**		
	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal	Optimal design	Min. FC removal	Max FC removal
Cost (N)	**1, 188**	1, 476	1, 538	**4, 931**	4, 974	5, 385	**7, 623**	9, 566	9, 626
Log removal	**0.30**	0.30	0.35	**0.71**	0.71	1.1	**0.81**	0.80	1.42
Reactor L/W ratio (r)	**2:1**	3:1	1:1	**2:1**	3:1	1:1	**1:1**	3:1	1:1
Depth (m)	**1.10E-1**	6.24E-2	7.44E-2	**4.80E-2**	4.80E-2	4.80E-2	**3.60E-2**	2.84E-2	2.84E-2
Baffle ratio	**29%**	33%	82%	**11%**	10%	85%	**42%**	90%	86%
Number of baffles	**2**	2	3	**6**	6	4	**2**	3	3

4.5.6 Summary of the optimization model results

The use of the optimization tool in this research has helped in finding a solution to satisfy specific cost objective. The outcome is an optimized model geometries that can predict precisely the velocity distribution, residence time distribution and faecal coliform concentration at all points in the reactor of which the reactor effluent and the accrued cost of material for construction is of utmost interest to the researcher. Several designs were produced of which the optimal, minimum and maximum faecal coliform removal designs were selected for the purpose of comparison and for selection by any designer who would like to choose designs based on some achievable objectives.

The trade off chart/pareto front of a range of reasonable possibilities in the design space for the three reactors was presented for the MOGA II objective optimization. In the fronts, feasible region containing solutions that satisfy the constraints for both log reduction and the cost objectives has been presented. The selection of any design solution would be based on the designers' preference. The optimal designs are the best compromise between the 2 objectives.

The results obtained from sensitivity analysis for the two optimization applied show that changing two parameters; the first-order decay constant (k) and the temperature (T) has significant effect on the effluent faecal coliform and the entire pond configuration. This would help designers and engineers to make informed decision as the insights given are paramount to the design, construction and maintenance of waste stabilization ponds.

Chapter 5

Laboratory-Scale WSP post-modeling results and verification of the Optimized models

5.1 Introduction

This chapter presents results of the treatment efficiency and the hydraulic performance of pre-modeled baseline reactor (published literature 70% pond-width) configurations of two-, four-, six-baffle 70% pond-width and the CFD/optimized designs from Simplex and MOGA II optimization for the laboratory-scale reactors. The experimental data of microbial (feacal coliform) and Physico-chemical (Phosphate, Chloride, Nitrate and Sulphate, PH, conductivity and Total dissolved solids) parameters in the influent and effluent are presented in Tables 5.1-5.7 and also the comparison of the reactor performance in bar charts as presented in Figures 5.1- 5.12. The in-situ pH profiles data are presented to assess the initiation of acidity or alkalinity of the wastewater which is characterized by the presence of nutrients present in the influent and effluents samples from the reactors.

The data are used to verify the CFD model with simulated effects of isothermal condition in the laboratory since it was a constant room temperature of 24^0C that was used in the simulation process. The ultimate goal of model verification is to make the model useful and to provide accurate information about the system being modeled in order to make the model to be used. The chapter also presents the CFD-predicted results of the effluents in terms of feacal coliform log kill. The CFD-predicted results are compared with the experimental data from the pre-modeled reactor configurations of two-, four-, six-baffle 70% pond-width and the CFD/optimized designs for Simplex and MOGA II laboratory-scale reactors to verify the CFD that was used in this research. The effect of reactor geometry on disinfection efficiency was assessed by comparing inactivation levels in the baseline reactor (published literature 70% pond width) and the CFD/optimized reactors. The improvement in the treatment efficiency of the baffled lab-scale reactors could be attributed to the reduction of hydraulic short-circuiting that usually diminishes pond performance.

5.2 Microbial and physico-chemical parameters

5.2.1 Feacal coliform inactivation in the reactors

It is interesting to note that the influent feacal coliform numbers of 59×10^3 per 100 ml compare well with the expected concentration of feacal coliform numbers found in raw sewage (Mara, 2004; Tchobanoglous *et al.*, 2003). Some levels of nutrient removal occur during all biological treatment processes. The main focus of these studies has been the need for a cost effective design that will not jeopardize the treatment efficiency. The laboratory-scale effluent was assessed to verify the results obtained from the CFD simulation and the CFD/optimization design configuration. The accuracy of the CFD model for feacal coliform prediction and its presentation has been based on the log units removal and it is interesting to know that the effluent quality results that was observed in the laboratory experiment are in compliance with restricted crop irrigation requirements (less than 10^5 feacal coliform per 100 ml) (WHO, 2006). All the baffled laboratory-scale reactor effluent (1×10^3 Feacal coliform per 100 ml) after the maturation reactors treatment does comply with restricted crop irrigation.

Figure 5.1 shows the combined laboratory set-up prior to modeling and CFD/optimized designs of different baffle configurations that was tested in the laboratory while Figure 5.2 shows the wastewater sampling bottles that were used during the laboratory experiment. Table 5.1 shows that the observed effluent quality do not give predictions that are identical to measurements of the hydraulic performance of all the CFD models. There are many possibilities why this may be the case, for example the discrepancies could be attributed to rate constant (k), temperature (T), the depth of flow (d), the wastewater density (ρ) that was used in the simulation and some other environmental factors. It would be good to recall here that the model is sensitive to temperature (T) and rate constant (k). This was demonstrated in the sensitivity analysis that was performed. These factors are thought to affect significantly experiments that are conducted on waste stabilization ponds (Shilton and Harrison, 2003a; Fredrick and Lloyd, 1996; Brissaud *et al.*, 2000, 2003). It is interesting to know that the temperature of the wastewater ranges between 22^0C and 24^0C in the laboratory.

Figure 5.1 Different tested laboratory-scale reactor configuration

Figure 5.2 Laboratory effluents sampling during the experiment

Table 5.1 Experimental and CFD feacal coliform log-kill in the reactors

Parameter Feacal coliform (log-kill) at k = 9.12 and 4.56 s^{-1}	**Two-baffle 70% pond-width transverse lab-scale reactor**	**Six-baffle 70% pond-width lab-scale reactor**	**Four-baffle 70% pond-width transverse lab-scale reactor**	**Four-baffle 70% pond-width longitudinal lab-scale reactor**	**Simplex optimized design**	**MOGA-II optimized design**
Anaerobic reactors						
Observed experimental Log-units removal	**0.28**	**0.32**	**0.34**	**0.31**	**0.35**	**0.30**
CFD log- units removal at k = 9.12 s^{-1}	0.53	0.66	0.61	0.51	0.61	0.60
Simplex log-units removal at k = 4.56 s^{-1}	**-**	**-**	**-**	**-**	**0.32**	**-**
MOGA II log units removal at k = 4.56 s^{-1}	-	-	-	-	-	0.30
Facultative reactors						
Observed experimental Log-units removal	**0.71**	**0.74**	**0.81**	**0.75**	**0.81**	**0.76**
CFD log- units removal at k = 9.12 s^{-1}	1.18	1.85	1.49	1.50	1.62	1.51
Simplex log-units removal at k = 4.56 s^{-1}	**-**	**-**	**-**	**-**	**1.01**	**-**
MOGA II log units removal at k = 4.56 s^{-1}	-	-	-	-	-	0.71
Maturation reactors						
Observed experimental Log-units removal	**0.70**	**0.70**	**0.60**	**0.70**	**0.60**	**0.70**
CFD log- units removal at k = 9.12 s^{-1}	1.32	2.14	1.84	1.81	1.81	1.90
Simplex log-units removal at k = 4.56 s^{-1}	**-**	**-**	**-**	**-**	**1.08**	**-**
MOGA II log units removal at k = 4.56 s^{-1}	-	-	-	-	-	0.81

In the anaerobic reactors, the observed feacal log unit's removal was close with the ones that were realized when the k value was at the lower range during the sensitivity analysis that was performed in this study. Similar results were found for the facultative and the maturation reactors. This suggests that the k value may perhaps be closer to the ones at lower range. When comparing the lower rate constant results as presented in Table 4.21 and 4.27 for Simplex and MOGA II sensitivity analysis with the experimental results, these give a satisfactory order of faecal coliform number reduction based on effluent quality. This suggests that the low end of the sensitivity analysis performed on the rate constant is actually quite good. It is interesting to note that the observed effluent feacal coliform numbers in the lab-scale reactors and the CFD have the same pattern of feacal reduction though the values are different. There is possibility that the lower temperature or change in the rate constant could be a clear reason for the difference.

For the anaerobic reactors, Simplex optimal design had the highest feacal log kill (0.35) as compared to other configurations followed by the four-baffle and Six-baffle 70% pond-width lab-scale reactors with 0.34 and 0.32 respectively. The MOGA II design configuration performed better than the Two-baffle 70% pond-width transverse lab-scale reactor (0.30 vs 0.28). For the facultative reactors, the same value of log unit removal was observed for Four-baffle 70% pond-width transverse lab-scale reactor and the Simplex optimized design (0.81 log unit). The MOGA II optimized design performed better than the remaining configurations with a log unit of 0.76 as compared to 0.71, 0.74 and 0.75 in the two- and six-baffle transverse and Four-baffle 70% pond-width longitudinal lab-scale reactors respectively. Surprisingly, there is no difference in the treatment performance of the observed maturation reactor effluents feacal log kill (0.70) except for the four-baffle 70% pond-width transverse lab-scale reactor and Simplex optimized design which has feacal log kill of 0.60. However, with the sensitivity analysis performed at low rate constant k, the possible range of log reduction results was predicted reasonably good.

These results suggest that the treatment efficiency of the lab-scale reactors improved significantly when baffles of various configurations were installed in the reactors to increase the length to width ratio. Plots of flow patterns from CFD/optimized designs indicate that the hydraulic flow patterns are significantly different in all the reactors. In addition, the CFD flow pattern show that there is a low degree of hydraulic short-circuiting in baffled waste stabilization ponds compared with those for unbaffled waste stabilization ponds due to the satisfactory mixing of wastewater that occurs in the baffle compartments. Based on the low rate constant data, it can be recommended that the CFD has estimated satisfactorily the predicted effluent feacal coliform numbers in all the set of reactors that was examined considering the influent feacal coliform numbers and the environmental conditions in the set of lab-scale reactors.

Pearson *et al.* (1995) and Buchauer (2006) argue that the treatment efficiency of ponds with high aspect ratios (length to width > 6:1) is not significantly different from those with low aspect ratios (2- 3:1). It was concluded that the treatment efficiency of ponds could not be significantly improved by modifying the pond geometry through the use of baffles. It was also argued that Marais' (1974) equation is adequate and could be used to model the decay of feacal coliform in waste stabilization ponds with complex geometry. The results presented in this work show that the findings of (Pearson *et al.* 1995; Buchauer, 2006) are not conclusive and could be misleading, as their research was limited to unbaffled ponds. Mangelson and Watters (1972); Shilton, (2001); Abbas *et al.* (2006) observed that the treatment efficiency of ponds was considerably improved when baffles are installed in the pond. It is interesting to note that results from other researchers (Kilan and Ogunrombi, 1984; Muttamara and Puetpailboon, 1996, 1997; Sperling *et al*. 2002; Zanotelli *et al.* 2002) showed that baffles improve the treatment performance of waste stabilization ponds.

Simulation of design parameters that vary over the residence time of the reactor could require more computational resources and it would require more experimental data to validate the CFD. Statistical significance to difference between the WSP geometries tested would have been interesting data to know. However, due to time and resources, the experiments were only limited to testing certain parameters and the author also had to balance the cost of resources to perform the experiments. The goal of this study was aimed at developing a CFD-based optimization model as an innovative tool for the design of waste stabilization ponds that incorporates the effects of different pond footprint and number, length, and placement of baffles on the WSP treatment performance. In addition, development of complex sub-model in CFD is not realistic for the design and performance assessments of waste stabilization ponds. This suggests that the CFD with simulated effects of isothermal condition is satisfactory in predicting the hydraulic and treatment performance of the baffled lab-scale WSPs.

The CFD has been well verified by the experimental data from the laboratory-scale WSPs. Simulations of feacal coliform removal, and tracer experiment were estimated satisfactorily in the CFD. The model results of the feacal coliform log removal at the low end rate constant are not significantly off the track as compared to the experimental results that was carried out in the laboratory-scale WSPs. The significance of the CFD verification and validation is that regulators and designers can use CFD confidently both as a reactor model and as a hydraulic tool to assessing realistically the treatment efficiency of baffled WSPs.

5.2.2 Phosphate removal

The experimental data of Phosphate in the influent showed a constant value of 2.05mg/l which signifies that the value is greater than 2.05mg/l because the analysis was done at low range phosphate dose. Phosphate is a common form of phosphorus in wastewater. 30 to 50 percent of phosphorus is from sanitary wastes, while the remaining 50 to 70 percent is from phosphate builders used in detergents. Phosphorus has been identified as the most important rate-limiting factor for algal growth in freshwater systems, and its removal from these wastewaters has therefore become increasingly important. The best way of increasing phosphorus removal in WSP is to increase the number of maturation ponds, so that progressively more and more phosphorus becomes immobilized in the sediments. From a well functioning two-pond system, 70% mass removal of total phosphorus may be expected (Hamzeh and Ponce, 2007).

Table 5.2 Experimental data of Phosphate removal for all the reactor configurations

Parameter Phosphate(mg/l)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle 70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent Phosphate conc.	> 2.05	> 2.05	> 2.05	> 2.05	> 2.05	> 2.05
Anaerobic Effluent Phosphate conc.	> 2.05	> 2.05	> 2.05	> 2.05	> 2.05	> 2.05
Facultative Effluent Phosphate conc.	> 2.05	> 2.05	1.94	> 2.05	> 2.05	1.94
Maturation Effluent Phosphate conc.	0.32	0.20	0.49	0.87	0.24	0.31

Table 5.2 shows a significant variation of phosphate removal in the range 0.20 - 0.87 mg/l that was observed in the maturation reactor effluent. The concentration of phosphate in the influent was relatively higher compared to that of the effluent (>2.5mg/l vs 0.20 - 0.87mg/l). Table 5.2 shows that the removal of phosphate in the six-baffle lab-scale maturation reactor and two-baffle simplex optimized design lab-scale reactor is generally lower compared to that of the four-baffled transverse and longitudinal lab-scale reactors. In the overall, one could conclude that the final effluent from the six-baffle 70% pond-width lab-scale reactor performed best and followed by the simplex and MOGA II optimized design configurations respectively. The removal efficiency ranges from 58% in the four-baffle longitudinal maturation reactor to 90% in the six-baffle maturation reactor. The efficiency of the optimized design for the simplex and MOGA II designs are 88% and 80% respectively. The performance of the optimized design can be said to be better compare to the four-baffle 70% pond-width transverse and longitudinal baffle arrangements. Overall, these results suggest that the treatment efficiency of the CFD/optimized reactors improved significantly than when the 70% pond-width baffles were installed in the pond.

5.2.3 Chloride removal

Chloride concentration in wastewater is an important parameter with respect to assessing the suitability of wastewater for agricultural purpose.

Table 5.3 Experimental data of Chloride removal for all the reactor configurations

Parameter Chloride (mg/l)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle 70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent Chloride conc.	485	485	490	490	480	480
Anaerobic Effluent Chloride conc.	430	300	450	480	470	420
Facultative Effluent Chloride conc.	420	280	430	460	350	300
Maturation Effluent Chloride conc.	320	210	380	430	80	270

Table 5.3 shows that more chloride reduction was achieved in the effluent from the maturation reactors with 320 mg/l for two-baffle, 210 mg/l for six-baffle, 380 mg/1 for four-baffle transverse 70% pond-width lab-scale reactor and 430 mg/l for four-baffle longitudinal reactor while the effluent of the CFD/optimized designs shows a significant reduction after the maturation to 80 mg/l for the six-baffle 69% pond-width and 270mg/l for the two-baffle maturation reactor. In all, one could say that the Simplex optimized design performed best compared to others while the MOGA II optimized design performed better than others except for the six baffle 70% pond-width configuration.

5.2.4 Nitrate removal

Nitrate is a common form of nitrogen. Nitrogen in municipal wastewater results from human excreta and ground garbage which is particularly from food processing and its average concentration in domestic wastewater is about 35mg/l. Nitrates test is used as a measure of the nutrients present and the degree of decomposition in the wastewater. The removal of nitrate depends on its concentration, detention time, and the available organic matter (Crites and Tchobanoglous, 1998). Table 5.4 shows all reactors nitrate concentration.

Table 5.4 Experimental data of Nitrate removal for all the reactor configurations

Parameter Nitrate (mg/l)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle 70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent Nitrate conc.	36.33	36.33	37.21	37.21	36.77	36.77
Anaerobic Effluent Nitrate conc.	35.93	35.88	36.81	34.73	34.11	35.35
Facultative Effluent Nitrate conc.	30.88	29.73	31.23	30.48	28.48	30.26
Maturation Effluent Nitrate conc.	16.83	19.94	16.39	19.05	20.11	21.35

Table 5.4 shows that the removal of nitrate in the sets of two-baffle and four-baffle lab-scale reactors is generally higher compared to that of the Simplex and MOGA II CFD/optimized reactor designs respectively. It is interesting to note that the removal of nitrate in the four-baffle transverse lab-scale reactor performed best as compared with other configurations. The Simplex and MOGA II optimized design configuration gave a considerable overall

nitrate reduction of 45% and 42% as compared to 54%, 45%, 56% and 49% nitrate reduction in the two-, six-, and four-baffle- transverse and longitudinal lab-scale reactors respectively. Mara et al (1992) emphasized that excess nitrate in wastewater are removed in maturation ponds. The result achieved confirms this statement because the maturation reactors in the entire configuration gave better efficiency as compared to the anaerobic and the facultative reactors respectively. Surprisingly, there is no much difference in the treatment performance of the anaerobic reactors in removing nitrate. It is also interesting to note that the removal of nitrate in the four baffle longitudinal lab-scale reactor did not perform better than two- and four-baffle lab-scale reactors.

5.2.5 Sulphate removal

Sulphate test is used to assess the potential for the formation of odors and to assess the treatability of the wastewater. Table 5.4 shows influent and effluent sulphate concentration in all the reactors.

Table 5.5 Experimental data of Sulphate removal for the reactor configurations

Parameter Sulphate(mg/l)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle 70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent Sulphate conc.	43	43	43	43	42	42
Anaerobic Effluent Sulphate conc.	40	41	41	40	38	40
Facultative Effluent Sulphate conc.	36	35	36	34	37	37
Maturation Sulphate conc.	30	29	30	33	33	34

The study showed higher overall sulphate reduction efficiency of 33% by the six-baffle 70% pond width transverse lab-scale reactor as compared to the other configurations which have

30%, 30%, 23%, 21% and 19%, Two-baffle, Four-baffle70% pond width transverse lab-scale reactors, Four-baffle 70% pond width longitudinal lab-scale pond and the CFD/optimized simplex and MOGA II designs respectively. More of the sulphate removal was achieved in the facultative reactors before the final stage maturation reactors. The comparison of the sulphate removal with other pollutant removal (Chloride, Nitrate and Phosphate) is presented in Tables 5.6-5.8 for anaerobic, facultative and maturation laboratory scale WSPs respectively. Sulphate reduction in all cases increased through the set up. This may be due to the fact that the mixed culture in the wastewater contains other bacteria which can efficiently hydrolyze given carbon source into simpler compounds.

Table 5.6 Experimental data of nutrient removal for the anaerobic reactor configurations

	2 and 6 baffles anaerobic reactors influent	2-baffle 70% pond-width transverse anaerobic reactor effluent	6-baffle 70% pond-width transverse anaerobic reactor effluent	**4 baffles Transverse and Longitudinal Anaerobic reactors influent**	4-baffle 70% pond-width transverse anaerobic reactor effluent	4-baffle 70% pond-width longitudinal anaerobic reactor effluent	**3 and 2 baffles anaerobic reactors influent**	3-baffle 49% pond-width Simplex optimized Anaerobic reactor design effluent	2-baffle 58% pond-width MOGA-II optimized anaerobic reactor design effluent
P	2.05	2.05	2.05	2.05	2.05	2.05	2.05	2.05	2.05
C	485	430	300	490	450	480	480	470	420
N	36.33	35.93	35.88	37.21	36.81	34.73	36.77	34.11	35.35
S	43	40	31	43	41	40	42	38	40

Table 5.7 Experimental data of nutrient removal for the facultative reactor configurations

	2-baffle 70% pond-width facultative reactor influent	2-baffle 70% pond-width transverse facultative reactor effluent	**6-baffle 70% pond-width facultative reactor influent**	6-baffle 70% pond-width transverse facultative reactor effluent	**4-baffle 70% pond-width facultative reactor influent**	4-baffle 70% pond-width transverse facultative reactor effluent	**4-baffle 70% pond-width longitudinal facultative reactor influent**	4-baffle 70% pond-width longitudinal facultative reactor effluent	**2-baffle 83% pond-width longitudinal simplex design facultative reactor influent**	Simplex optimized facultative reactor design effluent	**2-baffle 69% pond-width transverse MOGA II design facultative reactor influent**	MOGA-II optimized facultative reactor design effluent
P	2.05	2.05	2.05	2.05	2.05	1.94	2.05	2.05	2.05	2.05	2.05	1.94
C	430	420	300	280	450	430	480	460	470	350	420	300
N	35.93	30.88	35.88	29.73	36.81	31.23	34.73	30.48	34.11	28.48	35.35	30.26
S	40	36	31	29	41	36	40	34	38	37	40	37

Table 5.8 Experimental data of Sulphate removal for the reactor configurations

	2-baffle 70% pond-width maturation reactor influent	2-baffle 70% pond-width transverse maturation reactor effluent	**6-baffle 70% pond-width maturation reactor influent**	6-baffle 70% pond-width transverse maturation reactor effluent	**4-baffle 70% pond-width maturation reactor influent**	4-baffle 70% pond-width transverse maturation reactor effluent	**4-baffle 70% pond-width longitudinal maturation reactor influent**	4-baffle 70% pond-width longitudinal maturation reactor effluent	**6-baffle 69% pond-width transverse simplex design maturation reactor influent**	Simplex optimized maturation reactor design	**2-baffle 81% pond-width longitudinal MOGA II design maturation reactor influent**	MOGA-II optimized maturation reactor design
P	2.05	0.32	2.05	0.2	1.94	0.49	2.05	0.87	2.05	0.24	1.94	0.31
C	420	320	280	210	430	380	460	430	350	80	300	270
N	30.88	16.83	29.73	19.94	31.23	16.4	30.48	19.05	28.48	20.11	30.26	21.35
S	36	30	31	29	36	30	34	33	37	33	37	34

5.2.6 pH variation

Many chemical and biological reactions in wastewater treatment are pH dependent and they rely on pH control. Table 5.9 shows that as the wastewater moves through the reactors in series, the pH of the effluent from the reactors increases. The pH falls within a range of 7.43-7.50 before treatment and 7.79-7.89 after treatment. The influent to the "El Gallo" Wastewater Treatment Plant showed a significant variability in pH (from 6.8 to 8.9) (The city of San Diego, 2003). This further confirms the findings of Zehnder et al (1982) that states the optimum pH range for all methanogenic bacteria is between 6 and 8. The same observation was reported by Van Haandel and Lettinga (1994) and Droste (1997) that acidogenic populations are less sensitive to pH variations and acid fermentation will predominate over methanogenic fermentation. Table 5.9 shows the experimental data of PH variation that were observed in all the reactor configurations. It is also interesting to note that the observed data of pH in all the reactor configurations compare well with the expected pH that is usually found in literature (Pearson, 1987; Parhad and Rao, 1974). High pH (higher than 8) causes metal ions to precipitate and allows pond purification processes to occur normally. Therefore, raising the pH of the pond to about 8 will cause most of the sulphide- formed by the bacterial reduction of sulphate- to exist as the odourless bisulphide ions. Under these conditions, the release of the malodorous hydrogen sulphide gas (H_2S) will be reduced significantly.

Table 5.9 Experimental data of PH variation for all the reactor configurations

Parameter pH	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent pH	7.43	7.43	7.47	7.47	7.50	7.50
Anaerobic Effluent pH	7.47	7.69	7.54	7.53	7.62	7.58
Facultative Effluent pH	7.76	7.81	7.86	7.81	7.80	7.84
Maturation Effluent pH	7.79	7.88	7.89	7.85	7.82	7.89

5.2.7 Total dissolved solids removal

Table 5.10 shows that the experimental data of total dissolved solids in the influent was in the range of 340-343 ppm while the effluent concentration was in the range of 273-315 ppm respectively.

Table 5.10 Experimental data of TDS removal for all the reactor configurations

Parameter TDS (ppm)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent TDS	342	342	340	340	343	343
Anaerobic Effluent TDS	335	340	334	333	338	338
Facultative Effluent TDS	317	326	313	309	315	318
Maturation Effluent TDS	315	302	273	285	275	293

The average concentration of total dissolved solids in the influent could be taken as 341ppm. The CFD/optimized configuration performed well as compared to others configurations with TDS effluent of 275 ppm in Simplex and 293 ppm MOGA II optimized designs. It is interesting to know that the four-baffle 70% pond width transverse performed creditably well as compared to the two- and six-baffle transverse arrangement and the four baffle longitudinal arrangement.

5.2.8 Conductivity variation

Conductivity is a well recognized and indispensible parameter of state-of-the art wastewater analysis. Continuous measuring systems are employed to monitor the salt load at the influent and effluent of wastewater treatment facilities. Conductivity is a measure of the dissolved solids present in a sample. The results are consistent with those commonly found in wastewaters and the information is valuable for the design, upgrade or modification of wastewater treatment systems. At present, the conductivity of wastewater is one of the important parameters used to determine the suitability of wastewater for irrigation (Crites and Tchobanoglous, 1998). The salinity of treated wastewater to be used for irrigation is estimated by measuring its conductivity (Metcalf and Eddy 2003). Table 5.11 shows the conductivity experimental data for all the reactor configurations

Table 5.11 Conductivity experimental data for all the reactor configurations

Parameter Conductivity (μS)	Two-baffle 70% pond width transverse lab-scale reactor	Six-baffle 70% pond width lab-scale reactor	Four-baffle70% pond width transverse lab-scale reactor	Four-baffle 70% pond width longitudinal lab-scale reactor	Simplex optimized design	MOGA-II optimized design
Influent conductivity	700	700	690	690	697	697
Anaerobic Effluent conductivity	684	693	682	662	689	691
Facultative Effluent conductivity	657	656	627	642	653	639
Maturation Effluent conductivity	632	625	557	581	562	598

The anaerobic reactor had minimal reduction of conductivity values as compared to the performance of the maturation reactors which gave a significant reduction in the conductivity of the effluents. The four-baffle 70% pond width transverse lab-scale reactor series gave the optimal conductivity reduction for the entire configuration (557 μS) followed by the simplex optimized design with a conductivity value of 562 μS. It is evident that the Simplex design is reasonably predicting more of pollutant reduction in the entire set of laboratory-scale WSPs.

5.2.9 Summary of laboratory experimentation

The chapter has shown the treatment performance for the three reactors in series. The Simplex optimal design for the anaerobic reactors had the highest feacal log kill as compared to other configurations followed by the four-baffle and Six-baffle 70% pond-width lab-scale reactors. For the facultative reactors, four-baffle 70% pond-width transverse lab-scale reactor and Simplex optimized design gave better performance over other reactors with the same value of log unit removal. However, MOGA II optimized design performed better than the remaining configurations as compared to the two- and six-baffle transverse and Four-baffle 70% pond-width longitudinal lab-scale reactors respectively. The experimental data for the maturation reactors showed that the treatment performance was not significantly different from each other.

The result of the effluent quality tested for Nitrate, sulphate, chloride, phosphate, PH, conductivity and TDS showed that the WSPs performed well with different levels of pollutant removal. The result realized for phosphate reduction further confirms the suggestions of previous researchers that the best way of increasing phosphorus removal in WSP is to increase the number of maturation ponds, so that progressively more and more phosphorus becomes immobilized in the sediments. The Simplex and MOGA II optimized design configuration gave a considerable overall nitrate reduction as compared to the nitrate reduction in the two-, six-, and four-baffle- transverse and longitudinal lab-scale reactors respectively. The pH in the influent and effluent falls within a range of 7.43-7.50 before treatment and 7.79-7.89 after treatment while for TDS, the CFD/optimized configuration performed creditably well as compared to other configurations with effluent of 275 (ppm) in Simplex and 293 (ppm) MOGA II optimized designs.

The CFD has been verified by the experimental data from all the reactors. Simulations of feacal coliform removal and tracer experiment were estimated satisfactorily in the CFD. The model results of the feacal coliform log removal at the low end rate constant are not significantly off the track as compared to the experimental results that was carried out in the entire set of reactors in the laboratory-scale WSPs. The significance of the CFD verification and validation is that regulators and designers can use CFD confidently both as a reactor model and as a hydraulic tool to assessing realistically the treatment efficiency of baffled WSPs. The results of this research will directly impact the possible design decisions that wastewater treatment engineers must make related to WSPs design.

Chapter 6

DISCUSSION OF RESULTS

6.1 Experimental results of Laboratory-scale waste stabilization ponds in series

The experimental data presented in chapter (Table 5.1-5.11) shows the results of analysis that were tested for wastewater quality parameters (Faecal coliform, Chloride, Sulphate, Nitrate, Phosphate, pH, TDS and Conductivity) during the laboratory experiments based on the limited resources available as at the time the research was carried out. The decay of faecal coliform was decided to be used as a parameter in the simulation process because it shows the greatest resistance to treatment and inactivation among all other parameters tested. The effect of its inactivation is an excellent indication of other pollutant removal. This was chosen to be simulated because it is a reliable and commonly used indicator of effluent quality. It is also convenient from a computational point of view as its decay follows the first-order kinetic theory.

The mathematical model used in (Chapter 3) characterizing the residence time distribution curve provided a good fit. It could be recalled that the tracer experiment to verify the mean hydraulic retention time which is an indicator parameter of the hydraulic efficiency of the laboratory-scale pond was attempted but because the tracer chemical used did not perform as expected, it was therefore expedient to use mathematical models to characterize the RTD for the experimental data. The complete mixed tank (N-tanks) in series model that was adopted shows that more than average data points were able to match closely with the CFD model. The use of baffles as a physical design intervention to improve the hydraulic efficiency of laboratory-scale waste stabilization pond and to address hydraulic problems has been explored.

6.2 Hydraulic efficiency of CFD model laboratory-scale waste stabilization ponds in series.

(a) First Order Rate Constant

The CFD model has been verified using experimental data of the effluent faecal coliform numbers and other nutrients removal that were observed in the six sets of laboratory-scale WSP models. Different values of first order rate constant removal have been developed by various researchers to predict the removal of faecal coliform in waste stabilization ponds. Banda (2007) expressed that the derivation of the first-order rate constant has been based on the assumption of complete mix and the plug flow patterns. Table 6.1 presents the reported values of $k_{B(20)}$ and ϕ in the first-order rate constant removal equation $\left(k = k_{B(20)}\phi^{(T-20)}\right)$.

Table 6.1 Reported values of $k_{B(20)}$ and ϕ in the first-order rate constant removal equation of faecal coliform in waste stabilization ponds.

Source	$K_{B(20)}\left(day^{-1}\right)$	ϕ
Klock (1971)	1.1	1.07
Marais (1974)	2.6	1.19
Skerry and Parker (1979)	1.5	1.06
Arceivala (1981)	1.2	1.19
Mills et al. (1992)	0.7	1.17
Yanez (1993)	1.1	1.07
Mayo (1995)	1.9	1.08
Mara et al. (2001)	2.6	1.15
Banda et al. (2006)	4.55	1.19

Source: Banda (2007)

It can be deduced from the table that there is a wide variation in the first-order rate constant removal of faecal coliform especially the $k_{B(20)}$ values in a range of 0.7- 4.55.

The first-order rate constant removal has a great improvement in the treatment and hydraulic efficiency that were observed in baffled ponds. Banda (2007) established that a correlation coefficient of 0.8267 (R^2 = 0.8267) was realized and this predicted reasonably accurate the observed effluent faecal coliform counts in baffled ponds when used in the source term function (Figure 3.14). The author stressed that the first-order rate constants removal (0.7-2.6) were developed using unbaffled WSP that were characterized with poor hydraulic and treatment efficiency. Banda et al. (2006) equation was used in this research to simulate the faecal coliform removal in baffled reactors based on the recommendation from the expert in the field and the fact that it has been found to be satisfactory in predicting the faecal coliform removal in a baffled WSPs.

(b) Treatment Efficiency

The results of the treatment efficiency of a series of models that were tested using evenly spaced, 50%, 60%, 70%, 80% and 90% width baffles for the transverse arrangement and 60%, 70%, 80% and 90% width baffles for the longitudinal arrangement were presented in Table 4.1. These data are recalled here as a reference for discussion that follows. It can be noted that in the overall configurations, the longitudinal 2-baffle 90% pond-width gave the best performance. This is contrary to the findings of Watters et al (1973) who expressed that baffles of 70% width gave superior performance compared to the 50% and 90% pond width baffles. The author discovered that increasing the baffle width to 90% was found to give a lower hydraulic efficiency than was seen with the 70% pond width baffles. The longitudinal arrangement seems promising but could be costly in terms of construction. However, with cost in mind one would consider other options that have a closer performance.

A quick estimate of the log-removal in the 2-baffle, 90% baffle length longitudinal reactors in series shows that anaerobic, facultative and maturation reactor achieved a FC log-removal of 0.686, 1.910 and 2.599 making a total of 5.44 while the conventional 6-baffle 70% pond with transverse reactor in series achieved a log removal of 0.66, 1.85 and 2.15 respectively making cumulative log removal of 4.66.

Other configurations whose reactor treatment efficiency are estimated among others were the 2-baffle 60% pond length longitudinal reactors in series, 4-baffle 70% pond width and 6-baffle 90% pond width transverse reactor in series. The results indicated a FC log-removal of 0.47, 1.30 and 1.53 for the 2-baffle 60% pond length in the longitudinal configuration, 0.61, 1.49 and 1.85 for the 4-baffle 70% pond width and 0.66, 1.73 and 1.97 for the anaerobic, facultative and maturation reactors in the transverse arrangement respectively. It is also interesting to note that as the length of the baffle was increased in the anaerobic, facultative and maturation reactors from 60% to 90% in the longitudinal arrangement, there was improvement in the performance of the reactors.

In the transverse arrangement, it was discovered that the six baffle cases proved to be more efficient than the four baffle cases. This was attributed to channeling effects. The diagrams of the hydraulic flow patterns in the CFD model of baffled WSP (Chapter 4) show that the four and six 70% pond-width baffles encouraged the mixing of wastewater in the baffle compartments and this increased the length of flow path from the inlet to outlet. When the results were compared against the longitudinal baffle arrangements, it was found that the 2-baffle with 80% and 90% and 4-baffle with 80% and 90% pond-width configurations were more efficient.

The overall comparison of the longitudinal baffling and the transverse baffling gave similar results. The field scale interpretation of this is an increase of the hydraulic retention time that could lead to improvement in the hydraulic performance of the baffled ponds. Wood (1997) recognized that the design and performance of a WSP depends substantially on an adequate description of its mixing characteristics and estimation of its biological degradation rate constant. It is with this notion that (Pena, 2002) stated that the limitation in quantifying these factors seemed to explain the disagreement between expected and current operating performance data on full-scale WSP.

The average inlet velocities of 4.27×10^{-3} m/s, 6.17×10^{-3} m/s and 6.94×10^{-3} m/s for anaerobic, facultative and maturation ponds respectively were used in achieving the hydraulic retention times of the ponds which were given as 0.165 day, 0.563 day and 0.683 day which correspond to a flow rate of 0.12 m^3 per day.

(c) Faecal Coliform Removal

Banda (2007), Shilton and Harrison (2003a), Sperling et al. (2002), Muttamara and Puetpailboon (1996,1997) and Kilani and Ogunrombi (1984) all observed higher removal of faecal coliform, COD, BOD_5 and helminth eggs in waste stabilization ponds that were fitted with baffles of various configurations than in those that were not baffled. The results of the treatment efficiency of the laboratory-scale WSP presented in this research agree satisfactorily with the findings of these researchers.

The predicted faecal coliform count at the outlets for the unbaffled reactors was 5.53×10^7per 100 ml, 1.64×10^7per 100 ml and 4.4×10^6per 100 ml for anaerobic, facultative and maturation respectively. The effluent concentration of the anaerobic was used as the influent concentration into the facultative pond and the same was done for the maturation pond. This gave a total log removal of 1.35 of faecal coliform at the end of the maturation pond treatment. In tropical climate regions where unbaffled WSP are normally designed at short retention times, the effects of the hydraulic short-circuiting can be significant and the treatment performance of WSP could be diminished. The use of baffles could be desirable in this situation to reduce the occurrence of hydraulic short-circuiting that might deteriorate treatment efficiency and the performance of unbaffled WSPs. More so, less land would be required for the construction of baffled WSP. Shilton and Mara (2004) identified that baffled WSP may need about 50% of the total land required for the construction of unbaffled WSP because of the improved treatment efficiency achieved with baffled system.

(d) Summary of CFD model results

The close correlation/pattern of the predicted effluent faecal coliform in the CFD model and the experimental data for the six set of reactors that were examined in the laboratory is reasonable in simulating the removal of faecal coliform. It can be concluded that the satisfactory prediction of the CFD model of effluent faecal coliform in the anaerobic, facultative and maturation reactors suggests that it can be confidently used to design and evaluate the treatment efficiency of waste stabilization pond with simulated effect of baffle installation and different inlet and outlet configurations.

6.3 Optimization of laboratory-scale ponds by Simplex and MOGA II algorithms

The CFD model has reasonably predicted the faecal coliform inactivation within all the reactors. Simplex single objective optimization solver and MOGA II for the multi-objective optimization problem have been employed. As presented, it can be seen that the results obtained from the optimization process predicts well all the hydraulic properties in the reactors. While the results have provided useful optimal cost design analysis relative to treatment performance, they are very sensitive to the parameters that were used in the model. It is now understandable that the model has the capability to predict reasonably precisely the physics, chemistry, and biological processes occurring in the WSPs.

MOGA-II generates the optimal trade-offs among the two objectives specified in the workflow. The solutions from the multi-objective optimization model provide the choices for the decision maker to optimize the capital available, which is a constraint in most cases. In another case where the goal is to obtain a solution to meet the specified faecal coliform target, the solution should at least produce the specified pollutant reduction; therefore the optimized solutions that cost the least for achieving the particular wastewater quality goals were selected from several alternative configurations in the optimization output file.

The bold prints in Simplex designs for both transverse and longitudinal arrangements describe the overall optimal designs in Tables 4.11 and 4.14. These are the optimal cost of materials for the laboratory-scale construction of WSP. The cost objective has been limited to material cost only because it is a standard quantity from which other cost values can be estimated. It could be recalled here that it is the cost per unit area of plate used in constructing the laboratory-scale reactor and other cost such as labor, construction and maintenance can easily be evaluated.

Table 4.9 shows that a 3-baffle 49% pond-width anaerobic and a 6-baffle 69% pond-width maturation pond are the optimal solutions. Shilton and Harrison (2003a, 2003b) using a 2D model found that a four baffled facultative pond with 70% pond-width baffles was superior to six baffled facultative pond in removing faecal coliform with counts of 390 per 100 ml for the four-baffle facultative pond and 570 per 100 ml for the six-baffled facultative pond. The work presented in this research has found that six-baffled facultative pond provides superior removal of faecal coliform by 0.59 orders of magnitude than the four-baffled. The six-baffled facultative pond creates an approximate plug flow pattern that performs better than the four-baffled facultative pond, which is due to the constant flow channel width (flow channel width in baffle compartments and at baffle openings).

The maximum FC removal values indicate that the inclusion of more baffles and increasing the baffle length has significant contribution to the quality of effluent from the reactor. It would be the decision of the engineer to choose between the trade-offs depending on the objective to be achieved. However, with cost in mind for construction and maintenance, the optimized results would to be considered on a large-scale. The range of area ratio is within the limits for which a better performance can be achieved in waste stabilization pond as discussed and presented in literature.

Sensitivity analysis has been performed to determine the influence of the first order rate constant (k) and the temperature (T) parameters on the optimized solution. The optimization model developed can be easily extended to other wastewater treatment system designs provided all the variables, constraints and objective functions are known. The model gives a range of options available for pollutant reduction and their corresponding construction material costs. One notable finding of the sensitivity analysis is that the characteristics configuration of the entire reactor changed with different values in k and T. This conceptual output provides general ideas for changing pond configuration due to the change in the input parameters.

6.4 Summary of discussion

The results of CFD mathematical model for faecal coliform decay exhibited a good fit to the results obtained from the laboratory scale model. In practice, some variation must be expected when applying CFD to field ponds due to the great number of physical variations in the field situation that are simply not practical to measure and incorporate into CFD model. There are a number of factors that, typically, are either simplified or neglected in a pragmatic modeling approach.

There is need to address the purpose for which the modeling was undertaken and why the model may not fully represent the incongruities in the field system. The CFD model was based on steady flow rate condition, while in the field pond; the flow rate can vary continuously both diurnally and with rainfall. Wind shear was not included in this simulation and the wastewater density in the CFD model was taken to be uniform throughout the pond which may not be so on the field due to sludge material build up which could increase the inlet velocity higher than that which was allowed for in the CFD model and the effect of thermo-stratification.

The result of the effluent quality tested for Nitrate, sulphate, chloride, phosphate, PH, conductivity and TDS showed that the WSPs performed well with different levels of pollutant removal. The Simplex and MOGA II optimized design configuration gave a considerable overall nitrate reduction as compared to the nitrate reduction in the two-, six-, and four-baffle- transverse and longitudinal lab-scale reactors respectively. The pH in the influent and effluent falls within a range of 7.43-7.50 before treatment and 7.79-7.89 after treatment while for TDS, the CFD/optimized configuration performed creditably well as compared to other configurations with effluent of 275 (ppm) in Simplex and 293 (ppm) MOGA II optimized designs. With the results achieved from the CFD model and the laboratory scale model, one can now have confidence in the ability of CFD to simulate the effect of baffles arrangement and varying pond sizes to determine the hydraulic performance and treatment efficiency of waste stabilization ponds.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

7.1 Conclusions

A methodology and modeling technique for the analysis of hydraulic efficiency and optimization of waste stabilization pond has been presented. The following conclusions can be drawn based on the results of the CFD model, optimization process and experimental data from the operation of the laboratory-scale waste stabilization ponds:

1. The study has helped in determining the per capita demand and the total water supply for Covenant University. This further helps in revealing the wastewater flow rate for the design of a field-scale prototype WSP system.

2. Characteristics of a CFD-based model that incorporates the effects of different foot print size, baffle configuration and length on the treatment performance of the WSP have been explored. The use of CFD has proven to be a powerful tool to facilitate the design and evaluation of new and existing WSP systems as it gives direct insight into flow pattern that could not be done through experimental tracer study.

3. The weakness of applying CFD to the real life pond in the field would be that there are a number of incongruities (such as sludge deposits, variable climate conditions and variable flow rates) that in practical engineering applications cannot be accurately defined and incorporated into the model. The realization of this practical limitation of CFD modeling is a very important consideration for practicing engineers applying CFD result to full-scale pond design.

4. Rigorous assessment of WSPs that account for cost in addition to hydrodynamics and treatment efficiency utilizing CFD coupled with an optimization program to efficiently optimize the selection of the best WSP configuration has been performed. This is a novel approach.

5. Sensitivity analysis performed in this work has helped in knowing the nature and extent of the variation of the optimum pond configuration with changes in the first order rate constant (k) and the temperature (T) parameters. This conceptual output provides general ideas for changing pond configuration due to the changed inputs.

6. The results of faecal coliform concentration at the reactor outlet showed that the conventional 70% pond-width baffles is not consistently the best pond configuration as previously reported in the literature.

7. The outcome of this research would help designers and engineers to make informed decision as the insights given is paramount to the design, construction and maintenance of waste stabilization ponds.

7.2 Contributions to knowledge

The study has revealed the following contributions to knowledge among others things:

1. The optimum number and spacing of baffles of different configuration for achieving the highest microbial removal efficiency has been determined.

2. The work has also helped in knowing the nature and extent of the variation of the optimum pond configuration with changes in the first order rate constant (k) and the temperature (T) parameters. This conceptual output provides general ideas for changing pond configuration due to the changed inputs.

3. The application of COMSOL Multiphysics coupled with an optimization program to optimize the selection of the best WSP configuration based on cost and treatment efficiency has helped further to elucidate the nature of residence time and velocity distribution which are important parameters in waste stabilization ponds.

7.3 Recommendation for further work

In view of the experimental data obtained from the laboratory-scale waste stabilization ponds used in this research and the CFD modeling, further work is required as follows:

1. CFD models of nutrient removal in waste stabilization ponds should be investigated to assess the removal of other wastewater pollutant. The model should consider the various processes such as denitrification, sedimentation, evaporation and algae uptake that are responsible for the removal of nutrients in waste stabilization ponds.

2. A study of the microbiological, biophysical and biochemical processes that take place in waste stabilization pond would be useful to finding out similarities and differences of treatment processes within the reactors. This kind of study will provide useful information about the threshold limits for the steady operation of any reactor configuration in waste stabilization ponds.

3. The verification of the laboratory-scale results reported in this work could be confirmed by a full-scale construction of waste stabilization pond in Covenant University. This type of experience would provide valuable insight on the real investment and operational costs as well as the real requirements of operation and management for this technology. The data obtained from this would allow the sustainability of the technology to be assessed under real condition. This would serve as guide to physical planning units of institutions in Nigeria for the design of treatment systems that will enhance environmental quality and protection.

4. Wind speed effects and its prevailing direction should be investigated to assess the treatment efficiency of full-scale waste stabilization ponds with emphasis on the geometric design of pond, inlet and outlet structures and sludge accumulation as reported in literature.

5. Further work should be done on testing of this model using actual field data. This may require modification of the cost equation.

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Appendix A

A1. COMSOL Multiphysics Model M-file for Transverse baffle anaerobic reactor

```
% COMSOL Multiphysics Model M-file
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)

flclear fem

% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;

% Geometry
g1=rect2('.950','.320','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'.950','.01'},'rot','0');
g4=rect2('.003','.224','base','corner','pos',{'.156','0'},'rot','0');
garr=geomarrayr(g4,0.156,0,5,1);
[g99,g5]=deal(garr{:});
g5=move(g5,[0,0.096]);

g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g8},'ns',{'g8'},'sf','g8','edge','none');
g10=geomdel(g9);

% Analyzed geometry
clear s
s.objs={g10};
s.name={'CO1'};
s.tags={'g10'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','4.27e-3[m/s]', ...
  'k','9.124[1/d]'};
```

```
% Initialize mesh
fem.mesh=meshinit(fem, ...
                'hauto',5, ...
                'hmaxsub',[1,0.05*.320]);

% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0,1,0};
bnd.type = {'N0','C','Nc'};
```

```
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.D = 'eta/rho';
equ.v = 'v';
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;

% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;
% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);

% Solve problem
fem.sol=femstatic(fem, ...
        'blocksize',1000, ...
        'solcomp',{'v','u','p','coliform'}, ...
        'outcomp',{'v','u','p','coliform'}, ...
        'maxiter',50, ...
        'hnlin','on', ...
        'linsolver','pardiso', ...
        'uscale','none');

% Save current fem structure for restart purposes
```

```
fem0=fem;

I1=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
        'dl',[1], ...
        'edim',1)/0.005;

I2=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
        'dl',[32], ...
        'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

A2. COMSOL Multiphysics Model M-file for longitudinal baffle anaerobic reactor

```
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)
flclear fem
% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;
% Geometry
g1=rect2('.950','.320','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'-.01','.305'},'rot','0');
g4=rect2('.665','.003','base','corner','pos',{'.0','.0508'},'rot','0');
garr=geomarrayr(g4,0,0.0508,1,5);
[g99,g5]=deal(garr{:});
g5=move(g5,[0.285,0]);
g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g9},'ns',{'g9'},'sf','g9','edge','none');
g10=geomdel(g9);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','4.27e-3[m/s]', ...
  'k','9.124[1/d]'};

% Geometry

% Analyzed geometry
clear s
s.objs={g11};
s.name={'CO1'};
s.tags={'g11'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);

% Initialize mesh
fem.mesh=meshinit(fem, ...
              'hauto',5, ...
              'hmaxsub',[1,.05*.320]);
```

```
% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0, 1, 0};
bnd.type = {'N0','C','Nc'};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.D = 'eta/rho';
equ.v = 'v';
```

```
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;

% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;

% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);
% Solve problem
fem.sol=femstatic(fem, ...
            'blocksize',1000, ...
            'solcomp',{'v','u','p','coliform'}, ...
            'outcomp',{'v','u','p','coliform'}, ...
            'maxiter',50, ...
            'hnlin','on', ...
            'linsolver','pardiso', ...
            'uscale','none');


% Save current fem structure for restart purposes
fem0=fem;

I1=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
```

```
    'dl',[1], ...
    'edim',1)/0.005;

I2=postint(fem,'coliform', ...
    'unit','mol/m^2', ...
    'dl',[32], ...
    'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

A3. COMSOL Multiphysics Model M-file for Transverse baffle facultative reactor

```
% COMSOL Multiphysics Model M-file
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)

flclear fem

% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;

% Geometry
g1=rect2('2.1','.7','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'2.1','.01'},'rot','0');
g4=rect2('.003','.49','base','corner','pos',{'.348','0'},'rot','0');
garr=geomarrayr(g4,.348,0,5,1);
[g99,g5]=deal(garr{:});
g5=move(g5,[0,0.21]);

g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g8},'ns',{'g8'},'sf','g8','edge','none');
g10=geomdel(g9);

% Analyzed geometry
clear s
s.objs={g10};
s.name={'CO1'};
s.tags={'g10'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','6.17e-3[m/s]', ...
  'k','9.124[1/d]'};

% Initialize mesh
fem.mesh=meshinit(fem, ...
```

```
          'hauto',5, ...
          'hmaxsub',[1,.05*.70]);

% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0,1,0};
bnd.type = {'N0','C','Nc'};
bnd.ind = [1];
appl.bnd = bnd;
```

```
clear equ
equ.D = 'eta/rho';
equ.v = 'v';
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;
% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;
% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);

% Solve problem
fem.sol=femstatic(fem, ...
            'blocksize',1000, ...
            'solcomp',{'v','u','p','coliform'}, ...
            'outcomp',{'v','u','p','coliform'}, ...
            'ntol',1.0E-4, ...
            'maxiter',50, ...
            'hnlin','on', ...
            'linsolver','pardiso', ...
            'uscale','none');

% Save current fem structure for restart purposes
fem0=fem;
```

```
I1=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
        'dl',[1], ...
        'edim',1)/0.005;

I2=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
        'dl',[32], ...
        'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

A4. COMSOL Multiphysics Model M-file for longitudinal baffle facultative reactor

```
% COMSOL Multiphysics Model M-file
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)

flclear fem

% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;

% Geometry
g1=rect2('2.1','.7','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'-.01','.685'},'rot','0');
g4=rect2('1.47','.003','base','corner','pos',{'0','.114'},'rot','0');
garr=geomarrayr(g4,0,0.114,1,5);
[g99,g5]=deal(garr{:});
g6=move(g6,[0.63,0]);
g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g9},'ns',{'g9'},'sf','g9','edge','none');
g10=geomdel(g9);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','6.17e-3[m/s]', ...
  'k','9.124[1/d]'};

% Geometry

% Analyzed geometry
clear s
s.objs={g11};
s.name={'CO1'};
s.tags={'g11'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);
```

```
% Initialize mesh
fem.mesh=meshinit(fem, ...
                'hauto',5, ...
                'hmaxsub',[1,.07*.70]);

% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0,1,0};
bnd.type = {'N0','C','Nc'};
```

```
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.D = 'eta/rho';
equ.v = 'v';
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;

% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;
% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);

% Solve problem
fem.sol=femstatic(fem, ...
        'blocksize',1000, ...
        'solcomp',{'v','u','p','coliform'}, ...
        'outcomp',{'v','u','p','coliform'}, ...
        'maxiter',50, ...
        'hnlin','on', ...
        'linsolver','pardiso', ...
        'uscale','none');

% Save current fem structure for restart purposes
```

```
fem0=fem;

I1=postint(fem,'coliform', ...
	'unit','mol/m^2', ...
	'dl',[1], ...
	'edim',1)/0.005;

I2=postint(fem,'coliform', ...
	'unit','mol/m^2', ...
	'dl',[32], ...
	'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

A5. COMSOL Multiphysics Model M-file for Transverse Maturation reactor

```
% COMSOL Multiphysics Model M-file
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)

flclear fem

% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;

% Geometry
g1=rect2('2.470','.830','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'2.47','.01'},'rot','0');
g4=rect2('.003','.581','base','corner','pos',{'.409','0'},'rot','0');
garr=geomarrayr(g4,.409,0,5,1);
[g99,g5]=deal(garr{:});
g5=move(g5,[0,0.249]);

g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g8},'ns',{'g8'},'sf','g8','edge','none');
g10=geomdel(g9);
% Analyzed geometry
clear s
s.objs={g10};
s.name={'CO1'};
s.tags={'g10'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','6.94e-3[m/s]', ...
  'k','9.124[1/d]'};

% Initialize mesh
fem.mesh=meshinit(fem, ...
          'hauto',5, ...
```

```
        'hmaxsub',[1,.05*.830]);

% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0,1,0};
bnd.type = {'N0','C','Nc'};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
```

```
equ.D = 'eta/rho';
equ.v = 'v';
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;

% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet Velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;
% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);
% Solve problem
fem.sol=femstatic(fem, ...
        'blocksize',1000, ...
        'solcomp',{'v','u','p','coliform'}, ...
        'outcomp',{'v','u','p','coliform'}, ...
        'ntol',1.0E-4, ...
        'maxiter',50, ...
        'hnlin','on', ...
        'linsolver','pardiso', ...
        'uscale','none');

% Save current fem structure for restart purposes
fem0=fem;

I1=postint(fem,'coliform', ...
```

```
          'unit','mol/m^2', ...
          'dl',[1], ...
          'edim',1)/0.005;

I2=postint(fem,'coliform', ...
          'unit','mol/m^2', ...
          'dl',[32], ...
          'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

A6. COMSOL Multiphysics Model M-file for longitudinal Maturation reactor

```
% Generated by COMSOL 3.4 (COMSOL 3.4.0.248, $Date: 2007/10/10 16:07:51 $)
flclear fem

% COMSOL version
clear vrsn
vrsn.name = 'COMSOL 3.4';
vrsn.ext = '';
vrsn.major = 0;
vrsn.build = 248;
vrsn.rcs = '$Name:  $';
vrsn.date = '$Date: 2007/10/10 16:07:51 $';
fem.version = vrsn;

% Geometry
g1=rect2('2.470','.830','base','corner','pos',{'0','0'},'rot','0');
g2=rect2('.01','.005','base','corner','pos',{'-.01','.01'},'rot','0');
g3=rect2('.01','.005','base','corner','pos',{'-.01','0.815'},'rot','0');
g4=rect2('1.729','.003','base','corner','pos',{'0','.136'},'rot','0');
garr=geomarrayr(g4,0,.136,1,5);
[g99,g5]=deal(garr{:});
g6=move(g6,[0.741,0]);
g8=geomcomp({g1,g2,g3},'ns',{'g1','g2','g3'},'sf','g1+g2+g3','edge','none');
g9=geomcomp({g9},'ns',{'g9'},'sf','g9','edge','none');
g10=geomdel(g9);

% Constants
fem.const = {'rho','997.38[kg/m^3]', ...
  'eta','9.11e-4[Pa*s]', ...
  'u_in','6.94e-3[m/s]', ...
  'k','9.124[1/d]'};

% Geometry

% Analyzed geometry
clear s
s.objs={g11};
s.name={'CO1'};
s.tags={'g11'};

fem.draw=struct('s',s);
fem.geom=geomcsg(fem);

% Initialize mesh
fem.mesh=meshinit(fem, ...
          'hauto',5, ...
```

```
                 'hmaxsub',[1,.07*.830]);

% (Default values are not included)

% Application mode 1
clear appl
appl.mode.class = 'NavierStokes';
appl.module = 'CHEM';
appl.gporder = {4,2};
appl.cporder = {2,1};
appl.assignsuffix = '_chns';
clear prop
prop.analysis='static';
appl.prop = prop;
clear bnd
bnd.type = {'walltype','inlet','outlet'};
bnd.U0in = {1,'u_in',1};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
equ.eta = 'eta';
equ.gporder = {{1;1;2}};
equ.rho = 'rho';
equ.cporder = {{1;1;2}};
equ.ind = [1];
appl.equ = equ;
fem.appl{1} = appl;

% Application mode 2
clear appl
appl.mode.class = 'ConvDiff';
appl.dim = {'coliform'};
appl.module = 'CHEM';
appl.assignsuffix = '_chcd';
clear prop
prop.analysis='static';
clear weakconstr
weakconstr.value = 'off';
weakconstr.dim = {'lm8'};
prop.weakconstr = weakconstr;
appl.prop = prop;
clear bnd
bnd.c0 = {0,1,0};
bnd.type = {'N0','C','Nc'};
bnd.ind = [1];
appl.bnd = bnd;
clear equ
```

```
equ.D = 'eta/rho';
equ.v = 'v';
equ.u = 'u';
equ.R = 'fst';
equ.ind = [1];
appl.equ = equ;
fem.appl{2} = appl;
fem.frame = {'ref'};
fem.border = 1;
clear units;
units.basesystem = 'SI';
fem.units = units;

% Global expressions
fem.globalexpr = {'fst','-k*coliform'};

% Descriptions
clear descr
descr.const= {'eta','Viscosity','u_in','Inlet Velocity','rho','Density'};
fem.descr = descr;

% ODE Settings
clear ode
clear units;
units.basesystem = 'SI';
ode.units = units;
fem.ode=ode;
% Multiphysics
fem=multiphysics(fem);

% Extend mesh
fem.xmesh=meshextend(fem);

% Solve problem
fem.sol=femstatic(fem, ...
        'blocksize',1000, ...
        'solcomp',{'v','u','p','coliform'}, ...
        'outcomp',{'v','u','p','coliform'}, ...
        'maxiter',50, ...
        'hnlin','on', ...
        'linsolver','pardiso', ...
        'uscale','none');

% Save current fem structure for restart purposes
fem0=fem;

I1=postint(fem,'coliform', ...
```

```
        'unit','mol/m^2', ...
        'dl',[1], ...
        'edim',1)/0.005;

I2=postint(fem,'coliform', ...
        'unit','mol/m^2', ...
        'dl',[32], ...
        'edim',1)/0.005;

save output.dat -ASCII -DOUBLE
```

Appendix B

B1. Transverse baffle arrangement scripting

Geometry 1

```
Area=volume/depth
w=sqrt(Area/ratio)
l=ratio*w

if (floor (baffle_number/2)*2 == baffle_number)
{
Wm = w-0.015
//even baffles number
}
Else
{
Wm = 0.01
//odd baffles number
}



baffle_length=baffle_length_ratio*w
v_in=Q/ (width_c*depth)
m_b=w-baffle_length
of_out=w-0.01-width_c
dist = l/(baffle_number+1)-0.0015
```

Geometry 2

```
gb=zeros(baffle_number-1)
gd=zeros(baffle_number+1)
for (i=0;i<(baffle_number-1);i++) {
  gb[i]=i+5
}

for (i=0;i<baffle_number+1;i++) {
  gd[i]=baffle_number+4-i
}

gs1=gd[0]
gs2=gs1+1
gs3=gs2+1

numbc=12+4*baffle_number
bc=ones(numbc)
bc[0]=2
```

```
if (floor(baffle_number/2)*2 ==baffle_number)
{
bc[numbc-1]=3
//even baffles number
}
Else
{

bc[3]
numbc=4
//odd baffles number
}




string_move="
snum=m_b.toString()
for (i=0;i<floor(baffle_number/2); i=i+1)
{
  is=(5+i*2).toString()
  string_move=string_move+'g'+is+'=move (g'+is+',[0,'+snum+']); '
print(i)
}
```

B2. Longitudinal baffle arrangement scripting

Geometry 1

```
Area = volume/depth
w = sqrt (Area/ratio)
l = ratio*w

if (floor(baffle_number/2)*2 == baffle_number)
{
lm=1
//even baffles number
}
Else
{
lm= -0.01
//odd baffles number
}


baffle_length = baffle_length_ratio*l
v_in = Q/(width_c*depth)
m_b = l-baffle_length
of_out = w-0.01-width_c
dist = w/(buffle_number+1)-0.0015
```

Geometry 2

```
gb=zeros(baffle_number-1)
gd=zeros(baffle_number+1)
for (i=0;i<(baffle_number-1);i++) {
  gb[i]=i+5
 }

for (i=0;i<baffle_number+1;i++) {
  gd[i]=baffle_number+4-i
}

gs1=gd[0]
gs2=gs1+1
gs3=gs2+1

numbc=12+4*baffle_number
bc=ones(numbc)
bc[0]=2
if (floor (baffle_number/2)*2 == baffle_number)
{
bc[numbc-1]=3
//even baffles number
```

```
}
else
{

bc[3]=3
numbc=4
//odd baffles number
}



string_move=''
snum=m_b.toString()
for (i=0;i<floor(baffle_number/2;i=i+1)
{
  is=(5+i*2).toString()
  string_move=string_move+'g'+is+'=move(g'+is+',['+snum+',0]);'
print (i)
}
```