PREDICTION OF FLOW FIELD AND OXYGEN UTILIZATION RATE (OUR) IN ORBAL BIOLOGICAL SYSTEM (OBS) USING CFD

NOOR AIDA BINTI SAAD

UNIVERSITI SAINS MALAYSIA

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by

NOOR AIDA BINTI SAAD

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LIST OF ABBREVIATIONS

ASMs	Activated Sludge Models
BNR	Biological Nutrient Removal
BOD	Biochemical Oxigen Demand
CFD	Computational Fluid Dynamics
DO	Dissolved Oxigen
EA	Extended Aeration
EIA	Environmental Impact Assessment
HRT	Hydraulic Residence Times
IIM	Insoluble Inorganic Matter
IOM	Insoluble Organic Matter
IWK	Indah Water Konsortium
LCA	Life Cycle Assessment
MLVSS	Mixed Liquor Volatile Suspended Solids
OAT	One factor at a time
OBS	Orbal Biological System
OD	Oxidation Ditch
OUR	Oxigen Utilization Rate
P	Power Consumption
PA	Performance Assessment
PI	Performance Indicator
RAS	Return Activated Sludge

RBC	Rotating Biological Contactors
SA	Sensivity Analyses
SBR	Sequencing Batch Reactors
SCADA	Supervisory Control and Data Acquisition
SIM	Soluble Inorganic Matter
SND	Simultaneous Nitrification and Denitrification
SOM	Soluble Organic Matter
SPAN	Suruhanjaya Perkhidmatan Air negara
STP	Sewage Treatment Plant
TKN	Total Kjeldahl Nitrogen
WWTPs	Wastewater Treatment Plants
3D	Three Dimensional

LIST OF SYMBOLS

Re	Reynolds number
ρ	The fluid density ($water = 998.2\ kg/m^3$)
u	The velocity magnitude
d	The hydraulic diameter of the OBS
μ	The dynamic viscosity of the water ($1 \times 10^{-3} kg/ms$).
$k - \varepsilon$	k-epsilon
k-ω	k-omega
OUR	Oxygen utilization rate or also referred as oxygen requirement (kg/d)
Q	Influent flow (m^3/d)
S_e	Effluent BOD_5 concentration (soluble BOD) (g/ m^3)
S_o	Influent BOD_5 concentration (total BOD) (g/m^3)
Xv	Mixed Liquor Volatile Suspended Solids, MLVSS (g/ m^3)
V	Tank volume (m^3)
ΔTKN	$TKN_{influent}$- $TKN_{effluent}$ (g/m^3)
TKN	Total kjeldahl nitrogen, equal to organic nitrogen and the ammonia nitrogen
ΔTN	$TN_{influent}$-$TN_{effluent}$ (g/m^3)
TN	Total nitrogen
a	1.46-1.42Y
b	$1.42f_bK_d$
Y	Yield coefficient (gVSS produced per gBOD

	removed) ($gX_{v\,/g}BOD_5$)
f_b	Biodegrable fraction of MLVSS generated in a system subjected to a sludge age Θ_c (X_b/X_v)
$f_{b'}$	Biodegrable fraction of the VSS immediately after its generation in the system, that is, with Θ_c =0. This value is typically equal to 0.8 (80%)
Θ_c	Sludge sge (d)
K_d	Endogenous respiration coefficient (d^{-1})
P	Power consumption in kW
η_p	In-process oxygenation efficiency for mechanical aeration system in kg O_2/ kW.hr
a	$1.46 - 1.42\text{Y}$
1.46	Conversion factor (BOD_u/BOD_5)
Y	Yield coefficient (gVSS produced per gBOD removed) ($gX_{v\,/g}BOD_5$)
b	$1.42 f_b K_d$
θ_c	Sludge age (d)
K_d	Endogenous respiration coefficient (d^{-1})
m_{pq}	The mass transfer from phase p to phase q
m_{qp}	The mass transfer from phase q to phase p
α_q	The volume fraction of the q^{th} phase

RAMALAN MEDAN ALIRAN DAN KADAR PENGGUNAAN OKSIGEN DALAM SISTEM BIOLOGI ORBAL MENGGUNAKAN CFD

ABSTRAK

Sistem Orbal Biologi (OBS), yang merupakan salah satu daripada parit pengoksidaan yang telah diubah suai, telah direka untuk memberikan proses olahan biologi yang optimum. Oleh kerana reka bentuk strukturnya yang berpotensi besar untuk menggalakkan proses olahan yang optimum, adalah sangat penting untuk kita mengkaji keseluruhan sistem operasinya. Hasrat ini selari dengan permintaan semasa yang mahukan sistem olahan air sisa yang lebih baik yang bukan sahaja fokus kepada pencapaian standard kualiti efluen yang ditentukan, tetapi juga mengambil kira aspek-aspek lain seperti ekonomi dan alam sekitar. Oleh sebab ini, kajian ini dijalankan untuk memahami sistem olahan OBS yang kompleks ini. Model proses awal dan model *Computational Fluid Dynamics* (CFD) telah digunakan untuk mendapatkan gambaran yang lebih baik tentang OBS. Model CFD 3 dimensi, 2 fasa yang bersaluran terbuka telah dibangunkan untuk mengkaji mekanisma operasi OBS. Dalam kajian ini, model CFD telah digunakan untuk mewakili OBS untuk mengatasi beberapa limitasi model proses awal. Keputusan simulasi telah digunakan untuk mengkaji corak aliran di seluruh parit, taburan halaju pada kedalaman yang berbeza, agihan halaju untuk keadaan operasi yang berbeza, pecahan isipadu udara dan air di seluruh parit, pengedaran tekanan merentasi saluran dan masa tahanan hidraulik. Keputusan model CFD juga digunakan untuk mengira kadar aliran isipadu yang lebih khusus, nilai Oxygen Utilization Rate(OUR) dalam parit dan penggunaan kuasa yang berkaitan dengan nilai-nilai OUR. Berdasarkan kepada kajian kesan kepelbagaian

operasi shaf ke atas prestasi OBS, didapati bahawa bilangan shaf yang beroperasi mempengaruhi jumlah keseluruhan penggunaan kuasa dalam sistem ini. Penutupan kombinasi shaf 2(saluran luar) dan shaf 6 (saluran tengah dan dalam) menyumbang kepada penggunaan kuasa yang kurang tetapi masih mempunyai nila purata OUR yang sama (26,483 kg/d) seperti kes asal (26,594 kg/d). Keputusan menunjukkan, walaupun shaf ini ditutup, tetapi tindakbalas biologi di dalam parit masih berterusan seperti tindakbalas biologi bila kesemua shaf beroperasi. Ramalan penggunaan kuasa di masa hadapan yang dikaitkan dengan peningkatan nilai OUR, yang mungkin berlaku kerana standard kualiti efluen yang lebih ketat, juga telah dibentangkan dan dibincangkan dalam kajian ini. Berdasarkan keputusan, saluran paling luar merekodkan penggunaan oksigen tertinggi secara keseluruhannya, iaitu sebanyak 79% berbanding dengan saluran tengah dengan hanya 9% dan diikuti oleh saluran dalaman dengan 12%. Pada masa akan datang, jika standard kualiti efluen yang lebih ketat dikenakan, kadar penggunaan oksigen juga akan pasti meningkat. Jika kadar penggunaan oksigen pada masa kini iaitu 21109 kg/d meningkat kepada 30%, ia akan menyebabkan peningkatan penggunaan elektrik sebanyak 3,241 MW/tahun (hampir RM 1,092,284 setahun). Melalui kajian ini, peningkatan pemahaman tentang bagaimana simulasi seperti ini dapat digunakan dalam pemodelan proses olahan air sisa berjaya dilaksanakan. Ia amat berguna dalam proses permodelan proses olahan air sisa ddi masa akan datang.

PREDICTION OF FLOW FIELD AND OXYGEN UTILIZATION RATE (OUR) IN ORBAL BIOLOGICAL SYSTEM (OBS) USING CFD

ABSTRACT

Orbal Biological System (OBS), which is one of the modified oxidation ditches, has been designed to provide an optimized approach of the biological treatment process. Due to the structural design of the OBS that has great potential to promote optimal treatment processes, it is very crucial to study its operating system. This intention is in line with the current demand on wastewater treatment system, which not only focusing on the achievement of specified effluent quality standards, but also taking into consideration other aspects such as economic and environmental. Because of this reason, the study was conducted to understand the complicated process of OBS. The preliminary process model and Computational Fluid Dynamics (CFD) model were used to obtain a better picture of the OBS. Three-dimensional, two-phase and an open channel CFD-based model was developed to study the mechanism of OBS. In the study, the CFD model was used to represent the OBS to overcome some of the limitations of the preliminary process model. The simulation results were used to study the flow pattern across the ditch, the velocity distribution at different depths, velocity distribution for different operating conditions, the volume fraction of air and water in the ditch, the pressure distribution across the channels and hydraulic residence time. The results of CFD model were used to calculate more specific volumetric flow rate, the Oxygen Utilization Rate (OUR) of the ditch and the power consumption associated with the values of OUR. Based on the study on the effect of different operating shafts on the performance of OBS, it was found that turning off the combination of shaft 2 (outer channel) and shaft 6 (middle and inner channel) was contributing to less power consumption but still having almost the same average OUR value (26,483 kg/d) like the real case (26,594 kg/d). The results show that although these shafts are turned off, but the biological reactions inside the ditch still occur as the biological reactions when all the shafts are operated. Forecast future power consumption associated with the OUR values, which may occur due to effluent quality standards that are more stringent, was also presented and discussed in this study. Based on the results, the outermost channels recorded the highest overall oxygen consumption, which is 79% compared with the middle channel with only 9%, followed by inner channel with 12%. In the future, if more stringent effluent quality standards imposed, the rate of oxygen consumption will definitely increase. If the rate of oxygen consumption at present, namely 21,109 kg/d increased to 30%, it will lead to increased electricity use by 3,241 MW/year (nearly RM 1,092,284/year). The research provides a better understanding on how this simulation tool will be able to be applied within wastewater process modelling. It can considerably contribute to the further expansion of wastewater treatment process models.

CHAPTER ONE

INTRODUCTION

Biological treatment system is recognized as one of the most significant process of the overall wastewater treatment. At the same time, it is also identified as the system that consumes the highest portion of the overall power of the treatment plant. In consideration of the fact that its structural and operational system has a huge potential to promote an optimal treatment process, it is crucial to study its overall operational system using more effective, efficient and economical approach.

This intention is parallel with the current demand for an improved wastewater treatment system which not only focussing on achieving the required effluent quality standards, but also taking into consideration other sustainability aspects such as sustainable power consumption. Computational Fluid Dynamics (CFD) is becoming available to further characterize and further optimize bioreactor hydraulic flow patterns, thereby allowing improved designs to be developed. First part of the chapter gives an overview on the trends and challenges of wastewater treatment system. It discusses some issues in the wastewater treatment sector such as accomplishing the effluent water quality standards, optimizing the power consumption and the application of more advanced modelling approach. The problems, rationale, objectives, limitations and contributions of the research are also being discussed in the chapter. The last section of the chapter presents the thesis outline which consists of five chapters in overall.

1.1 Trend and challenges in wastewater treatment system

Since the world's first sanitary sewerage system was completed in 1843 (Qasim, 1999), various types of wastewater treatment technologies have been introduced and established, with the aim to improve the wastewater treatment systems. In the past, to about 1980s (Metcalf & Eddy, 2013), the objective of development of wastewater treatment systems was entirely based on the environmental perspective. At that time, wastewater treatment facilities were designed, operated and maintained with only one aim, to accomplish the effluent standards written in the set of laws. Wastewater treatment system is considered as one of the energy-intensive technologies, which since the past decade had received serious consideration with regard to energy consumption (Owen, 1982).

Wastewater treatment has been viewed as an energy intensive industry, and energy consumption is always the main operation cost of wastewater systems (Li et al., 2016). Lately, there has been a lot of discussion about the energy implications of the increasingly stringent wastewater discharge standards(Yong et al., 2016, Nowak et al., 2015, Wang et al., 2015, Morera et al., 2016, Morera et al., 2015, Angelakoglou and Gaidajis, 2015, Zang et al., 2015). These days, the design and operation of wastewater treatment plants (WWTPs) are progressively focused on improving the efficiency of energy consumption and reducing the treatment costs. Water and wastewater systems are significant energy consumers(Zhang et al., 2015), where recently, the successful design and reliable operation of treatment plant is one of the foremost challenges in the wastewater sector (Yong et al., 2016). Energy consumption is referring to primary energy equivalents, which have been defined as the energy content extracted from the earth (Wang et al., 2015). The common energy forms consumed by WWTPs, either direct or indirect consumption, are electricity, fuel, oil, diesel fuel, natural gas, coal, and sometimes propane. Direct energy consumption is referring to on-site energy use; whereas indirect consumption includes the energy utilization in the production of materials used at the plant during construction, in producing construction materials, and for transportation services (Wang et al., 2015).

WWTPs play a significant role within the municipal water cycle in protecting receiving waters from untreated discharges(Morera et al., 2016). The sewage treatment is proved to cause an increasing impact on the national energy consumption and economic growth (Zhang et al., 2015). The aims of WWTPs are extended, not only to accomplish the effluent quality criteria, but also to apply the holistic approach of wastewater treatment that incorporating the sustainability principles to become more sustainable and comprehensive in terms of environmental, social and economic perspectives. In order to achieve a state of environmental sustainability, it is crucial to keep improving the current practice of wastewater treatment system. Lately, one of the most critical challenges in wastewater treatment system is to have a successful design and dependable treatment operations, which may ensure high treatment efficiencies to comply with the more stringent effluent quality standards and at the same time keeping the operation and maintenance cost as low as possible(Karpinska and Bridgeman, 2016).

In order to fulfil more stringent requirements of the effluent standards, a variety of strategies have been proposed to improve the wastewater treatment system. However, whether or not these strategies are sustainable in terms of energy consumption and other related issues, such as pollutants emission are still uncertain and difficult to be determined. The decision makers in this field should realize that maximizing the effluent quality of WWTPs does not guarantee the achievement of sustainability of other resources such as energy (Xie et al., 2011, Wang et al., 2015).

'Win-win situation' approach should be applied in this case to the point where both, effluent quality and energy consumption are optimized. In order to reach the optimum point, there is no other way other than conducting researches against a real subject matter, which is the WWTP itself. The differences are only the approaches or the strategies involved in the research. Some of those strategies have been comprehensively evaluated either by practical tests or by computer simulations. Practical test is normally time-consuming and costly. Furthermore, it is frequently incompetent of taking into account all possible changes in process. In contrast, computer simulation proposes a useful and more efficient means to assess and justify the strategies that may facilitate the enhancement of treatment efficiency and reduction of energy input(Xie et al., 2011, Karpinska and Bridgeman, 2016).

WWTPs are central to water-energy interactions as they consume energy to remove pollutants and thus reduce human grey water footprint on the natural water environment (Gu et al., 2016). In view of the fact that the energy cost is rising rapidly in the present day economy, and that prices of certain forms of energy are increasing more rapidly than the prices of other goods and services, it is important to ensure that the energy intensive facilities such as WWTPs are being designed, operated and monitored effectively and efficiently. As a result of many factors, the proportion of wastewater treatment plant operating budgets allocated to energy has been increasing at an alarming rate. As mentioned by Zhang et al. (2015), in recent years, high energy consumption in WWTPs has received a serious attention by wastewater treatment sector..

Wastewater treatment used about one percent of the national electricity in Sweden, 3-4% in U.S. and about 20% in German (Zhang et al., 2015). Most of the energy consumed for wastewater treatment is directed at just a few basic process objectives; the major one being stabilization or removal of organic matters. As the key to wastewater treatment, biochemical treatment process consumed 50-70% of total energy cost in wastewater treatment (Li et al., 2016). Thus, many research on energy-saving in WWTPs have been widely carried out (Zhang et al., 2015).

Energy consumption represents a significant part of the operational costs of WWTP (Panepinto et al., 2016) but, with an appropriate approach, the limitation may be overcome. In order to improve the existing wastewater treatment facilities in terms of its treatment costs, great emphasis should be given to plant operation and maintenance to optimize the treatment costs. Since energy has become a major cost in wastewater plant operations, energy conservation has turn out to be an important issue to wastewater industry so that the unfavourable impacts of WWTPs should not exceed the benefits of remediation (Zang et al., 2015).

High energy consumption of wastewater treatment system is mostly due to the biological treatment process particularly the aeration and mixing process(Wu et al., 2012). Aeration is an important unit operation in a large number of aerobic wastewater treatment processes (Sperling and Chernicharo, 2005c). Aeration process is well known as a highly energy-intensive step in the wastewater treatment process because of the operation of its aeration devices. There are varieties of aeration devices, which also known as aerators involved in aeration process. Aerators are needed in WWTPs to transfer oxygen from the air to the wastewater being treated. The oxygen is consumed by the microorganisms to oxidize organic and inorganic wastes. Aeration devices are also been used to provide turbulence to keep particulate matter suspended in the water.

1.2 Research Problem and Rationale

Improving effluent quality standards has become one of the main issues in wastewater industry. One of the reasons is because of a broader range of constituents in the wastewater due to more variety living activities. Sustainability concept requires various types of wastewater technologies to comply with more stringent effluent standards. However, the motivation of improving the wastewater quality principles may vary among environment-related government agencies due to the countries capacity. The capacities in terms of financial, knowledge, technical and institutional are the factors affect the way that wastewater quality standards can be improved. It is ineffective to define more stringent effluent quality standards if it is causing inefficient and unsustainable usage of other resources. It should be realized that naturally, the environment be able to deal with a certain level of contamination. It is inefficient to minimize the concentration of the pollutants in the discharges if the significant costs applied to other resources (i.e. monetary and energy), it is enough to optimize it to the degree where the environment is still conserved.

In countries that are still developing and struggling to achieve a better quality of life economically, environmentally and socially, the most important thing to do currently is to increase the knowledge capacity of optimizing the wastewater treatment system. In more developed countries, which are surrounded with various facilities and capabilities, the most appropriate thing to do is to keep searching for the best approach to optimize the treatment system. Knowledge and experiences from developed and developing countries in wastewater industry can be shared to achieve more sustainable development, which fulfils the needs of today and the future.

Biological treatment system is already recognized as the most significant part of the overall wastewater treatment. However, it is also identified as the system that consumes the highest portion of the overall energy of the treatment plant. Typically about 50-70% of the total wastewater treatment's energy cost is consumed by the biological treatment system(Li et al., 2016, Yang et al., 2011, Hamilton et al., 2006). Due to this reason, more and more studies have been performed from time to time with the purpose of improving the existing as well as the future biological treatment facilities. One of the modified oxidation ditches known as the Orbal Biological System (OBS), was marketed as providing an optimised approach of the biological treatment process with more process flexibility, efficiency, reliability and simplicity. Even with all these advantages, OBS is still not been viewed as a consistent and reliable system for biological nutrient removal for the reason that a lack of understanding of its operating mechanisms. All the benefits of OBS can be achieved provided that the underlying operating systems are well understood(Daigger and Littleton, 2014).

The energy consumption of OBS operation is mainly from the power requirements for the aeration. Power consumption is associated with the amount of oxygen transferred into the channels by the aeration devices. Therefore, it is this energy utilisation of the OBS through its aeration that must be targeted for minimisation or improvement.

However, with the complicated system such as OBS, it is difficult to clearly understand the real things happened throughout the ditch. The complication of the system is worsened when some of the water in each channel flow continuously and take much longer time before they exit the system. This scenario affects the hydraulic residence time of the water inside the ditch and create inconsistency environment throughout the channels. In reality, there will be a distribution of hydraulic residence time throughout the channels. It is crucial to understand the residence time distribution inside the system before understanding the oxygen and power consumption of the system. When the oxygen needs are well understood, then it can be related to the energy consumption. Even though, the most important goal of wastewater treatment is to comply with effluent water quality regulations and discharge permits, but it does not mean that energy has to be wasted or over-consumed just to achieve that particular goal. There must be a balance between complying with effluent quality regulations and minimising energy consumption by optimising process operation wherever possible.

Sometimes, on-site measurements or field studies are limited by the obstacles of getting a full access into the treatment systems. These problems can be overcome with the development of more advanced modelling methods that able to provide more efficient studies of the hydraulic flow patterns and other parameters related to the treatment system itself. Since the past few years, due to increasing availability and accessibility of commercial and open-source software suites, CFD application has developed into robust and defined method for design, optimization and control of the biological treatment systems(Karpinska and Bridgeman, 2016). In spite of that, there are several areas in modelling practice, which still need further study.

One of the gaps in the application of CFD in biological treatment system is the modelling of unsteady flow conditions in multiphase and open channel system with the incorporation of discrete phase model. To date, discrete phase modelling is very limited even though it is really useful, since it involves long computational time and requires a large number of CPUs (Karpinska and Bridgeman, 2016). Different CFD models serve different applications (Karpinska and Bridgeman, 2016), so it is important to start developing more reliable CFD application in the context of Malaysia’s wastewater treatment systems in order to gain more confidents of its application in this field.

1.3 Specific research objectives

The research aims to develop a model that can represent a full-scale OBS that may contribute to a better understanding of the unit process and hydraulic flow patterns as well as improving the understanding of power consumption of more stringent effluent requirements. In order to accomplish the aim, the following objectives will be carried out:

1. To apply existing mathematical formulae as a preliminary process model in order to determine the relationship between oxygen utilisation rate (OUR) and biological reactions in each channel of the OBS.

2. To develop a three-dimensional, multiphase and an open channel CFD-based model to simulate the hydrodynamics in an Orbal Biological System.

3. To assess the current operational performance (related to aerations shafts) of the OBS at the Bayan Baru Sewage Treatment Plant using the developed model.

4. To evaluate the power consumption and its implications of effluent quality targets.

1.4 Scope and Limitations

Previously developed fundamental relationships describing the oxygen uptake and oxygen saving of biological processes has been used to develop a preliminary process model. Simplifying assumptions made for the preliminary process model development (e.g. constant values of HRT throughout each channel) affect the accuracy of describing the real conditions of the system. In a system such as OBS, where simultaneous biological nutrient removal takes place, it usually involves a complex hydraulic flow pattern that can cause in the cycling of mixed liquor through the different environments required for the system to function (Daigger and Littleton, 2014). In reality, there will be non-ideal mixing due to some water flows around the channel more than once. In reality, there is a distribution of residence time. A key factor that makes CFD better than the preliminary process model is its ability to visualize the distribution of the HRT throughout the channels. Moreover, properties of the flow such as velocity can be determined at any location inside the OBS.

A three dimensional CFD model has been developed to extend the preliminary process model. Unsteady state simulations or also known as transient model has been carried out to take into consideration the time variations inside the ditch. Two phase model (liquid-gas) has been developed and an open channel model has been defined to capture the actual flow process. The developed model has the ability to determine the properties of the flow such as velocity at any location inside the OBS. It also has the capacity to visualize the distribution of the HRT throughout the channels. In CFD modelling, the target spatial region was meshed into interconnected elements and the flow field was calculated for each element by solving the governing equations (Guo et al., 2013),

In general, the water entering the OBS consists of impurities in the form of suspended particulates, or other fluid impurities. This model does not incorporate these impurities. In order to consider these impurities, it comes under the domain of multiphase flow regimes, which requires the defining of more accurate description of their properties and their composition, which is a difficult scenario considering the extent of OBS and varying nature of impurities that enter the OBS channel.

1.5 Contributions to knowledge resulting from the research

This thesis provides several constructive contributions. The application of the existing mathematical formulae as a preliminary process model has provided an overview of the overall processes took place inside the ditch. It has contributed to a better study of the relationship between oxygen utilisation rate (OUR) and biological reactions in each channel of the OBS. Since the preliminary process model has incorporated a better value of the distribution of hydraulic residence times (not only the fix design value), it gives the indication of the actual oxygen consumptions throughout the system. The behaviour and property of biological processes in OBS can be deeply understood.

The developed CFD-based model has been used to study the velocity profiles of OBS. The model is very useful tool in view of the fact that the OBS is having complicated hydraulic patterns that always cause a high internal recycle flow or the cycling of mixed liquor throughout the system.

The developed CFD model has also been used to assess the current operational performance that is related to aerations shaft of the OBS and to obtain distribution of hydraulic residence times (HRT). To date, limited information is available on the use of CFD simulation for operational performance as well as on the distribution of hydraulic residence times. The CFD analysis performed in the research may enhance the use of CFD in this field. The CFD application in wastewater industries has expanded and the confidence level of using it has also increased. This research has also lead to more studies related to this field. All pros and cons of this research can be used as the guideline for a better CFD application in the related research. When further understanding is achieved in CFD-based process modelling, anyone working in this related field may possibly accomplish a better knowledge or experience to put forward possible development in the CFD models themselves.

Apart from that, the research gives some indications of the power consumption and its implications of effluent quality targets. In terms of policy implications, this research has given a broad overview of the impact of more stringent effluent standards to the overall energy consumption of the WWTPs. The authority or the policy makers related to this field may gain some benefits through this research. In addition, the policy makers in Malaysia will definitely benefit from this research as they are currently deciding to include the ammoniacal-nitrogen, total kjeldahl nitrogen (TKN) and nitrate in the new effluent standards. This research will provide some guidelines to them in terms of energy usage implications of their action.

1.6 Thesis outline

The thesis is presented in 5 chapters. This chapter as a first chapter is an introductory, providing an overview of the problems, the rationale of the research and defines the specific objectives and contributions of the work. Chapter two gives a detailed literature review covering the objectives and the scopes of the thesis. Chapter three describes the research methodology starting with the underlying principles and assumptions of the preliminary process model development. A detailed of CFD simulation including the case set up and solution are presented in this chapter. Chapter four presents the results and discussions for preliminary process model and CFD model. All useful graphics from the simulation are presented in the chapter. It also discusses on the policy implications of the model of the research findings. Chapter five gives some conclusions and recommendations based on the findings of the research. These include the recommendations for the wastewater industry and for the further research.

CHAPTER TWO

LITERATURE REVIEW

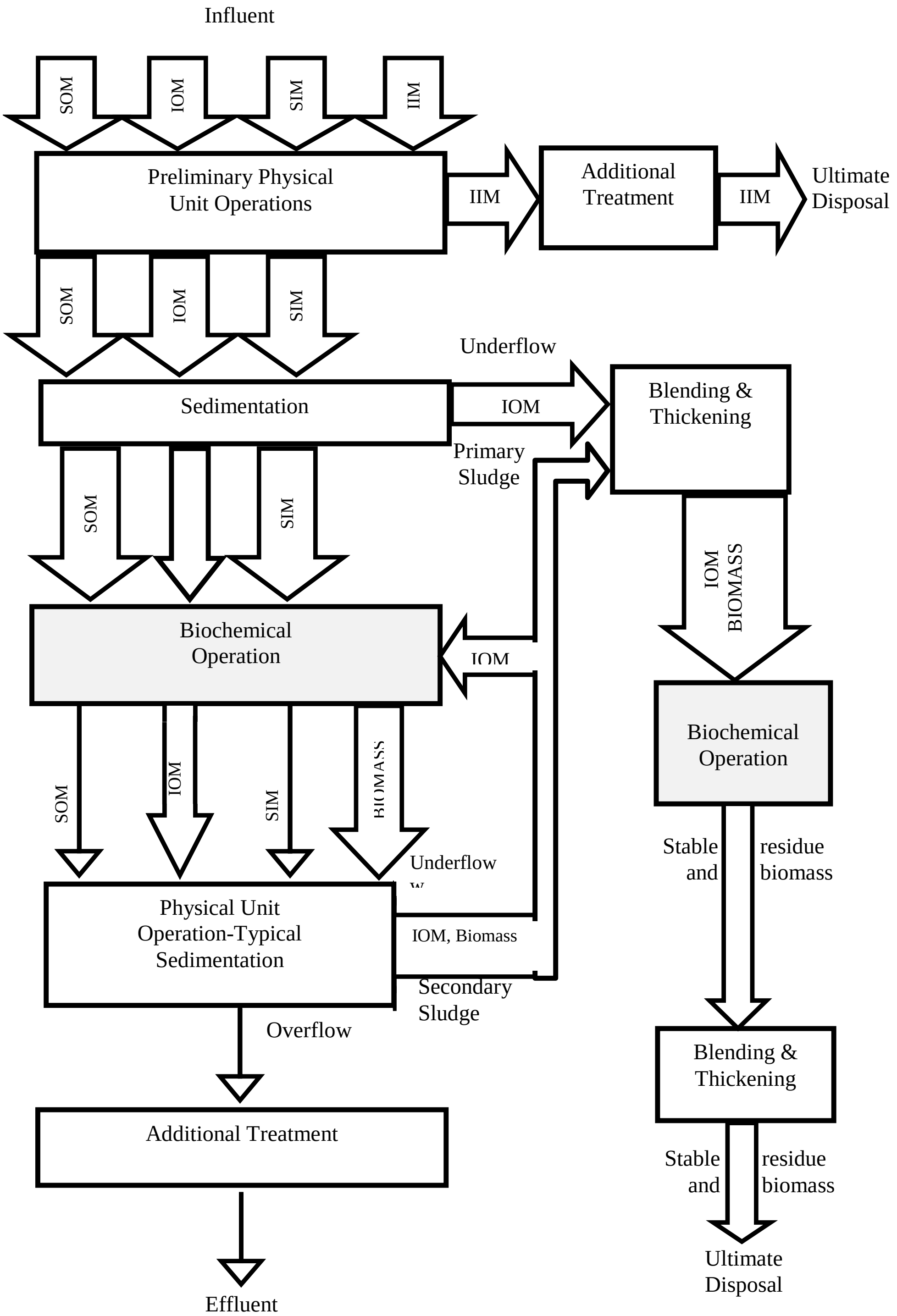
The main purpose of chapter two is to: 1) provide fundamental background information on wastewater treatment specifically the biological wastewater treatment and 2) discuss CFD application in wastewater treatment processes.

2.1 Overview of Wastewater Treatment

Wastewater Treatment Plants(WWTPs) play a significant role in protecting receiving waters from untreated discharges (Morera et al., 2016) . Wastewater that referring to a mixture of water-carried wastes(Metcalf & Eddy, 2013) originated from different sources (residences, institutions, commercial and industrial establishments) and activities requires to be treated before it can be discharged into the receiving waterbodies. The wastewater is treated with the purpose of removing the contaminants present in the wastewater to a level that natural systems can safely tolerate(Hamilton et al., 2006). A high degree of treatment is required if the effluent discharges to upstream of an abstraction point for water supply or a fishery area, while a lower level of treatment may be acceptable for discharges to the less risky area such as coastal waters where there is rapid dilution and dispersion. On a national scale, wastewater treatment represents 0.1 to 0.3% of total energy usage and within the local community, water and wastewater treatment operations consumed the largest portion of the overall energy consumption (Stillwell et al., 2010b).

One of the most significant stages of wastewater treatment is the biological treatment process. Biological treatment is normally aimed to coagulate and remove nonsettleable colloidal solids and to stabilize the organic content of wastewater (Tchobanoglous and Burton, 1991). In biological treatment processes, wastewater is brought into contact with microorganisms which degrade the pollutants, especially those that are carbon-based and nitrogen-based (Sardeing et al., 2005). The main factor of the biological treatment efficiency is to build up and preserve an acclimated, healthy biomass, adequate in quantity to handle maximum flows and the organic loads to be treated (Schultz, 2005). Figure 2.1 shows the flow diagram of typical wastewater treatment system.

One of the extensively used methods of biological treatment is an activated sludge treatment process. Activated sludge processes are widely used to treat municipal and industrial wastes (Insel, 2007) since they are adaptable, flexible and can be used to generate an effluent of desired quality by varying process parameters. The process was so-named because it produces an active mass of microorganisms capable of aerobically stabilizing a waste. If in the past wastewater treatment only focused on the removal of BOD and suspended solids, but nowadays, the aims are widen to the removal of aquatic toxicity, priority pollutants and volatile organics. Several types of activated sludge systems are now being applied around the world, including the conventional activated sludge, Extended Aeration (EA), sequencing batch reactors, and activated sludge with biological nutrient removal (BNR).



SOM = soluble organic matter; IOM = insoluble organic matter; SIM = soluble inorganic matter; IIM = insoluble inorganic matter.

Figure 2.1 : Flow diagrams of typical wastewater treatment system describing the role of the biochemical operations(Grady et al., 1999)

The Oxidation Ditch(OD) is one type of modified activated sludge process, which is also known as Intermittent Cycle Extended Aeration System (ICEAS)(Wu et al., 2012). There are many types of OD available, which are usually different in terms of aeration and mixing devices. Conventional wastewater treatment is considered an industrial activity where wastewater is changed by means of different processes, which use chemicals and energy, in treated water (of a better quality), which produces by products (mainly solid wastes and gaseous emissions) (Morera et al., 2016). Other than the water supply, wastewater facilities are among the largest users of electricity and usually considered as being only an energy sink (Nowak et al., 2015). Energy represents a significant cost to wastewater utilities, since it is normally required for all levels of the treatment processes, starting the collection of raw sewage until the discharge of treated effluent(Zhang et al., 2015).

2.1.1 Typical Processes in Wastewater Treatment

Wastewater treatment processes are designed to ameliorate the quality of the wastewater in order to prevent water-borne transmission of disease and to protect the aquatic environment(Tee et al., 2016). Wastewater constituents can be classified in several ways. According to Grady and his colleagues (1999), the contaminants in wastewater can be categorized based on their; 1) physical characteristics (soluble or insoluble), 2) chemicals characteristics (organic or inorganic) 3) susceptibility to alteration by microorganisms (biodegradable or nonbiodegradable) 4) origin (biogenic or anthropogenic) and 5) effects (toxic or nontoxic). In order to effectively treat these pollutants, it is crucial to have a wastewater treatment system that takes into consideration all these complicated and overlap characteristics.

Wastewater treatment can be classified into a few levels, such as preliminary, primary, secondary and tertiary level (as shown in Figure 2.2).The main difference between these levels is the objective of the treatment which parallel with the desired discharge quality of that particular treatment (as described in Table 2.1).In specific, the treatment level is associated with types of pollutant that going to be removed from the wastewater. Even though the most vital stage of any wastewater treatment process is the secondary level or well known as biological treatment, the processes which precede it also give significant effects in determining the efficiency of the secondary stage (Forster, 2003).

At the preliminary stage, wastewater constituents that may damage the treatment equipment are removed. These include the grit, grease, sticks, floatables and rags. At the primary treatment level, floating and settleable materials are removed. In some treatment plants, they also have advanced primary treatment, where at this stage, certain chemicals are added in order to improve the removal of suspended solids and to a smaller amount, dissolved solids(Metcalf & Eddy, 2013). Following the primary treatment is the secondary treatment or better known as biological treatment which is used for the removal of organic carbon and in some plants soluble nitrogen and phosphates. In several treatment plants, there is an additional treatment level called a tertiary or an advanced level. At this stage, the residual suspended solids which still remain in the wastewater even after the secondary treatment, are being removed.

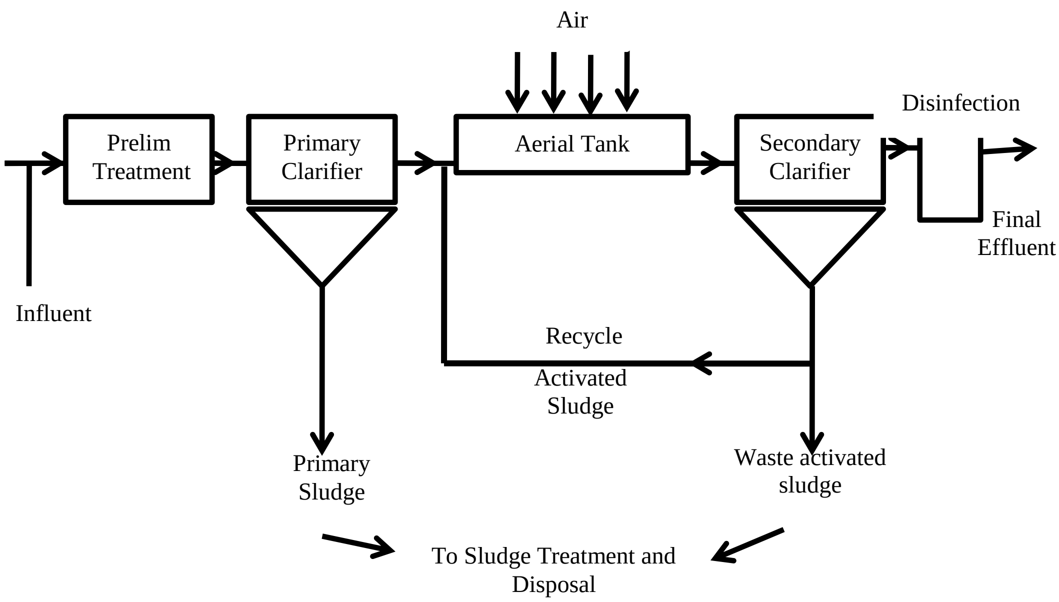


Figure 2.2: Typical processes in wastewater treatment

Table 2.1: Wastewater treatment level (Sperling and Chernicharo, 2005a)

Level	Removal
Preliminary	• Coarse suspended solids (larger material and sand)
Primary	• Settleable suspended solids • Particulate (suspended) BOD (associated to the organic matter component of the settleable suspended solids)
Secondary	• Particulate (suspended) BOD (associated to the particulate organic matter present in the raw sewage, or to the non settleable particulate organic matter, not removed in the possibly existing primary treatment) • Solubel BOD (associated to the organic matter in the form of dissolved solids)
Tertiary	• Nutrients • Pathogenic organisms • Non-Biodegradable compounds • Metals • Inorganic dissolved solids • Remaining suspended solids

2.1.2 Biological Wastewater Treatment

Biological wastewater treatment process is the most energy-intensive process of the overall wastewater treatment. It consumed 50-70% of total energy cost in wastewater treatment (Li et al., 2016). It was reported that the aeration requirement for the biological treatment process is the main cause of high energy costs of the overall wastewater treatment processes (Gu et al., 2016).Since 1980, many improvements have been achieved in terms of biological wastewater treatment, especially the identification of many actions that can happen simultaneously in biological processes(Grady et al., 1999).More advanced biological wastewater treatment technologies are being applied for designing new biological treatment plant facilities and upgrading the existing treatment facilities in order to achieve more effective and economical carbonaceous BOD and nutrients removal systems. Figure 2.3 shows the typical methods of biological treatment.

Generally, the wastewater that flows into the biological treatment plant still contains significant amounts of colloidal organics and dissolved materials that needs to be removed before it can be discharged into receiving water bodies (Metcalf & Eddy, 2013, Gray, 2004, Grady et al., 1999, Qasim, 1999). In particular, biological wastewater treatment process is meant to transform the dissolved and non-settleable materials into acceptable end products (Metcalf & Eddy, 2013) and biological cells which can then be removed through the settlement process that takes place in the secondary sedimentation tank (Gray, 2004).

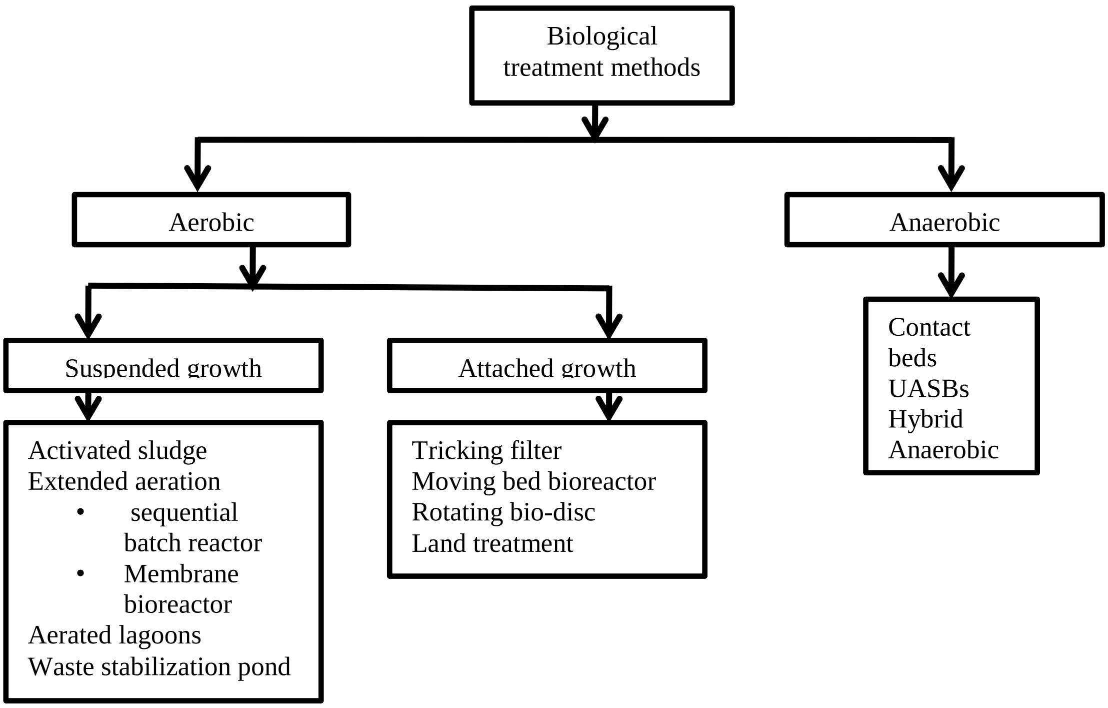


Figure 2.3: Typical methods of biological treatment

Conventional biological treatment processes capable of removing more than 85 percent of the BOD_5 and suspended solids (Qasim, 1999). On the other hand, the conventional biological treatment system does not work efficiently in terms of removing the nutrients and certain types of heavy metals and nonbiodegradable organics. Because of these reasons, in some treatment plants, more advanced biological treatment technology, which known as Biological Nutrient Removal (BNR) processes are designed to remove the nutrients such as nitrogen, phosphorus and in some cases, the specific trace organic compounds (Metcalf & Eddy, 2013).Since the BNR processes accomplish better nitrogen and phosphorus removal in addition to BOD and total suspended solids removal(Qasim, 1999), these processes have been applied extensively particularly in medium and large WWTPs.

One of the important aspects of biological treatment processes is the presence of mixed groups of microorganisms such as bacteria, protozoa, fungi, rotifers, nematodes algae and others (Metcalf & Eddy, 2013, Henze et al., 2002, Qasim, 1999) which originates either from the outside or inside the plant. The basis of the whole biological process is the effective contacts between the active microorganisms involved in the process, and the substrate (organic matter, nutrients and other substances) contained in the wastewater. Microorganisms that present in the wastewater are used to 'consume organics, nitrify ammonia, denitrify nitrate, and release and uptake phosphorus' (Qasim, 1999). The microorganisms transform the biodegradable organic matters into carbon dioxide, water, more cell material (through the growth and reproduction of the microorganisms) and other inert products. Another important factor for this biological decomposition of the organic matters to occur is the presence of oxygen as a fundamental component of the aerobic processes, besides the maintenance of other constructive environmental circumstances, such as temperature, pH, contact time, etc. In aerobic environment, the organic carbon is transformed into its most oxidized form, CO_2 (carbon in the oxidation state of 4+), whilst in anaerobic circumstances, the organic carbon is not only converted into its most oxidized form, but also into its most reduced form, CH4 (carbon with an oxidation state of 4-)(Sperling and Chernicharo, 2005c).

A structure or system where the biological wastewater treatment takes place is known as biological reactor or bioreactor. Wastewater treatment bioreactors can be classified into two main categories that are based on how microorganisms grow. It is either they are suspended growth or attached growth. In suspended growth processes, mixing mechanisms are used in order to keep the biomass in suspension, and some types of physical unit operation (i.e. sedimentation) are used to remove the biomass from the treated wastewater (Grady et al., 1999) before it may be discharged into the receiving water bodies. Most of the suspended growth processes used in domestic and industrial wastewater treatment are operated in aerobic condition. However, in some cases such as for high organic concentration industrial wastewaters and organic sludges, anaerobic suspended growth processes are applied (Metcalf & Eddy, 2013).Aerobic processes are the processes occur in the presence of oxygen, whereas anaerobic processes are referring to the treatment processes take place in the absence of oxygen.

Attached growth processes can either be operated in aerobic or anaerobic conditions. In attached growth processes, microorganisms are attached to a solid support (Grady et al., 1999) such as rock, slag, wide range of plastic and other synthetic materials (Metcalf & Eddy, 2013). The organic matter and nutrients are removed from the wastewater flowing past the solid support or also known as biofilm. According to Gray (2004), suspended growth processes are more intensive than attached growth systems and capable to treat up to ten times more effluent per unit volume of reactor. In terms of capital costs, suspended growth systems are much cheaper compared to attached growth systems. However, suspended growth processes need high costs to be paid for their operation systems, including the aeration systems and pumping sludge from the settlement tank back to the reactor.

Biological wastewater treatment is viewed as the most important and very complex unit processes in wastewater treatment(Henze et al., 2002, Olsson and Newell, 1999).

In order to deal with the complexity of the biological processes, it is crucial to understand the entire concept of the processes. These include the objectives and the requirements so as to provide an appropriate environment for effective and efficient biological processes. In order to highlight the objectives of applying biological treatment system, Metcalf and Eddy (2003) have summarized at least four purposes of the biological treatment of domestic wastewater which are to; 1) transform or oxidize soluble and particulate biodegradable constituents that escape the primary treatment into acceptable end products, 2) detain and integrate suspended and nonsettleable colloidal solids into a biological floc or biofilm, 3) convert or remove nutrients such as nitrogen (through the nitrification and denitrification processes) and phosphorus and 4) in more advanced cases, remove specific trace organics constituents and compounds in order to upgrade the effluent quality(Qasim, 1999) or to accomplish advanced wastewater treatment (particularly applied when the receiving waters classified as a sensitive area).

The objectives of the biological wastewater treatment can only be accomplished within the proper environment. Thus, it is vital to develop and control the biological processes' environment based on the information of their basic requirements. Qasim (1999) in his book emphasized that the basic requirements of the biological treatment processes are the; 1) diverse population of microorganisms, 2) good contact between the microorganisms and substrate, 3) the presence of optimum amount of oxygen (when the biological treatment applied aerobic conversion), 4) accessibility of nutrients and 5) preservation of other encouraging environmental factors, such as temperature, pH, adequate contact time and others.

2.1.2.1 Oxygen utilization in biological treatment

Basically, the oxygen consumption in biological treatment is due to a few processes such as the;1) oxidation of the carbonaceous organic matter including the oxidation of the organic carbon to supply energy for bacterial synthesis and endogenous respiration of the bacterial cells, 2) the stabilisation of the sediment and 3) nitrification process.

With the presence of oxygen, organic substances remain in the watercourse will be transformed into simple compounds such as water and carbon dioxide by a number of microorganisms. The simplified equation for the oxidation of organic substance is shown in equation below(Sperling and Chernicharo, 2005c).

$$\text{Organic matter} + O_2 + \text{bacteria} \rightarrow CO_2 + H_2O + \text{bacteria} + \text{energy} \qquad \textbf{(2.1)}$$

Apart from the oxidation of carbonaceous organic materials, the oxygen also consumed to stabilise the upper part of the settled organic matter in suspension. The oxygen consumption associated with the sludge is known as benthic or sediment demand(Sperling and Chernicharo, 2005b).

In aerobic biological treatment, the oxygen also consumed in the nitrification process which involving the conversion of ammonia into nitrite and then into nitrate. The conversion of ammonia into nitrite is illustrated in Equation 2.2, whereas Equation 2.3 expresses the transformation of nitrite into nitrates. The overall reaction of nitrification process is demonstrated by Equation 2.4.

$$2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 4H^+ + 2H_2O + Energy \qquad \textbf{(2.2)}$$

$$2NO_2^- + O_2 \rightarrow 2NO_3^- + Energy \qquad \textbf{(2.3)}$$

$$NH_4^+ + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O + Energy \qquad \textbf{(2.4)}$$

The oxygen consumption in this process is recognized as second-stage demand, because it happens after the oxidation of most of the carbonaceous matter. This is because of the slower growth rate of nitrifying bacteria compared to the heterotrophic bacteria. This explains that the occurrence of nitrification process also at the slower rate compared to the oxidation of the carbonaceous organic matter.

2.1.2.2 BOD reduction and nutrient removal

Previously, biological treatment systems were only concentrating on the removal of the oxygen-demanding materials that would reduce the dissolve oxygen (DO) in receiving body. As the knowledge capacity of wastewater treatment increased, more problems related to the wastewater constituents are recognized. The latest problem discovered that related to the wastewater constituents is the discharged of toxic organic chemicals to the water receiving bodies. Hence, researchers who involved in this field are struggling to overcome all these problems and attempting to discover a better approach for a better effluent quality.

Usually, biochemical oxygen demand (BOD) caused by the organic matter is used to indicate the quality of the wastewater. In other words, this is to estimate the amount of oxygen required to oxidize all the substrates present in the wastewater. Carbonaceous BOD removal can be accomplished either in aerobic or anaerobic conditions. BOD reduction can be performed either in suspended growth or in attached (fixed film) growth treatment processes. Any of these processes requires sufficient contact time between the heterotrophic microorganisms and the substrates present in the wastewater. In aerobic treatment processes adequate amount of oxygen are also required. The BOD removal is carried out by the microorganisms through two separate processes which known as biological oxidation and biosynthesis (Gray, 2004).According to Metcalf& Eddy (2013), more than half of the carbonaceous organic matter is oxidized during the initial biological uptake, while the residue is digested as new biomass and may be further oxidized by endogenous respiration.

In aerobic condition, through the oxidation and respiration processes, the organic material is transformed into inert materials such as carbon dioxide and water. These inert materials remain in solution and discharged in the final effluent. At the same time, biosynthesis process also taking place where the colloidal and soluble organic matters are converted into particulate biomass or new cells(Gray, 2004). In any case, either it suspended or attached growth processes, the excess biomass is removed and processed on a daily basis in order to maintain good environment within the biological reactor(Metcalf & Eddy, 2013). The biomass is removed from the treated effluent using the gravity separation or more often known as settlement process. In a situation when the organic matter which considered as the food supply becomes limited, the microbial cell tissue is endogenously respired (auto-oxidation) by the microorganisms to acquire energy for maintenance(Gray, 2004).

Gray (2004) stated the stoichiometry and generalized reaction of three processes that happen simultaneously in the biological reactor as shown in Equations 2.5, Equation 2.6 and Equation 2.7. COHNS in these equations represent the organic matter.

Oxidation:

$$COHNS + O2 + bacteria \longrightarrow CO2 + NH3 + other\ end\text{-}products + energy \quad \mathbf{(2.5)}$$

Biosynthesis:

$$COHNS + O2 + bacteria \longrightarrow C5H7NO2 \quad \mathbf{(2.6)}$$

Auto-oxidation:

$$C5H7NO2 + 5O2 \longrightarrow 5CO2 + NH3 + 2H2O + Energy \quad \mathbf{(2.7)}$$

Process control options have been successfully implemented for the upgrade of existing carbon removing activated sludge systems together with improvement of nitrogen removal performance in BNR (Insel, 2007). In activated sludge system, intensive interaction occurs between different species in the sewage and activated sludge (Lei and Ni, 2014). Many types of BNR have been developed to remove the major sources of eutrophication, which are nitrogen and phosphorus. In domestic wastewater, nitrogen is generally present as ammonia and organic nitrogen. Amino acids, proteins and nucleotides are some examples of organic nitrogen present in the wastewater. The organic nitrogen is converted to ammonia through ammonification process, which starts in sewerage system itself and does not involve any oxygen consumption(Sperling and Chernicharo, 2005c).

Simultaneous nitrification and denitrification (SND) was evidently observed in a full scale multi-channel oxidation ditch, especially occurring within the outer channel (Zhou et al., 2015).Nitrification process can be expressed as a two-step biological process(Metcalf & Eddy, 2013) in which ammonia (NH_4-N) is oxidized to nitrite (NO_2-N) and nitrite is oxidized to nitrate (NO_3-N). The whole reactions of nitrification processes are shown in Equations 2.2, 2.3, 2.4. First step of nitrification is a conversion of ammonia into nitrites by authotrophic bacteria such as Nitrosomonas. This process is followed by a conversion of nitrites into nitrates by another group of autotrophic bacteria such as Nitrobacter. In comparison to heterotrophic bacteria, nitrifying bacteria have a very slow growth rate. Therefore, nitrification process occurs at 3 to 4 times slower rate compared to carbonaceous oxidation process. Concentration of the dissolved oxygen may also have an effect on nitrification. Nitrifying organisms use dissolved oxygen as the terminal electron acceptor. According to Gray (2004), nitrification process does not occur below 0.2-0.5 mg O_2 l^{-1}.

Nitrification process just transforms and does not remove any nitrogen, where the total amount of nitrogen still remains the same. The removal of nitrogen only occurs through denitrification process where nitrates are reduced to nitrogen gas which then escapes to the atmosphere. Denitrification process occurs in anoxic conditions, where in the absence of oxygen and the presence of nitrates as the terminal electron acceptor (Grady et al., 1999). The biological reduction of nitrate involves a few steps which start from nitrate to nitrite, to nitric oxide, to nitrous oxide and finally to nitrogen gas (shown in Equation 2.8).

In biological reactor such as OD, nitrification and denitrification processes occur concurrently caused by the natural internal recirculation. As concluded by Zhou et al. (2015), the observations of nitrogen removal and micro environment characteristics coupled with mass balance analysis demonstrated simultaneous nitrification and denitrification is seemed as an underlying mechanism of TN removal in a multichannel OD system(Daigger and Littleton, 2014).Apart of reducing the oxidized nitrogen concentration, denitrification process releases oxygen into the water, which can be used by heterotrophs. In addition, denitrification also consumes some of the residual organic matter during nitrification (Gray, 2004).

$$NO_3^- \rightarrow NO_2^- \rightarrow NO \rightarrow N_2O \rightarrow N_2 \qquad \textbf{(2.8)}$$

Apart from nitrogen, another type of nutrients, which is getting a serious attention from wastewater industries, is phosphorus. Biological phosphorus removal is performed to control eutrophication because phosphorus is a limiting nutrient in the majority freshwater systems (Metcalf & Eddy, 2013). In domestic wastewater, phosphorus can be found in organic and inorganic forms. Influent phosphorus occurs mostly as soluble orthophosphate and appears to be finally removed as insoluble polyphosphate nodules inside a class of organisms known as Phosphorus Accumulating Organisms(PAOs)(Olsson and Newell, 1999).

In order to achieve efficient oxidation processes, where the microorganisms can efficiently break down the organic material, supplying an adequate amount of oxygen is viewed as a main issue in biological wastewater treatment. For instance, simultaneous nitrification and denitrification occurred at very low DO concentrations between 0.10-0.25mg/l (Zhou et al., 2015). Many approaches have been used to support or accomplish the maximum oxidation processes. Either it is conventional or an advanced method, the objective is still the same, which is to ensure that enough oxygen comes into contact with the wastewater being treated. Gray (2004) pointed out three main methods usually used to achieve oxidation processes, which are by; 1) dispersing the wastewater into a film of liquid with a huge surface area so that all the needed oxygen can be provided by gaseous diffusion (e.g. percolating filter), 2) aerating the wastewater by pumping in bubbles of air or stirring vigorously (e.g. activated sludge) and 3) depending on the algae present to generate oxygen by photosynthesis (e.g. stabilisation pond).

2.1.2.3 Types of biological treatment processes

Biological treatment processes can be classified into various groups based on two different factors, which are metabolic function (aerobic processes, anaerobic processes and combined processes), and treatment processes (suspended growth processes, attached growth processes and combined processes). Description of these processes can be divided into the following categories; 1) aerobic suspended growth processes, 2) aerobic attached growth processes, 3) anaerobic suspended growth processes, 4) anaerobic attached growth 5) combined processes that include anaerobic, anoxic and aerobic processes. Table 2.2 shows the major biological treatment processes applied for wastewater treatment.

Aerobic suspended growth processes are the treatment operated under the presence of oxygen and the microorganisms remain in suspension. One of the common suspended growth process used for biological treatment is activated sludge. Activated sludge has been widely a used technology for efficient biological nitrogen removal from domestic and industrial wastewater (Insel, 2007). As shown in Table 2.2, the aerobic biological wastewater treatment processes are used to bio-oxidize biodegradable organic matter and to convert ammonia and organic nitrogen to nitrate via the biological process of nitrification (Daigger and Littleton, 2014). The mixed liquor flows from the biological reactor will settle in the secondary clarifier and some part of the settled sludge will return to the bioreactor (shown in Figure 2.4) in order to maintain the proper food to microorganism ratio (Qasim, 1999). The other part of the sludge (excess sludge or surplus sludge) is withdrawn from the system and sent to the sludge treatment stage (Sperling and Chernicharo, 2005b). The main types of biological reactors for the activated sludge processes are plug-flow, complete mix and arbitrary flow. In this process, the microorganisms are mixed thoroughly with influent and the mixture is known as mixed liquor.

Table 2.2: Major biological treatment processes used for wastewater treatment (Metcalf & Eddy,2013)

Type	Common name	Use
Aerobic processes		
Suspended growth	Activated sludge process(es)	Carbonaceous BOD removal, nitrification
	Aerated lagoons	Carbonaceous BOD removal, nitrification
	Aerobic digestion	Stabilization, Carbonaceous BOD removal
Attached growth	Trickling filters	Carbonaceous BOD removal, nitrification
	RBC	Carbonaceous BOD removal, nitrification
	Packed-bed reactors	Carbonaceous BOD removal, nitrification
Hybrid (combined) suspended and attached growth processes	Trickling filter/activated sludge	Carbonaceous BOD removal, nitrification
Anoxic processes		
Suspended growth	Suspended-growth denitrification	Denitrification
Attached growth	Attached-growth denitrification	Denitrification
Anaerobic processes		
Suspended growth	Anaerobic contact processes	Carbonaceous BOD removal
	Anaerobic digestion	Stabilization, solids destruction, pathogen kill
Attached growth	Anaerobic packed and fluidized bed	Carbonaceous BOD removal, waste stabilization, denitrification
Sludge blanket	Upflow anaerobic sludge blanket	Carbonaceous BOD removal, especially high-strength wastes
Hybrid	Upflow sludge blanket/attached growth	Carbonaceous BOD removal
Combined aerobic, anoxic, and anaerobic processes		
Suspended growth	Single- or multistage processes, various propriety processes	Carbonaceous BOD removal, nitrification, denitrification, and phosphorus removal
Hybrid	Single- or multistage processes with packing for attached growth	Carbonaceous BOD removal, nitrification, denitrification, and phosphorus removal
Lagoon processes		
Aerobic lagoons	Aerobic lagoons	Carbonaceous BOD removal
Maturation (tertiary) lagoons	Maturation (tertiary) lagoons	Carbonaceous BOD removal, nitrification
Facultative lagoons	Facultative lagoons	Carbonaceous BOD removal
Anaerobic lagoons	Anaerobic lagoons	Carbonaceous BOD removal, waste stabilization

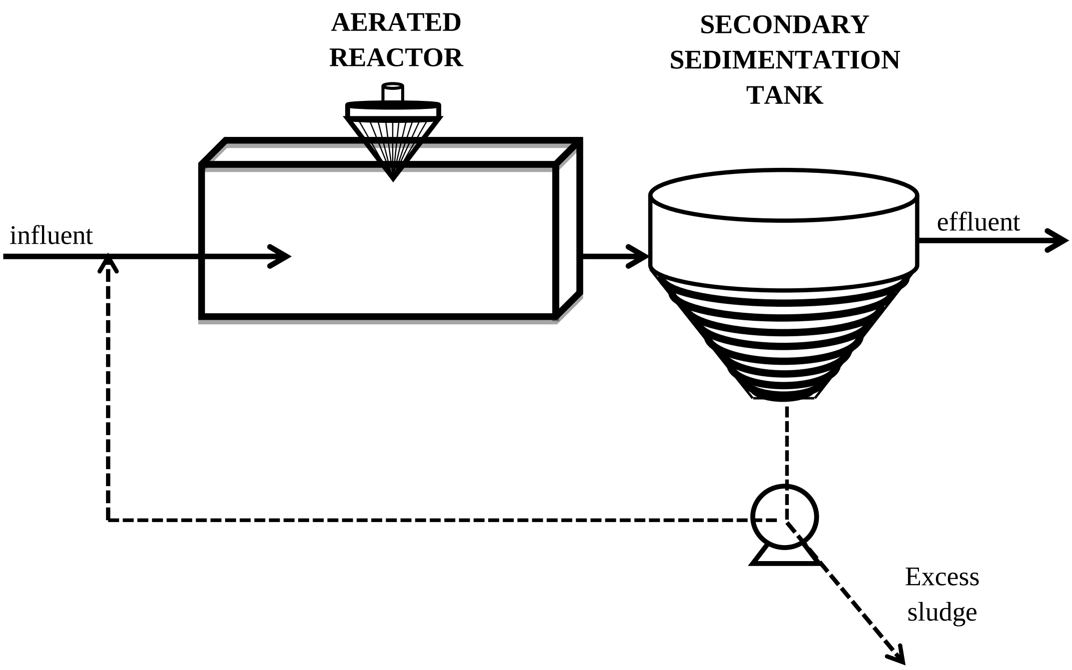


Figure 2.4 : Mixed liquor flows from bioreactor to secondary clarifier
Source: (Metcalf & Eddy, 2013)

Normally, about 99 percent of the suspended solids can be removed in the clarification stage (Metcalf & Eddy, 2013). The air is supplied by the mixers or some type of aeration devices such as mechanical mixers or diffuser. The aerator devices consist of submerged or partly submerged impellers that are attached to motors mounted on a float or on fixed structures. Characteristics of various mechanical aeration devices are given in Table 2.3. One of the most important criteria in the selection of aeration devices is having an effective system and least costly methods. As described in Table 2.3, mechanical aerators can either be surface designs or submerged. Some authors viewed the submerged propeller as an important dynamic source in an extended aeration system such as oxidation ditch (Wu et al., 2012). Submerged aerators such as aeration discs have known with its high oxygen transfer efficiency and its flexibility. For instance, the changes of the discs' immersion and speed will change the oxygen delivery and power consumption (Evoqua Water Technologies, 2015). The disc itself is split into two half sections and can be directly attached to the aerator shaft at any location. This makes it easy to add on discs to existing shaft sections for future purposes.

Other applications of a suspended growth biological treatment process are aerated lagoon and stabilisation ponds (Qasim, 1999). Stabilisation ponds system is the simplest type of wastewater treatment. According to Sperling and Chernicharo(2005a),this type of treatment is strongly recommended for warm-climate areas and developing countries due to some factors such as; 1) simple operation, 2) favourable climate (high temperature and sunlight), 3) sufficient land availability in a large number of locations and 4) little and sometimes no equipment required.

Table 2.3: Characteristics of mechanical aeration devices (Qasim, 1999)

Aeration System	Description	Advantages	Disadvantages
Vertical axis	The shaft is vertical, and blades are attached in the shaft. The motor sits on top of a fixed or floating platform.		
Surface aerator	The impeller is submerged or partly submerged. The radial flow aerator is low speed (20-100rpm) and has a gear box to reduce speed. The motor is mounted on the float or on a fixed structure. The motor action induces updraft flow. Draft tube may be placed below the impeller to induce circulation	Flexibility in tank shape and size; good mixing	Initial cost high, icing in cold climate; gear reducer may cause maintenance problem.
	The high speed surface aerators (axial flow) have speeds of 300 to 1200 rpm and are mostly mounted on floats	Low initial cost, can be adjusted to varying water level, flexible operation.	Icing in cold climate, poor accessibility for maintenance, mixing

			inadequate.
Submerged or turbine aerator	The impeller is submerged and piped air or oxygen is delivered to a point below the impeller. The impeller disperses the air into fine bubbles and mixes the contents of the tank. Draft tube may also be used to increase circulation. Air flow may vary from 4 to 8 L/	Good mixing, suitable for deep tank, operational flexibility, no icing or splash.	High initial cost, require both gear reducer and blower, high power requirement.
Horizontal axis	The aerator has a horizontal axis. A cylinder or drum either exposed or submerged provides aeration and forward movement of the liquid. Commonly used in OD.		
Surface aerator or brush aerator	Consists of cylinder or drum with bristles of steel protruding from the perimeter into wastewater; provide aeration and move the liquid forward	Provide aeration and circulation; moderate initial cost, good maintenance accessibility.	Tank geometry is limited, gear reducer, low efficiency
Submerged disc aerator	Consists of discs that are submerged in the liquid approximately one-eighth to three-eighth of the diameter. The recesses in the disks introduce entrapped air into the submerged section. The disc spacing and submergence can vary depending upon the oxygen requirement	Same as surface aerator.	Tank geometry is limited, gear reducer

Aerobic attached growth processes involve the formation of active microorganisms over a solid media such as rock or plastic. Two major types of attached growth processes are trickling filters and contactors Rotating Biological Contactors(RBC) Trickling filters are usually classified based on the hydraulic and organic loading (Sperling and Chernicharo, 2005b). It can be classified into low-rate, intermediate-rate, high rate and super rate. Rotating biological contactor or also known as bio-disc process consists of a series of circular plastic discs mounted over a shaft that slowly rotate. (Qasim, 1999). Even though the rotating biological contactor is not frequently used in developing countries, but this treatment system has become an alternative for the sewage treatment in small and medium urban areas (Sperling and Chernicharo, 2005b).

Another type of biological wastewater treatment is anaerobic suspended growth treatment system.Some of the reactors that applied anaerobic suspended growth treatment system are anaerobic digestion, anaerobic contact process and upflow anaerobic sludge blanket reactor (UASB).According to Sperling and Chernicharo (2005a), some of the advantages of this system are; 1) solids production 3 to 5 times lower than aerobic processes, 2) low energy consumption, 3) low land requirements and 4) tolerance to high organic loads. Nevertheless, the use of anaerobic processes for the wastewater treatment was considered uneconomical and problematic due to some disadvantages such as; 1) substandard removal of nitrogen, phosphorus and pathogens, 2) some form of post-treatment is usually required and 3) reduced growth rate of the anaerobic biomass makes the control of the process delicate, since the recovery of the system is very slow when the anaerobic biomass is exposed to adverse environmental conditions(Sperling and Chernicharo, 2005b).Anaerobic attached growth process is another system of biological treatment which using a support medium for microorganisms to attach. Anaerobic filter and expanded bed process are two common processes of anaerobic attached growth biological treatment.

The biological nutrient removal processes (BNR) have been studied by numerous researchers and observed in numerous treatment systems over an extended period of time (Daigger and Littleton, 2014). Because of that, many existing biological treatment processes have been modified and upgraded in order to meet requirements of the effluent quality. Some of the processes are modifications of the basic biological treatment processes. These include the modifications of the activated sludge processes to achieve more effective and efficient nutrients (nitrogen and phosphorus) removal. Many of developed countries have already overcome most of the problems of carbonaceous matter (BOD and COD) in their effluents and currently aim to move to a second stage of priorities, which concerns BNR (Sperling and Chernicharo, 2005b). Biological nitrogen removal can be achieved in bioreactor with the occurrence of nitrification and denitrification processes. Combined nitrification-denitrification can be accomplished in a single reactor such as Oxidation Ditch(OD) or a series of reactors that create aerobic and anoxic conditions (Qasim, 1999).

OD, which is an EA system,consists of oval-shaped or a ring channel equipped with aeration devices. Screened wastewater flows into the channel and is combined with the return activated sludge (Metcalf & Eddy, 2013). Anoxic and aerobic condition is developed and maintained in zones up-stream and downstream of the rotor(Qasim, 1999). OD process is a highly reliable process and competent of treating shock loads without having an effect on the effluent quality(Metcalf & Eddy, 2013). The Orbal process is one of the modified oxidation ditches that applying a sequence of concentric channels inside a single biological reactor. OD is better than other biological treatment system because of its unique mixing performance (Wu et al., 2012).

A selection of any biological wastewater treatment system should be based on the objective of the treatment, the capacity of the area where the treatment system is built and the quality of the discharge that need to be accomplished. The requirements of one location may possibly be different from the others. For instance, the effluent standards in developed countries are much more stringent compared to the developing countries. A successful biological wastewater treatment system requires good understanding of fundamental factors including; 1) the nature of biochemical transformation involved, 2) the environment in which the transformation occurred and 3) the reactor configuration employed (Metcalf & Eddy, 2013).

2.2 The Orbal Biological System (OBS)

The Orbal Biological System (OBS)is a modification of OD which is equipped with mechanical aeration and mixing devices. The popularity of the OD is mainly due to its reliability, simplicity of operation and good treatment performance. This system is suitable for various processes such as conventional activated sludge, advanced secondary sludge treatment, simultaneous nitrification-denitrification processes, biological phosphorus removal and storm water treatment (Evoqua Water Technologies, 2015). OBS uses a series of concentric channels (shown in Figure 2.3) within the same structure (Metcalf & Eddy, 2013),with the outer channel having half of the total volume. Screened wastewater enters the outer channel and flows from there to middle channel and finally to inner channel before the mixed liquor flows to the clarifier. Return activate sludge (RAS) from the secondary clarifier is also added to the outer channel. The channels are interconnected and the flow is directed in an inward direction.

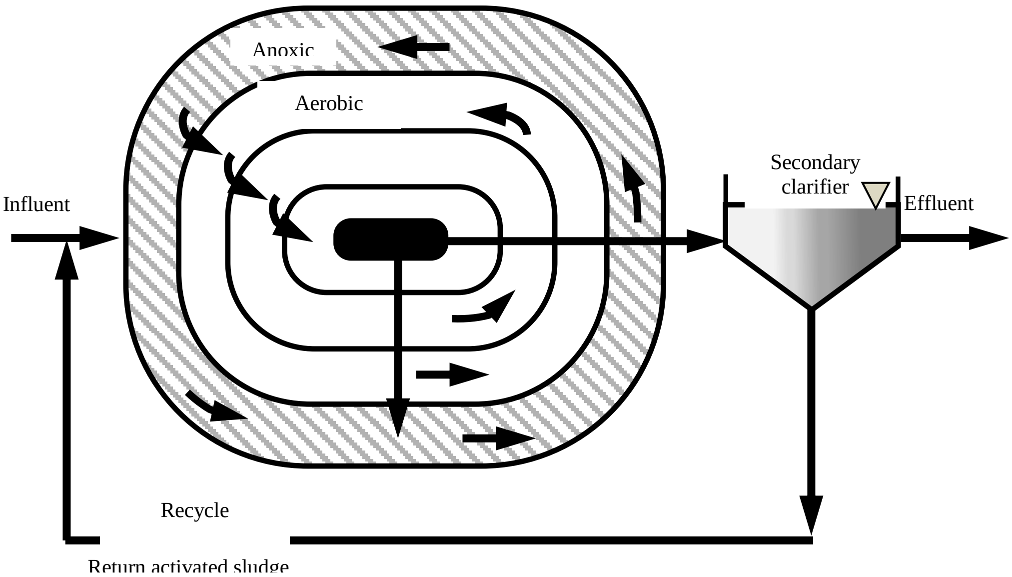


Figure 2.5: Orbal Biological System(Metcalf & Eddy, 2013)

The transfer ports are placed so that the inlet to a channel is downstream of the outlet from the same channel, thereby minimising short-circuiting. A more detailed description of each channel is described well by Siemens Water Technologies (2007)as given in Table 2.4. The most important feature of OBS is its aeration device. The unique OBS aeration discs are claimed to provide high oxygen transfer and mixing efficiency(Yong et al., 2016).Making dissolved oxygen (DO) transfer from gas to liquid phase is very energy intensive activity in the treatment plant, as well as crucial for the treatment results (Amand, 2011). The system is designed to provide an extensive operational flexibility especially in terms of the ability to manipulate oxygen transfer rates. There are many advantages of this system claimed by its manufacturer. However, WWTPs that are currently applying the system faced some significant problems related to its operational system. High power usage by aerators has commonly been a major issue for EA biological system such as Orbal and also other multichannel ditches .

It is well known that full-scale bioreactors do not provide an entirely uniform environment. A system such as OBS, intense oxygen transfer occurs in one portion of the bioreactor, limited oxygen transfer occurs throughout the rest of the bioreactor and mixed liquor is recycled between the aerated and non-aerated zones (Daigger and Littleton, 2014).

Table 2.4: Detailed Channel Description(Technologies, 2007)

Channel	Design Description
1st channel (outer)	The first channel is the place where the majority of the process 'work' takes place. It is operated under an oxygen deficit condition in order to promote simultaneous nitrification-denitrification processes. Majority of the system's nitrification takes place in this channel. Although the real oxygen demand of the outer channel might be up to 75% of the total, the aeration discs fixed to this channel provide only 30-60% of the systems' overall oxygen requirements to ensure a constant oxygen deficit condition and an operating DO of zero throughout the channel.
2nd channel (middle)	The DO of the second aeration channel operates in a swing-mode. Although designed for 1 mg/L DO, the actual operating DO varies with the daily load conditions, being reduced to near zero during the peak loads of the day, and rising to 2 mg/L during low load conditions.
3rd channel (inner)	The DO of the last channel is designed for 2 mg/L, keeping this channel in a polishing mode to remove any remaining BOD and ammonia before the flow exits to final clarifiers, Since the oxygen demand of the last channel is only a fraction of the first, only a small amount of oxygen need to be delivered to maintain a high DO.

2.3 Sewage Treatment in Malaysia

Based on the data provided by the Malaysia Department of Statistics (2016), Malaysia's population has exceeded 31 million. A population growth has caused more pressure on the environment and threatens the sources of fresh water supplies due to the waste management problems. Malaysian generates more than six million tons of sewage every year, most of which is treated and released into the rivers. In

Malaysia, 98 percent of fresh water supply comes from surface water. Hence it is very important to make sure that the effluent is well treated before discharging it into the river. WWTPs are escalating in Malaysia as a reaction to the increasing demands for better and more effective treatment systems. Most wastewater treatment processes in Malaysia involve primary treatment systems such as communal septic tanks followed by low cost secondary systems such as oxidation ponds. Based on the data provided by IWK, 38% of public STPs in the country are mechanical plants and individual septic tanks are usually used in urban areas. At present, there are over one million individual septic tanks in Malaysia (Indah Water Konsortium, 2016).

Currently, the water services industry in Malaysia is regulated by National Water Services Commission. Its mission is to provide a sustainable, reliable and affordable water services for all by regulating the water services industry through fair, effective and transparent implementation of the Water Services Act (ACT 655). National Water Services Commission or Suruhanjaya Perkhidmatan Air Negara (SPAN) aims to enhance efforts towards improving standards, quality and operational efficiency of water and sewerage services industry to ensure sustainability (National Water Services Commission, 2016). Malaysia's sewerage services are provided by the Indah Water Konsortium(IWK). Indah Water Konsortium, a national sewerage company wholly-owned by the Minister of Financed Incorporated, is responsible for providing sewerage services, operating and maintaining over 5567 public STPs and 14 190 km networks of sewerage pipelines (Indah Water Konsortium, 2016).

IWK is currently improving their operational systems at WWTPs, especially secondary biological treatment systems. Currently, the trend in Malaysia is moving towards implementing and upgrading mechanical plants such as EA activated sludge, OD, RBC, sequencing batch reactors (SBR) and trickling filters (Indah Water Konsortium, 2016) . Currently, there are approximately 30 oxidation ditches in Malaysia. The new modified oxidation ditches in Malaysia are located in Sg. Besi (Kuala Lumpur), Bayan Baru (Penang) and Cyberjaya (Selangor). Table 2.5 provides an overview of the wastewater treatment system in Malaysia.

A systematic monitoring approach is introduced in order to control the effluent quality. In Malaysia, the regulation set by the Environmental Quality Act (EQA) 1974 stipulates two sets of limit for effluent discharge which documented as Standard A and Standard B. The standards are listed in the Third Schedule of the Environmental Quality Act 1974, under the Environmental Quality (Sewage and Industrial Effluents) Regulations, 1979, regulations 8 (1), 8 (2) and 8 (3)'(Sewerage Services Department, 1998). Standard A is applied to discharge upstream of any raw water intake points or any sensitive areas recreational, coastal areas with tourism interests and areas with high ecological values such as marine parks and wet lands. Standard B is applied to discharge downstream of any raw water intake or any other areas that do not fall under Standard A.

Table 2.5: Profile of sewerage system 2013-2014 (National Water Services Commission, 2016)

Sewerage Facilities	2013		2014	
	Quantity	Population Equivalent (PE)	Quantity	Population Equivalent (PE)
Public Sewage Treatment Plant (a+b)	6167	21,549,561	6374	22,517,132
a. Multipoint Plant b. Regional Plant	6085 82	15,142,046 6,407,515	6288 86	15,724,638 6,792,494
Private Sewage Treatment Plant	2762	2,697,074	2972	2,813,248
Septic Tank & Pour Flush				
Communal Septic Tank	4378	525,240	4,377	529,780
Individual Septic Tank	1,306,662	6,669,142	1,324,083	6,739,192
Pour Flush	894,859	4,474,293	894,859	4,474,293
Others				
Network Pumping Station	975	N/A	1027	N/A
Sewer Network (km)	17,384	N/A	18,076	N/A
Notes:N/A: Not Applicable				

2.4 Computational Fluid Dynamics (CFD) Model Development

Computational Fluid Dynamocs (CFD) is a branch of fluid mechanics that uses numerical methods and algorithms to solve and analyse problems that involve fluid flows. CFD is one of the computer-based modelling approaches which extensively used to improve the wastewater treatment system as well as energy consumption (Bosma and Reitsma, 2008). CFD model incorporate an account of flow geometry, a set of differential equations demonstrating the physics and chemistry of the flow, boundary conditions and grids at which these equations are solved (Yang et al., 2011). Governing equations which based on fundamental equations of fluid dynamics such as continuity, momentum and energy equations are programmed in Fluent using the Navier-Stokes equations (Yang et al., 2011, Littleton et al., 2007b).

The riches of physical models in Fluent allow an accurate prediction of either laminar or turbulent flows, various modes of heat transfer, chemical reactions, multiphase flows and other phenomena with complete mesh flexibility and solution-based meshed adoption (Hadad and Ghaderi, 2015). Fluent can use both structured and unstructured grids. Based on the past experience of other researchers, meshing or grid generation is the most important step of CFD modelling. The accuracy of the outputs given by CFD modelling is mostly dependent on the meshing sizes. Finer meshing produces the most accurate results. However, in order to run a model with fine grids, a very high computer capacity is needed, so the desired accuracy level and the available computer capacity have to be balanced. The meshing procedure is somewhat trial and error and continues until the produced grids achieve the acceptable accuracy level and satisfy the available computer capacity.

Generally, CFD model includes a description of flow geometry, a set of differential equations describing the physics and chemistry of the flow, boundary conditions and mesh points at which these equations are solved (Yang et al., 2011). These governing equations are programmed in FLUENT for conservation of mass, momentum and energy using the Navier-Stokes equations (Yang et al., 2011, Littleton et al., 2007b).

The Reynolds-averaged, Navier-Stokes equations govern the motion of fluids and can be seen as Newton's second law of motion for fluids. In the case of a compressible Newtonian fluid, this yields.

$$\underleftrightarrow{\rho \left(\frac{\partial u}{\partial t} + u.\nabla u\right)} = \underleftrightarrow{-\nabla p} + \underleftrightarrow{\nabla.\left(\mu(\nabla u + (\nabla u)^T) - \frac{2}{3}\,\mu\,(\nabla.u)\mathrm{I}\right)} + \underleftrightarrow{F} \qquad \textbf{(2.9)}$$

The different terms correspond to the inertial forces (1),pressure forces (2), viscous forces (3) and external forces applied to the fluid (4). These equations are always solved together with the continuity equation:

$$\frac{\partial \rho}{\partial t} + \nabla.(\rho u) = 0 \qquad \textbf{(2.10)}$$

The Navier-Stokes equations represent the conservation of momentum, while the continuity equation represents the conservation of mass.

2.4.1 Reynolds number

The presence of turbulence is determined by Reynolds' dimensionless number. To obtain this value a dimensionless parameter is used, which is called Reynolds' dimensionless number and flows more than 2100 Reynolds number were considered as turbulent streams. Reynolds number is represented by the ratio of inertial forces to viscous forces acting on a fluid element. Reynolds numbers of the OBS water flow can be obtained using equation 2.11(Hadad and Ghaderi, 2015):

$$\mathrm{Re} = \frac{\rho u d}{\mu} \qquad \textbf{(2.11)}$$

Where ρ is the fluid density ($water = \ 998.2\ kg/m^3$), u is the velocity magnitude, d is the hydraulic depth of the OBS and μ is the dynamic viscosity of the water

($1 \times 10^{-3} kg/ms$). The initial prediction of the Reynolds number value is important in order to properly simulate the OBS water flow. The flow will be defined as laminar for $Re < 2100$ and turbulent for$Re > 2100$.

2.4.2 Turbulence model

There are many choices of turbulence models provided by Fluent as summarized in Figure 2.6. Based on the Figure 2.6, there are more than seven different turbulence models are available. The selection of the model is depending on case studies. Unfortunately, there is no single turbulence model is generally acknowledged as being advanced for all classes of problems. In order to choose the most suitable selection of model for CFD application, the capabilities and limitations of the various options need to be understood. Comparison of turbulence models is given in Table.2.6. Among all the turbulence models, standard k-epsilon is the simplest, well-established in academic and industrial application. Even though it also has some limitations (as described in the Table 2.6) it is still the most economical turbulence model in terms of computational effort and satisfactory accuracy in diverse turbulent flow issues.

2.4.3 Multiphase model

Multiphase flow regimes can be categorized into four groups as follows: gas-liquid or liquid-liquid flows; gas-solid flows; liquid-solid flows; and three-phase flows. In

Fluent, two main approaches are available for the numerical calculation of multiphase flows. They are Euler-Lagrange approach and Euler-Euler approach. Three different Euler-Euler models are accessible in Fluent. These include the volume of fluid (VOF) model, mixture model and Eulerian model. The details of the comparison of these models are given in Table 2.7.Based on the Table 2.7, the differences between the multiphase model are depending on the modelling concept and their applicability. Each model has its own advantage and disadvantages. Among the entire multiphase model, the VOF model has the capability to model each phase as separate fluid but at the same time can still track the interface between the phases.

VOF approach is applied to track the moving free surface of the incompressible viscous flow. The VOF method has also been used to calculate a transient solution of the open channel flow. Apart of solving a momentum equation for each phase, the interface is treated as an Eulerian variation, where the secondary phase does not disperse within the primary phase. Solution of a continuity equation for the volume fraction of one (or more) of the phases has been used to accomplish the interface between the phases. Based on Chapter 23 Fluent's user's guide, the following continuity equation has been used for the q^{th}phase.

$$\frac{1}{\rho_q}\left[\frac{\partial}{\partial_t}\left(\alpha_q\rho_q\right) + \nabla.\left(\alpha_q\rho_q\vec{v}_q\right) = S_{\alpha q} + \sum_{p=1}^{n}\left(m_{pq} - m_{qp}\right)\right] \quad \textbf{(2.12)}$$

Where;

m_{pq} indicates the mass transfer from phase p to phase q.

m_{qp}indicates the mass transfer from phase q to phase p

By default, the source term $S_{\alpha_q}$is equal to zero. The equation of volume fraction will not be solved for the primary phase. The primary phase volume fraction will be computed based on:

$$\sum_p^n = 1\ \alpha_q = 1 \qquad \textbf{(2.13)}$$

$$\frac{\partial \alpha_q}{\partial_t} + \nabla.\left(\theta.\alpha_q\right) = 0 \qquad \textbf{(2.14)}$$

Where α_q is the volume fraction of the q^{th} phase

If α_q = 0, the cell is empty.

If α_q = 1, the cell is full.

If 0< α_q< 1, the cell contains the interface between theq^{th} phases.

By means of the presence of the phases, the properties involved in the transport equations are determined in each of the control volumes. In a two-phase modelling, if the phases are represented by the subscripts 1 and 2, the mixture density in each cell is given by:

$$\rho = \alpha_2\rho_2 + (1 - \alpha_2)\rho_1 \qquad \textbf{(2.15)}$$

Generally, for n phase system, the volume-fraction-averaged density is represented by:

$$\rho = \sum \alpha_q \rho_q \qquad \textbf{(2.16)}$$

All other properties including viscosity are also calculated using the same method. A single momentum equation is solved throughout the domain, and the resulting velocity field is shared among the phases. The momentum equation, shown below, is dependent on the volume fractions of all phases through the properties ρ and μ.

$$\frac{\partial}{\partial_t}(\rho\vec{v}) + \nabla.(\rho\vec{v}\vec{v}) = -\nabla\rho + \nabla.[\mu(\nabla\vec{v} + \nabla\vec{v}^T)] + \rho\vec{g} + \vec{F} \qquad \textbf{(2.17)}$$

2.4.4 Discrete phase model

Apart from multiphase modelling, particle tracking is a beneficial means to simulate RTDs in the process of wastewater treatment. Nevertheless, the major problem is the number of particles being tracked within the simulated system and in the case of a full-scale AS tank, tracking too many particles will result in extensive computational times. Therefore, when comparing capability of different approaches in multiphase modelling, discrete phase model is the most expensive, concerning long computational times and involving a large number of CPUs (central processing Units) so restraining its popularity in the simulation of wastewater treatment systems (Karpinska and Bridgeman, 2016).

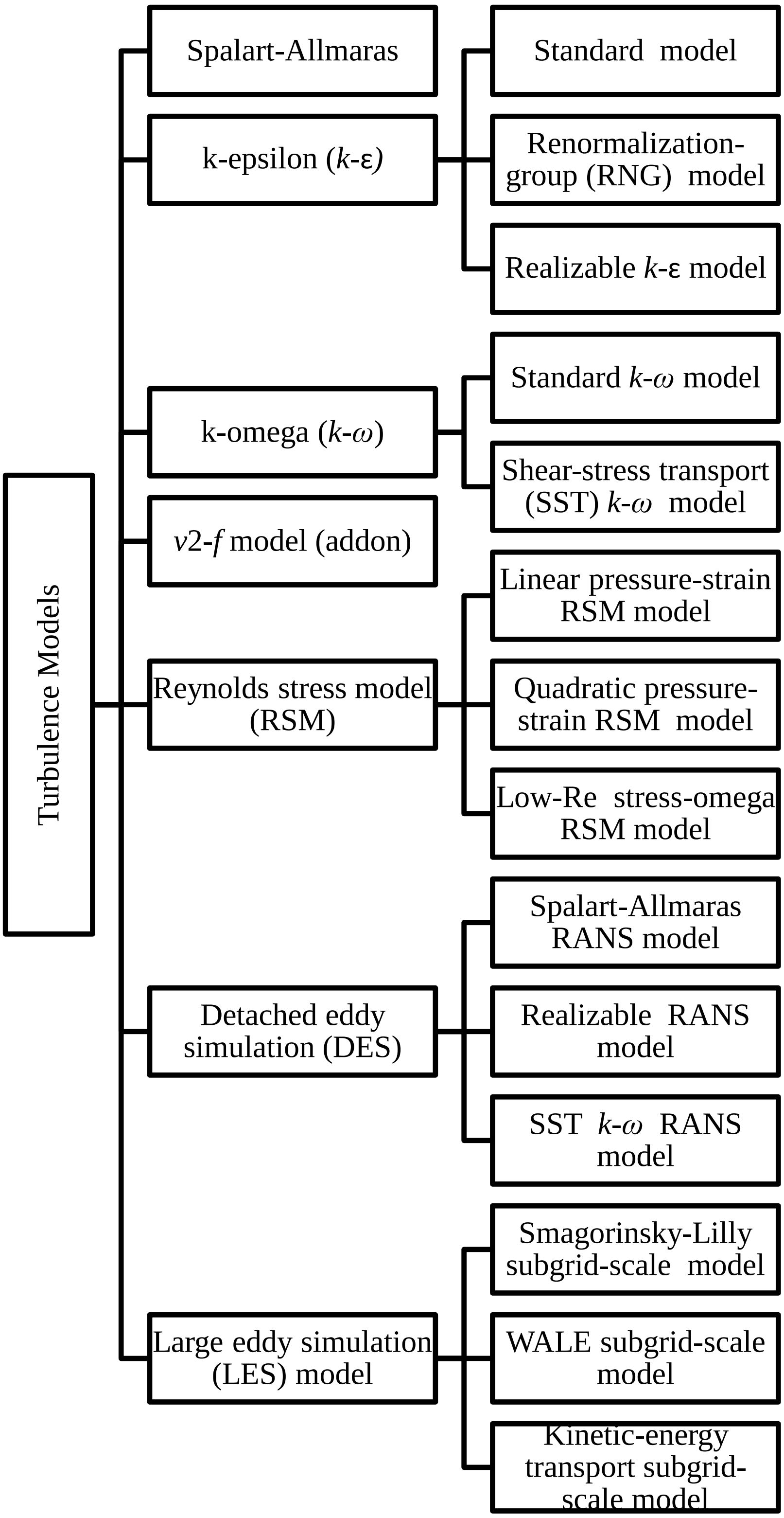


Figure 2.6: Turbulence models provided by Fluent

Table 2.6: Comparison of different turbulence models(Karpinska and Bridgeman, 2016)

Model	Advantages	Disadvantages
Standard *k*-ε (s*k*- ε)	• Simplest and complete turbulence model. • Excellent performance for many flows. • Well established in academia and industry. • Robust, economic in terms of computational effort and satisfactory accuracy in diverse turbulent flow issues. •	• Poor performance in some scenarios (strong streamline curvature, flow separation, adverse pressure gradients). • Assumes locally isotropic turbulence. • Poor prediction of the lateral expansion in 3D wall jets.
Renormalized Group (RNG) *k*- ε	• Improved performance for swirling and high-strained flows compared to sk-ε	• Less stable than s*k*- ε
Realizable *k*- ε	• Suited for planar and rounded jets, swirling and separating flows and wall-bounded flows with strong adverse pressure gradients.	• Not recommended to use with multiple reference frames.
Standard *k-ω*	• Valid throughout to boundary layer, subject to fine grid resolution. • Accounts for the stream-wise pressure gradients. • Applicable for detached separated flows and fully turbulent flows.	• Pressure induced separation is typically predicted to be excessive and early.
Shear Stress Transport (SST) *k-ω*	• The most accurate from two-equation eddy viscosity models. • Suitable for adverse pressure gradients and pressure-induced flow separation. • Accounts for the transport of the principal shear stresses.	• Less suitable for free shear flows.
Reynolds Stress Model (RSM)	• Accurate calculation of the mean flow properties and all Reynolds Stresses. • Accounts for the streamline curvature, rotation and rapid changes in strain rate yielding superior results to two-equation models for complex flows, e.g. with stagnation points.	• Computationally expensive. • Not always more accurate than two-equation models. • Harder to obtain converged result.

Source:(Karpinska and Bridgeman, 2016).

Table 2.7: Comparison of different multiphase models(Karpinska and Bridgeman, 2016)

Model	Concept	Modelling	Applicability	Issues
Eulerian-Lagrangian	• Liquid phase treated with Eulerian approach. Every particle (bubble) tracked along trajectory.	• A set of averaged NS equations for the liquid phase. A set of Newton's 2nd Law (N2L) equations applied to all particles. Momentum transfer terms in both NS and N2L to be modelled.	• Flows where at least one phase is clearly dispersed into a principal phase. • Particles smaller than mesh cell size.	• Computational expense not a priori computable, and proportional to the particles number. May be prohibitive for high particle numbers.
Eulerian	• Each phase modelled as a separate fluid	• A set of averaged, volume fraction-weighted Nvier-Strokes (NS) equations per phase. Momentum transfer terms and constitutive equations to be modelled	• Theoretically, every type of flow; depending on the additional terms' modelization.	• Additional terms' modelization is determinant, but their modelling is difficult.
VOF	• Each phase modelled as a separate fluid. The interface between the phases is tracked.	• Interface tracked via a continuity equation and the domains of the single phases are defined. • A set of phase-specific NS equations with momentum exchange terms is solved for each domain.	• Flows where the interphase surface is clearly defined (e.g. a single, large bubble inside a liquid).	• Inapplicable if the interphase surface is too complex.
Mixture	• Both phases treated as a whole.	• Single set of NS equations. • Effective mixture density and viscosity to be modelled.	• Homogeneous fluids or non-homogenous fluids that are treated as homogeneous.	• Inapplicable in every case in which there is clear distintion between the phases.

Source: (Karpinska and Bridgeman, 2016)

2.5 Application of CFD modelling in Wastewater Treatment Processes

Initially, CFD was almost exclusively associated with aerospace and mechanical sectors, but in the last two decades, CFD application has been extended to the civil and environmental industries (Karpinska and Bridgeman, 2016). CFD models have been recognized as an efficient and useful means for evaluation of hydraulic problems at WWTPs (Laurent et al., 2014). CFD is a computer-based mathematical modelling tool that incorporates; 1) the solution of the fundamental equations of fluid flow, such as continuity, momentum and energy(Metcalf & Eddy, 2013), 2) the Navier-Stokes equations and 3)other allied equations (Reddy, 2003). Improvement in the speed and memory of computers resulted in increased use of CFD as a tool to understand, troubleshoot and optimize the unit processes of WWTP. This development coupled with the development of accurate and efficient numerical algorithms for solving problems on the computers, made it possible to simulate complex flows that are not easily observable in an experimental set-up (Brannock et al., 2002). One of the advantages of CFD application is its potential to predict the behaviour of the subject being studied towards different configurations (Abbas et al., 2006).

CFD is widely used within the wastewater treatment industry (Essemiani et al., 2004, Harwood, 2006, Anon, 2001). CFD is mainly used to study the hydraulic behaviour within the wastewater treatment system and to model the fluid flow patterns in order to achieve a better understanding of the treatment plant.CFD application is currently significant for some purposes such as design, evaluation, and optimisation of wastewater treatment system. Modern wastewater treatment equipment is progressively being designed with the assistance of CFD (Nisipeanu and Harwood, 2002). CFD analysis is used to predict the performance of units before real implementation of the design (Brouckaert and Buckley, 1999) and to troubleshoot and confirm efficient designs (Knatz, 2005).

CFD is also applied to existing processes for a variety of reasons. One of the simplest uses is the assessment of unit operation performance at different process conditions (e.g. different flow rates, different oxygen transfer rates, etc.). CFD analysis is applied by some researchers for various unit operations and processes of wastewater treatment, including the aeration and mixing process (Fan et al., 2010, Littleton et al., 2007a, Fayolle et al., 2007, Joshi and Kumaresan, 2006, Ortiz and Ducoste, 2004, De Kretser et al., 2003), settling tank (Stamou, 2006, Jensen et al., 2006, Kowalski et al., 2003), activated sludge process(Le Moullec et al., 2011, Karama et al., 1999, Alex et al., 2002), nutrient removal process(An et al., 2007, Littleton et al., 2007b), disinfection process(Reddy, 2003, Santoro et al., 2005), microbial reaction(Oda et al., 2006) and pond systems(Shilton and Mara, 2005, Sweeney et al., 2003, Wood et al., 1995). CFD analysis allows the researchers the flexibility to modify any of the conditions associated with the unit without disrupting actual production. Modification of the existing operational system due to the CFD application may possibly lead to an optimization of the system.

2.5.1 Case studies of previous CFD models

Previously, CFD application in wastewater industry was mostly explored in most of developed countries due to the keenness of improving the existing wastewater treatment system. At the beginning, CFD analysis was only applied for the purpose of understanding the basic flow patterns. According to Oda and his co-authors (2006), one dimensional (1D) CFD models were used since the late 1970s. As the results of the huge improvement in computational technology as well as the increasing confidence of CFD application, 2D and 3D CFD techniques were broadened into a range of wastewater treatment units' operations. One of the studies was 2D CFD modelling of wastewater ponds in Australia, evaluating the hydrodynamics of pond systems (Wood et al., 1995). In 1999, a study was conducted by Karama and his colleagues, for evaluating the efficiency of an activated sludge reactor using the CFD technique. The main objective of the research was to study the performance of an anaerobic zone in the activated sludge reactor (Karama et al., 1999) .

During the last decade, many papers on the CFD application of wastewater treatment industry were published, especially in the area of biological treatment. The principles and benefits of CFD application are always being highlighted and shared by the researchers through their publications such as Anon (2001), Knatz (2005) and Harwood (2006). Biological processes, which involve the mixing and aeration devices are mostly explored for the CFD application. This probably due to the complexity and significance of the biological process and its high operation and maintenance costs. In 2004, Ortiz and Ducoste have conducted a study to evaluate the CFD application for analyzing the mixing effectiveness of low-energy mixers.

CFD technique for optimisation of microbial reaction and sludge flow has been developed by Oda and his colleagues (2006).These 3D multiphase models were applied for the optimisation of an intermittent agitation in anoxic reactors by coarse bubbles (Oda et al., 2006). Four different numerical models including the oxygen transfer model of coarse bubbles, a model of fine bubbles, activated sludge model to simulate microbial reaction and a sludge settling model were added to the solver. Even though the study proposed some optimum conditions of the aeration process for the particular reactor, but the authors also mentioned that these optimum conditions were changeable because they were depending on the shape of the tanks and some other factors. In other words, the proposed optimum conditions were only applied to the particular reactor that was involved in the research. It suggested that in the process of optimization of any biological reactor, specific case studies need to be performed for the particular reactor.

Glover and his co-authors (2006) evaluated the value of CFD model and Activated Sludge Model (ASM) and the potential of combining both models. In the study, both models were applied to pilot plant and a full scale OD system as testing scenario (Glover et al., 2006).CFD model was used to study the hydrodynamic behaviour and the ASM was used to simulate the nutrient removal. The study was based on two potential approaches, including the potential of adding biological model equations in the CFD code and the potential of relating the outputs obtained from CFD analysis to the biological model, keeping the two models as a separate component (Glover et al., 2006). The researchers used the k-ε turbulence model to represent the turbulent flow energy caused by aeration and agitation, and the dissipation of this energy throughout the reactor with the standard wall conditions and functions(Glover et al., 2006). One of the research findings stated that the conventional biological modelling can gain huge benefits of indirectly using the CFD results in order to acquire hydraulic structure of the reactor, particularly in troubleshooting configurations(Glover et al., 2006). It explains that CFD application can contribute to a better understanding of the biological reactor even though with only hydraulic analysis outputs.

Another study involved CFD application of biological reactor was performed by Jensen and his colleagues in year 2006. The objective of CFD application was to improve the design of less effective aeration tank settling operation at Lundtofte wastewater treatment plant in Denmark. Two phases 3D model was developed to represent the tank. The standard k-ε model was used to model the turbulence. The domain of the model was meshed into 216 000 cells using tetrahedral meshes. The mixers in the tank were modelled by adding a directional source of momentum in subdomains based on the real location of the mixers in the tank (Jensen et al., 2006).According to the authors, the outputs given by CFD model in the research can only be interpreted qualitatively and not quantitatively because of the settling experiment was not performed for the particular treatment plant. The model can only describe the flow pattern, but not able to provide accurate sludge concentrations in the entire tank volume (Jensen et al., 2006). However, the produced model was able to describe the main features during aeration tank settling and possible to be used for the improvement of the tank.

Some authors made further steps to expand the use of CFD in the wastewater industry. A few studies were performed to understand the nutrients removal processes in biological reactor. These include the application of CFD to study the efficiency of nitrogen removal (An et al., 2007) and to simulate phosphorus removal in the biological reactor (Littleton et al., 2007b).Young Ann and his colleagues performed the CFD analysis of the rotating distributor in the up flow multi-layer bioreactor (UMBR). UMBR was patented as pre-anoxic tank and was applied to a pilot-scale plant. 2D CFD model was developed with Fluent version 5.4 and the model was set up using the turbulence and k-ε model. The CFD model was created based on the total influent flow rate as 100 cubic meter m^3 per day. The domain was meshed for about 3,000 elements. The aim of CFD application in the case was to design and configure the rotating distributors in the UMBR. As mentioned by the authors, it was understood that unequal distribution through the distributors leads to the short-circuits or non-ideal flow, which can adversely affect the efficiency of the nutrients removal(An et al., 2007). Velocity vectors and velocity contours obtained from the CFD analysis visualized the distribution observed at each outlet port. In the study, CFD model was complementing a pilot-scale plant and CFD was complemented by some measurements such as BOD5, SS, total nitrogen, total Kjeldahl nitrogen, etc. From the study, it clearly shows that CFD model is reliable to complement other methods of study to achieve an optimum design and configurations of the system being studied.

In the cases studied by Littleton and his colleagues, CFD was not only used to complement other methods, but it was also used to integrate other methods in its analyses. In the first case study, CFD was used to characterize and simulate the flow pattern and oxygen transfer of a full-scale, closed loop bioreactor (Littleton et al., 2007a). CFD model was established by imparting the known momentum which calculated by tank fluid velocity and mass flowrate of the fluid at the aeration region (Littleton et al., 2007a). The results given by CFD analyses were validated with the field data. After that, Activated Sludge Model No. 2 (ASM2) that representing the biochemistry rate expressions for general heterotrophs and phosphorus-accumulating organisms (PAOs) was introduced to a 3D CFD model (Littleton et al., 2007b).

In 2011, there was a study of CFD application on two operating conditions of a full scale OD for optimization of energy consumption. This study was performed by Yang and his colleagues in Henan, China. A 3D CFD model was developed to predict flow pattern and oxygen mass transfer in the full-scale Carrousel oxidation ditches. Energy consumption of the existing biological treatment system was compared to the improved (involved CFD application) system.

According to the cases as discussed above, it is noticed that CFD was applied broadly through different continents, different times and different processes of wastewater industry. In conclusion, as the knowledge capacity and confidence of CFD application increased, more biological reactors were chosen as case studies. The application was continued for the assessment and optimization of more complicated systems such as conventional and modified ODas well as BNR systems. Glover et al. (2006) used CFD to model a full-scale oblong OD as a case study. However, the OD was assumed as a batch process, even though in reality it is operated in a continuous mode. Littleton et al. (2007) used CFD to model the flow and oxygen concentrations of the outer channel of an OD and to simulate the removal of biological phosphorus. Even though their research does not represent the details of the processes that occur in each channel of the ditch, the research increased the confidence that simplified mixing processes can be successfully modelled using CFD. Aeration and mixing processes are always related to high operational costs due to the high energy consumption. Lately,CFD is not simply used to design new equipment and assess performance of the existing treatment plant, but it has been widely used for the optimization purposes including the optimization of energy consumption (Bosma and Reitsma, 2008).

Currently, CFD application is well established not just among the researchers in the developed countries, but also in the developing countries. Many case studies from China (Yang et al., 2011, Yang et al., 2010b, Luo et al., 2005, Guo et al., 2013, Xie et al., 2011) involve the application of CFD for the optimization of the biological treatment plant. The most recent published papers on CFD application in China, which related to OD are written by Xie and his colleagues and also published by Lei and Nie. These papers are established to stimulate the hydrodynamics and processes occurred in an OD (Xie et al., 2014, Lei and Ni, 2014). Although the CFD modelling of biological treatment systems has taken place for over 15 years now(Karpinska and Bridgeman, 2016), there are number of concerns which still unresolved and which, if successfully be surmounted, would improve model reliability and stability more. There are a number of areas in modelling practice, which remain uncertain and call for additional efforts to study the complexity of the fluid flow and oxygen mass transfer in the biological treatment systems. One of the future needs of CFD application is for it to be used for parametric studies(Lei and Ni, 2014), to recommend parameters for evaluation (i.e. aeration discs), characterize the parameter range, identify the design limitations, and examine the results of each parameter disparity.The last five years research on CFD application in biological treatment systems is summarized and discussed in Table 2.8. All these researches have established the reliability of CFD application in biological treatment studies.

Table 2.8: Summary of CFD application in biological treatment modelling

Reference	Observation of the research
Karpinska & Bridgeman, 2016	A critical review of CFD modelling of activated sludge systems. It discussed the rationale behind the CFD application to model aeration, to facilitate the improvement of treatment efficiency and reduction of energy usage. It highlighted some CFD applications in single and multiphase modelling. According to the authors, the use of gas-liquid CFD model enables fast and straight forward prediction of the multiphase velocity field induced by the aerators and mixers. It mentioned the problems of modelling assumptions used in the evaluation of the mixing pattern and mass transfer in an aeration tank. It emphasized on the unaddressed modelling issues including the coupling of the AS tank with the secondary clarifier.
Hadad & Ghaderi, 2015	It explored the simulation of the flow pattern of the aeration tank using the Fluent programme. It called attention to the ability of Fluent program in simulating the two phases' fluids flow. Based on the research, the authors suggested that the ratio of aeration to the tank area in the wastewater treatment system should be increased.
Xie et al., 2014	It involved simulation of the flow field and sludge settling in an oxidation ditch via a two-phase (liquid-solid) CFD model. One interesting finding of the research was the comparison between single-phase and two-phase simulations. Compared to the single-phase simulation, the relative error between the simulation results and field data in the two-phase model was decreased from 8% to 5%.
Laurent et al., 2014	It studied a practice for CFD application as a supportive tool for wastewater treatment modelling. It suggested further studies in biological treatment unit processes caused by the multifaceted intersection between potential macro and micro scale reactions that occur outside and inside biological floc particles. According to the authors, if the conventional model fails, e.g. in case of dynamic flows and significant variations in the contamination, the strength of CFD methods is evident.

Lei & Ni, 2014	3D three-phase model was developed for the simulation of hydrodynamics, oxygen mass transfer, carbon oxidation, nitrification and denitrification processes in an oxidation ditch. The numerical predictions of flow field in the OD showed the great importance of the impellers and stirrers in promoting mixing. The findings of the research have recommended that CFD model should be used for parametric studies in order to help the selection of optimum arrangements of impellers or aeration devices in ODs.
Guo et al., 2013	Through CFD application, the flow field and DO distribution inside the outer channel of the Orbal oxidation ditch in Beijing were monitored under the actual operation conditions. The results of the study revealed that flow velocity and DO were heterogeneously distributed in the outer channel. This heterogeneous DO distribution created anoxic and aerobic zones, which may have facilitated simultaneous nitrification-denitrification in the channel. The findings provide supporting information for rational optimization of the oxidation ditch's performance.
Wu et al., 2012	The submerged propeller is simulated using Fluent software. The findings revealed that the change of submerged propeller installation position could keep away from the condition of back mixing caused by the drives. In addition, the problems of sludge deposit and the low velocity near to the bend could also be solved.
Yang et al. 2011	Performance revelation and evaluation of two operating conditions (existing and improved) were carried out in two full-scale Carrousel ODs at the Ping Dingshan WWTP in Henan, China. Only a steady-state CFD model was developed. It has been highlighted as one of the limitations of the model since the influent flow rate, concentration and composition are expected to vary on a daily basis. It proposes that an optimal operating condition should be developed from a dynamic mode to accommodate the real process. Based on the research, a surface aerator has been suggested to be relocated to around 15m from the curve bend entrance to reduce energy loss.
Le Moullec et al., 2011	It developed an experimental and numerical study of an activated sludge system to validate several types of models including a systemic approach, compartmental modelling and CFD model. However, based on the findings, the other models (not the CFD model) still need to have a more detailed kinetics model for the design of the bioreactors.
Fan et al., 2010	The research was focussed on the hydrodynamics of an oxidation ditch using the experimental and simulation approach. Based on the findings, it was stated that when the speed of the aerators are increased, the velocities were also increased.

2.5.2 Benefits of previous CFD models over simpler models

The use of CFD techniques allows detailed study of the transport phenomena taking place and hence provides insight mixing efficiency, spatial distribution of particles, and chemical concentrations(e.g. dissolved oxygen) (Glover et al., 2006). According to Brouckaert and Buckley (1999), CFD provides many advantages compared with experimental fluid dynamics. Most methods used to design, operate and control WWTPs employ heuristic and empirical techniques (Brannock et al. 2002). CFD makes it possible to model processes that involving fluid flow from a fundamental level. Flow modelling provides insights into the fluid flow problems that would be too expensive or costly by experimental techniques alone.

Another advantage of CFD application is simulating a range of operating conditions to assess performance before designs and operating changes are confirmed (Metcalf & Eddy, 2013). CFD has proven to complement physical modelling and other experimental techniques by providing a detailed look into fluid flow problems, including complex physical processes such as turbulence, chemical reactions, heat and mass transfer(Fluent, 2007). Jensen and his co-authors positively think that CFD capable of providing opportunities for the optimisation of a system. The authors also highlighted the possibility of combining the hydraulics model with the biological and chemical processes.

2.6 Previous Strategies for Reducing Energy Consumption of Wastewater Treatment

Wastewater management approaches are introduced to assess and improve the energy efficiency of wastewater treatment plant, mainly in biological treatment plant. A variety of management tools are used in order to accomplish the goal of energy optimization. These include the application of Life Cycle Assessment (LCA) as one of the tools to assess the environmental impacts of WWTPs. Recently, Zang and his colleagues have published a review of the LCA studies dealing with biological WWTPs, with the aim to give a qualitative interpretation of the associated environmental impact categories, and one of the categories is the energy balance (Zang et al., 2015) . They concluded that LCA is commonly used by the people in wastewater industries to compare and optimize the energy consumption of WWTPs systems.

Other than LCA, there are a few more methods that have been developed to evaluate and optimize the energy usage of wastewater treatment system generally and biological treatment process specifically. These include the application of: 1) Performance Assessment (PA) and Performance Indicator (PI) (Yang et al., 2010a, Poulsen and Hansen, 2009), 2) Environmental Impact Assessment (EIA) (Venkatesh and Brattebo, 2011) 3) fuzzy logic supervisory control system (low-level and high-level controller) (Baroni et al., 2006, Fiter et al., 2005, Serralta et al., 2002, Kalker et al., 1999) and 4) Supervisory Control and Data Acquisition (SCADA) system. One example of PI application that studies the influence of sewage treatment to the overall energy consumption is undertaken by Zhang et al. (2015). They have adopted a set of indicator system based on energy to investigate the influence of sewage treatment on China's energy consumption. they have concluded that the sewage treatment has an increasing impact on the national energy consumption (Zhang et al., 2015).

Some of the main mechanisms of an effective energy management plant are highlighted by the Environmental Protection Agency (2006) including; 1) developing a system to track energy consumption, 2) implementing energy audits of major operations, 3) upgrading equipment and systems, 4) optimizing load profiles by changing operations where possible and 5) providing in-house energy management training for operators. Most of the management systems proposed were only giving some qualitative data and also some qualitative concluding remarks, which might be less constructive for further actions of energy optimization.

Rather than focussing on the wastewater management approach, some researchers attempted to use more proactive approach by proposing an innovative treatment process. More studies were conducted to increase the energy efficiency of the wastewater treatment plant by manipulating the microbial fuel cells in nutrient removal processes (Liu et al., 2011, Lefebvre et al., 2011, Sturm and Lamer, 2011, Neira and Jeison, 2010, Kim et al., 2010). In these cases, energy is viewed as a resource rather than a waste, where the energy supply is obtained from organic matters of wastewater as well as from its thermal content (McCarty et al., 2011).

An energy evaluation of wastewater treatment plant where the algal biomass production is coupled with the nutrient removal processes were studied by(Sturm and Lamer, 2011). Recently, hybrid energy systems for wastewater treatment (wastewater treatment and energy generation are accomplished simultaneously) and bio energy production have gained much attention for pollutant removal of wastewater (Tee et al., 2016). One of the limitations of these treatment systems is its high capital, operating and maintenance costs. All the limitations have caused this approach to be seen as less efficient, especially from the economic perspective.

Numerical analysis technique is currently the most established approach for optimising the energy consumption of wastewater treatment plant. Various numerical simulations are used in order to enhance the performance of treatment plant, particularly in terms of its energy consumption. These include the conventional mathematical modelling and advanced computational modelling. Mathematical models are used for several goals such as design and optimisation at different levels. Some mathematical models which have made a most important contribution of promoting numerical analyses in wastewater treatment are Activated Sludge Models (ASMs) (Oda et al., 2006). These models are capable of modelling at the large-scale level, while some advanced computational models are usually applied for more detailed study (Glover et al., 2006).

2.7 A summary of literature review

One of the main concerns related to the wastewater treatment system is the conflict between application of more stringent effluent discharge standards and optimization of energy consumption. Energy consumption for wastewater treatment is expected to boost up in the future as a result of increasing population, more stringent discharge requirements, and aging infrastructure. Based on the literature review, biological treatment system has consumed the highest amount of the total energy usage of wastewater treatment plant. Pressures to reduce the costs associated with energy consumption have led to the development of various approaches. CFD has gained popularity over conventional wastewater treatment modelling methods, as it is a high-precision technique allowing assessment of the treatment systems, which are costly, complicated and sometimes unsafe to reproduce in laboratory or pilot scale.

One of the gaps in the application of CFD in biological treatment system is the modelling of unsteady flow conditions in multiphase and open channel system with the incorporation of discrete phase model. This research may enhance the CFD simulations of an open channel system (2 phases) with the incorporation of discrete phase model. In view of the fact that the simulation of discrete phase modelling is very limited since it involves long computational time and requires a large number of CPUs, this study is considered another significant contribution to the overall CFD application. Another benefit of CFD as a computational modelling technique is the influential visualization competency permitting comprehensive description of the local-scale phenomena in varying operating conditions.

CHAPTER THREE

RESEARCH METHODOLOGY

This chapter presents the methods and procedures involved in order to develop a preliminary process model and CFD model.

3.1 Model Development

Two different approaches have been used in order to discover the link between oxygen utilisation rate (OUR) and biological reactions inside the OBS. These include the development of preliminary process model as initial study and development of CFD model as an advanced study.

3.1.1 Fundamental bases for the model

Through the application of previously developed fundamental relationships describing the oxygen uptake by biological processes and the power required to supply oxygen to water (referred as Equation (3.1) and Equation (3.2), respectively), the correlation between oxygen utilisation rate, BOD reduction and power consumption was established.

3.1.1.1 Oxygen consumption by biological treatment processes

A simplified model for the biological treatment process of OBS is as described in Figure 3.1.

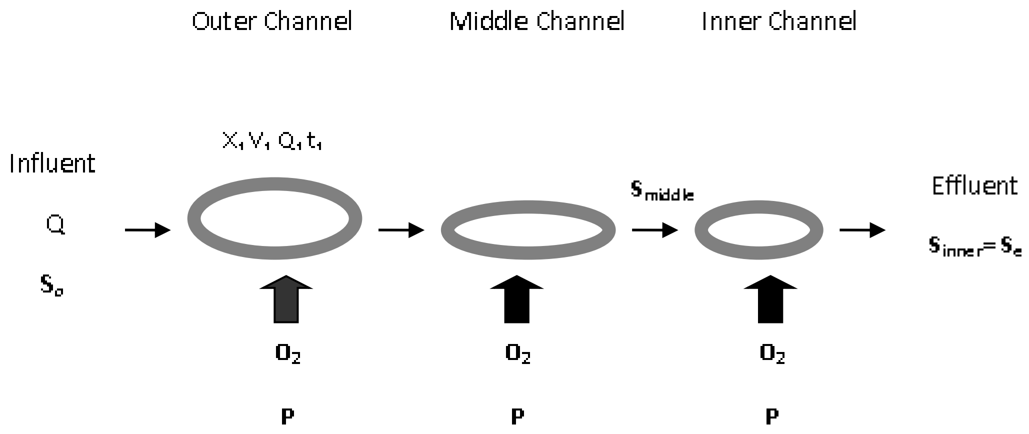


Figure 3.1: Simplified model for the biological treatment process of the OBS

Equation 3.1(Barnes et al., 1983, Sperling and Chernicharo, 2005b) was used to demonstrate the correlation between oxygen utilisation rates and BOD removal. The fundamental equation below (refer to Figure 3.2) is commonly used as a basic formula to estimate the total oxygen requirement in any biological treatment process (originally developed to describe the activated sludge process (Sperling and Chernicharo, 2005b). The total oxygen utilisation rate can be calculated based on the oxygen demand for the oxidation of the carbonaceous and nitrogenous organic matter, oxygen demand for endogenous respiration and oxygen saving due to denitrification (since denitrification releases oxygen from nitrate).

$$\mathrm{OUR}\ \left(\frac{kg}{d}\right) = \frac{aQ(S_o - s_e)}{10^3} + \frac{bX_vV}{10^3} + \frac{4.57Q(\Delta TKN)}{10^3} - \frac{2.86Q(\Delta TN)}{10^3} \qquad \textbf{(3.1)}$$

where,

OUR= oxygen utilization rate or also referred as oxygen requirement (kg/d)

Q= influent flow (m^3/d)

S_e=effluent BOD_5 concentration (soluble BOD) (g/ m^3)

S_o =influent BOD_5 concentration (total BOD) (g/m^3)

Xv= Mixed Liquor Volatile Suspended Solids, MLVSS (g/ m^3)

V= Tank volume (m^3)

$\Delta TKN = TKN_{influent} - TKN_{effluent}$ (g/m^3)

TKN= Total kjeldahl nitrogen, equal to organic nitrogen and the ammonia nitrogen

$\Delta TN = TN_{influent} - TN_{effluent}$ (g/m^3)

TN= Total nitrogen

a= 1.46-1.42Y

b= $1.42 f_b K_d$

1.46=conversion factor (BOD_u/BOD_5)

Y= yield coefficient (gVSS produced per gBOD removed) ($gX_{v\,/g}BOD_5$)

$f_{b=}\ f_{b'} / [1+(1- f_{b'})K_d\ \Theta_c]$

f_b =biodegrable fraction of MLVSS generated in a system subjected to a sludge age Θ_c (X_b/X_v)

$f_{b'}$=biodegrable fraction of the VSS immediately after its generation in the system, that is, with Θ_c =0. This value is typically equal to 0.8 (80%)

Θ_c =Sludge sge (d)

K_d =endogenous respiration coefficient (d^{-1})

3.1.1.2 Linking oxygen requirements to power consumption

Equation (3.2) is used to calculate the total power consumption by the aeration devices. This formula is used to correlate the total oxygen consumption to the total power consumption of the OD.

$$P = \frac{OUR}{\eta_p} \quad \textbf{(3.2)}$$

Where,

P= Power consumption in kW

OUR= oxygen requirement in kg/hr

$\eta_{p=}$ in-process oxygenation efficiency for mechanical aeration system in kg O_2/ kW.hr

Aeration disc is considered as a 'mechanical backbone of the OBS because of its high oxygen transfer efficiency and unmatched mixing efficiency'(Technologies, 2007). In mechanical aerators, the submergence of the discs in relation to the water level is a very important aspect in terms of oxygen transfer and energy consumption (Sperling and Chernicharo 2005). In process oxygenation efficiency for mechanical aeration system is represented by a coefficient known as $\eta_{p.}$ For the purpose of this research, the value for coefficient η_p is obtained from the design specification provided by IWK. The value is given as 0.9.

3.1.1.3 Justification for the assumed coefficients

Equation (3.1) also involved the use of the coefficients a and b, which are calculated based on the Equations (3.3) and (3.4) as shown below.

$$a = 1.46 - 1.42Y \qquad \textbf{(3.3)}$$

1.46=conversion factor (BOD_u/BOD_5)

Y= yield coefficient (gVSS produced per gBOD removed) ($gX_{v\,/g}BOD_5$)

The oxygen consumption for the oxidation of carbonaceous organic matter is representing the oxygen consumption for the ultimate BOD ($BOD_{u)}$ removed by the system (Sperling and Chernicharo 2005). The ultimate BOD is referring to the total oxygen consumption for substrate oxidation and endogenous respiration. In order to get the value of the ultimate BOD, the value of BOD_5 has to be multiplied by a conversion factor. The value of the conversion factor for the domestic sewage is in the range of 1.2 to 1.6 (Sperling and Chernicharo 2005) and the typical value of the conversion factor (BOD_u/BOD_5), for the domestic sewage is 1.46 (Sperling and Chernicharo 2005).

In a real biological reactor, the organic matter of the influent is not only oxidized but also transformed into new cells. Based on mass equivalent principles, in a steady state system, the cells produced are equal to the cells wasted from the system (Sperling and Chernicharo 2005). Because of this reason, the total oxygen consumption has to be discounted based on the fraction corresponding to the oxygen consumed by these cells, which will not be consumed inside the system. As mentioned by Sperling and Chernicharo (2005) in their book, each 1 g of cells consumes 1.42 g of oxygen for its stabilisation. Y is a yield coefficient, which is representing the fraction of the amount of volatile suspended solid produced per amount of BOD removed. The value of coefficient b is calculated based on the Equation (3.4) as written below.

$$b = 1.42 f_b K_d \qquad \textbf{(3.4)}$$

$$f_b = \frac{f_{b'}}{[1+(1-f_{b'})K_d \theta_c]} \qquad \textbf{(3.5)}$$

f_b =biodegradable fraction of MLVSS generated in a system subjected to a sludge age

$$\theta_c = \left(\frac{X_b}{X_v}\right) \qquad \textbf{(3.6)}$$

$f_{b'}$=biodegradable fraction of the VSS immediately after its generation in the system, that is, with Θ_c =0. This value is typically equal to 0.8 (80%)

Θ_c =Sludge age (d)

K_d =endogenous respiration coefficient (d^{-1})

The coefficient b is a function of f_b, which is indirectly related to the sludge age (Θc). The higher sludge age range, is less sensitive to the values of the variables Y and $K_{d.}$ The value of variable Y is ranged from 0.5 to 0.7, while the value of variable $K_{d.}$ is ranged from 0.09 to 0.07. Table 3.1 shows the standard assumptions of variable Y and $K_{d'}$(Sperling and Chernicharo, 2005b).

Table 3.1 : The values of variable Y and $K_{d.}$

Variable Y (g/g)	Variable $K_{d.}$ (d^{-1})
0.5	0.09
0.6	0.08
0.7	0.07

In this research the value of variable Y in used is equal to 0.6 while the value of $K_{d.}$ used is equal to 0.08. These values are chosen because they are the average value. However, the sensitivity analyses of these coefficients were performed and discussed in the subsection 4.2.

The application of Equation 3.1 in the research also based on some assumptions as stated below:

- The oxygen utilisation rate for the ultimate BOD removed by the system is the same as the carbonaceous demand (including the oxidation of the substrate and for the endogenous respiration of the biomass).
- The oxygen concentration is constant in each channel which remains unchanged through space and time.

- TKN represents the total influent nitrogen. TKN is the nitrogen potentially oxidisable to nitrate.
- 1 g TKN requires 4.57 g O_2 for conversion to nitrate. Stoichiometrically, in overall nitrification reaction, '1 mol of ammonia-N requires 2 moles of oxygen for its oxidation (MW of N=14 g/mol; MW of O2 =64 g/mol; 64/14= 4.57)' (Sperling and Chernicharo, 2005b)
- Nitrification takes place systematically. It is assumed that where there is dissolved oxygen available (aerobic conditions), ammonia will be oxidized to nitrite and then to nitrate.
- Denitrification occurs without oxygen consumption.
- The reduction of $1 g/m^3$ of nitrogen in the form of nitrate releases 2.86 g O_2/m^3 (Sperling and Chernicharo, 2005b)
- The value of Y= 0.6
- Sludge age, Θ_c = 17 days
- $f_{b'}$= 0.8
- K_d =0.08
- HRT for each channel is constant (no distributions).
- Flow (Q) is given by V/t (i.e. plug flow assumption)
-

Most of the assumptions are based on the justification and explanation made by Sperling and his colleagues in their book published in 2005. There are also some assumptions based on other publication (Barnes et al. 1983).

3.1.2 CFD modelling framework

The flow and hydraulic residence time in a biological reactor such as OBS is complex and usually cannot be accurately represented by simple techniques such as this current process model. In order to get a clearer picture of the real conditions inside the OBS, it is necessary to attempt to model the distribution of OUR throughout the channels. This objective can be achieved by using more advanced modelling techniques that potentially gives a better description of the system. In this case, the CFD was chosen as the tool to extend the preliminary process model.

In general, CFD model development involves at least five major steps as shown in Figure 3.2. Initially, all necessary information which defining the problem was assigned, including the geometry, computational grid properties, model definition, number of phases involved, time step and schemes of the numerical. In this case, the model development was initialised with the application of GAMBIT (a sub-component of FLUENT software) for the purpose of building the geometry model (similar to CAD). The geometry model of the OBS was created based on the design layout and dimensions provided by IWK. The details of the basin and discs dimension as well as the design parameter of OBS are listed in Appendix A. In order to create the computational geometry of the fluid flow region, a consistent framework of axes coordinates was applied. Basically, x axis was corresponded to the fluid flow direction. Y axis indicated the width of the channels, whereas the Z axis was representing the depth of the fluid inside the ditch.

After the development of 2D geometrical model was completed, the meshing or grid generation procedure was started. Mesh construction involved discretizing or subdividing the geometry into a number of cells or elements where the variables will be numerically computed. At this step, the computational domain was split into a number of smaller, non-overlapping domains. Using Cartesian coordinate system, the fluid flow governing equations such as the equations of continuity and momentum were solved based on the domain discretization A grid independency analysis was performed for OBS computational model. The grid independence study was conducted for different size of mesh. The results of this grid independency test were given the grid that was independent of any effects due to the grid size.

Specifying appropriate boundary conditions at the domain boundary was the next step after grid generation. Geometry creation, meshing and boundary conditions setup were classified as pre-processing stage. Pre-processing works were performed before solver and post-processing stages took place. Computational analysis was considered as the solver stage, where the case which had been set up at the pre-processing stage was computed or iterated using the Fluent software. Fluent software was also applied for the post-processing works, where the CFD results were visualized, examined and processed for further analysis. The details of CFD model development's procedures are illustrated in Figure 3.3

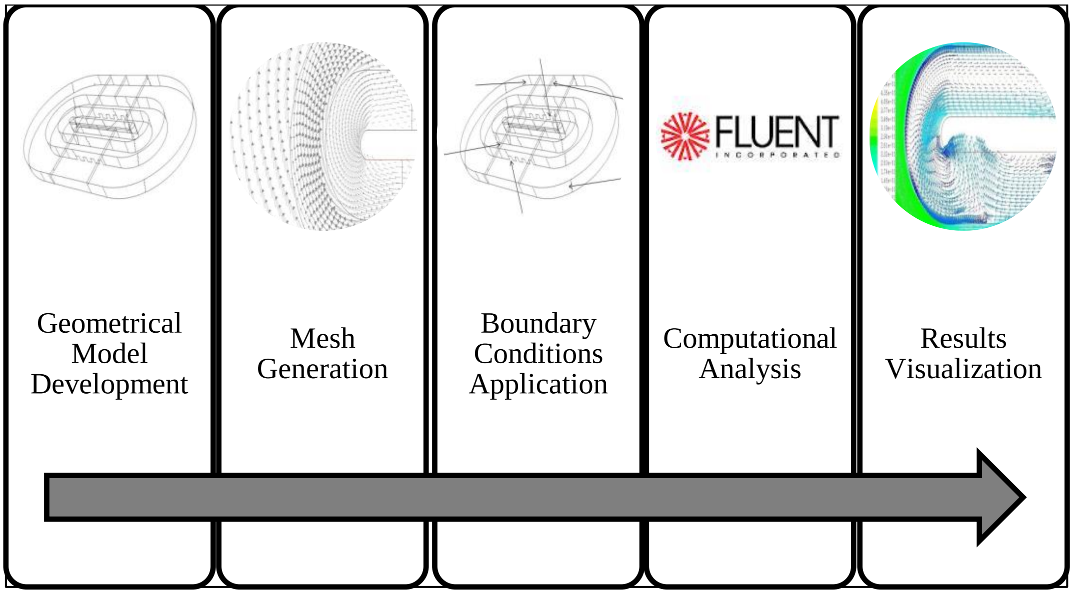


Figure 3.2: Five major steps of CFD simulation

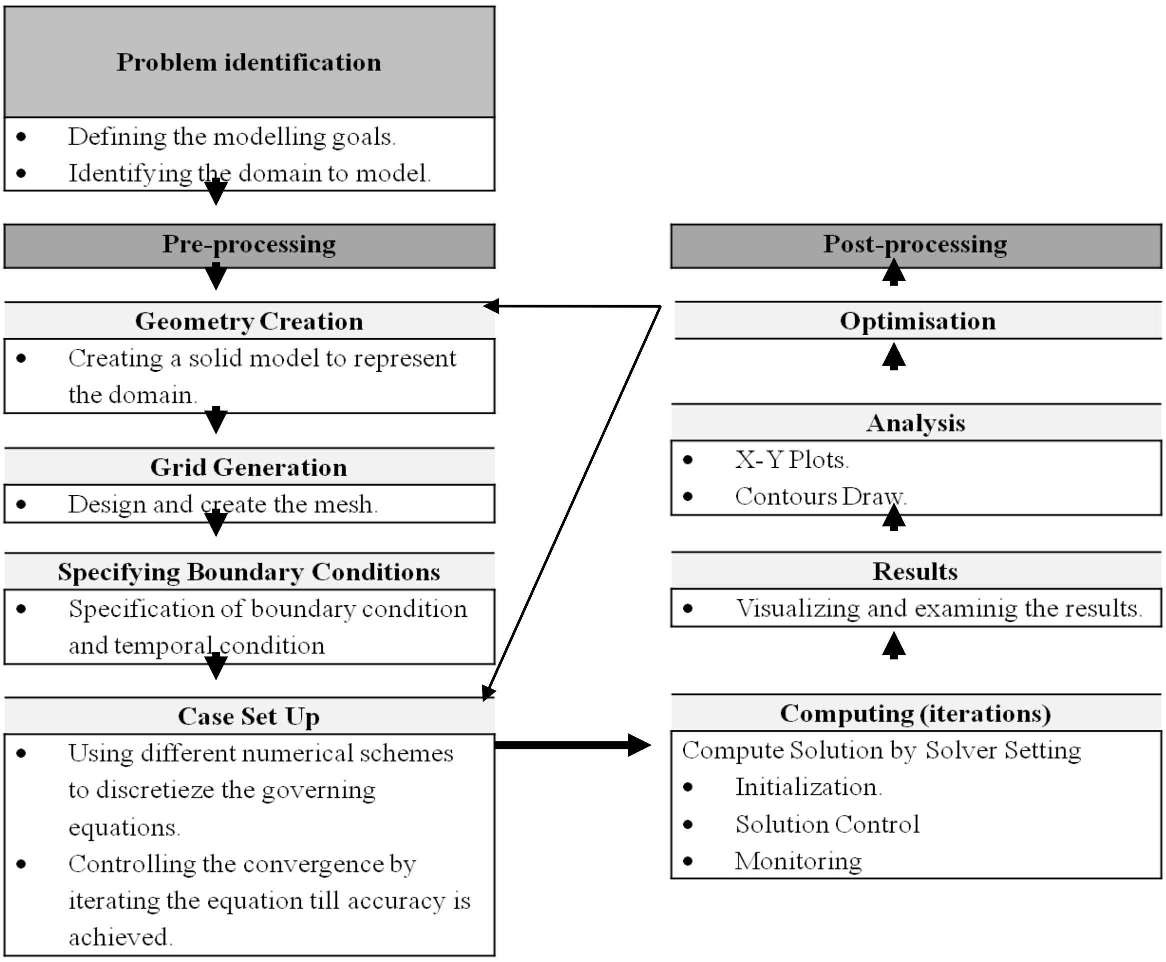


Figure 3.3: Procedures of CFD model development

3.1.2.1 Geometry creation

Initially, a two dimensional (2D) geometrical model was developed using the application of GAMBIT. The length of the geometrical model is 91.5m and the total width is 54.2m. This is based on the actual values of OBS plant onsite. The visual in Figure 3.4 is the outputs given by GAMBIT application of the OBS model. The geometrical developed in gambit consisted of 3 sections known as channels. The outer channel is the biggest and also where the influent is located. The cross sectional area of outer channel is 53 m^2, while the cross sectional area of middle and inner channel is the same value which is 26 m^2. The water from the distribution chambers enters the system through the inlet located in the outer channel. This water moves along the length of the outer channel and enters the middle channel through several penstocks. The water continues to flow along the middle channel and finally flow into the inner channel throughout the penstocks and exits the system throughout the effluent penstock.

At the beginning, two dimensional CFD model is chosen due to the small depth dimension relative to length and width, and the desire to reduce computational time (3D models can be very computationally intensive). 2D CFD model was developed to see the appropriateness of this biological treatment plant to be represented by CFD model. 2D CFD model is able to show the flow pattern inside the ditch however, it has limitation to represent the residence times throughout the system since the depth dimension is not included. Based on the performance of 2D basic CFD model, more advanced model, which is the 3D model, was developed. Model development processes of 3D CFD model are the same as 2D CFD model. However, the computational time has increased since the model is more complicated. In terms of geometrical model, only the depth dimension was added. The depth of the water is 4.4m.

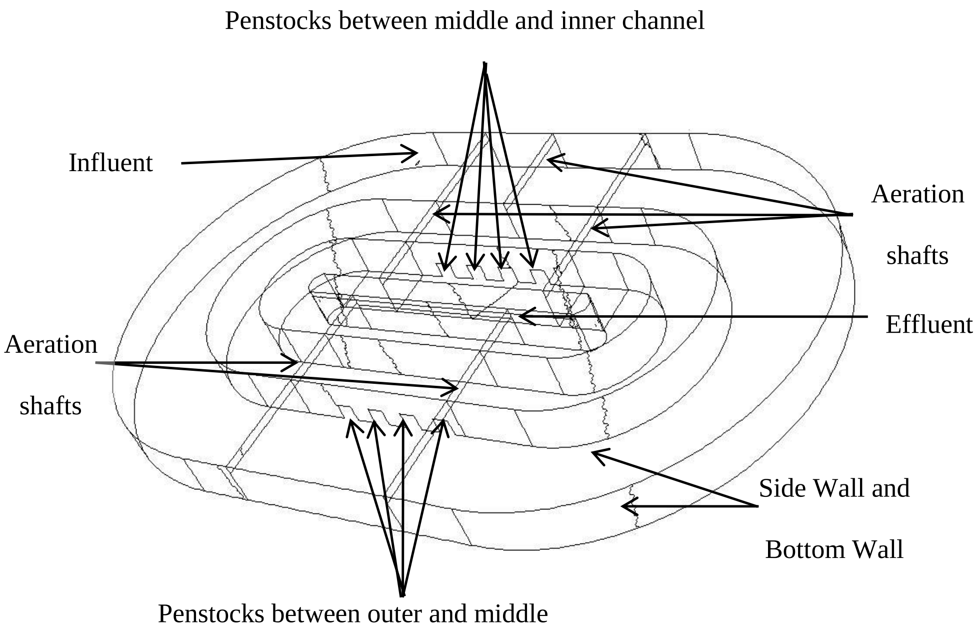


Figure 3.4: Geometrical layout of 3D model

3.1.2.2 Grid generation

The next step in modelling is the mesh generation. For the 2D CFD model, a structured grid consisting of quad elements was created to resolve the flow. A grid with 92152 numbers of elements was generated from the grid independency study. The grid independency study as explained in the next section resulted in the optimum grid for the present case to be around 92152 elements. Near wall model approach was applied, where the mesh close to the wall was refined in order to resolve the near wall flow for turbulent water flow. This ensures the wall boundary effects are taken into consideration while modelling turbulent flows. In order to ensure the accuracy of the flow simulation near the wall surfaces, the y+ values obtained for the OBS model are less than 5. Based on the equiangular skew approach, the worst element's quality value is 0.23. This was considered suitable for resolving the flow inside the system. The modelling domain was discretised using structured quad elements. An optimum skew angle for the quad elements must be ideally less than 0.6 to avoid diffusion related errors. During the grid generation of 3D model(as demonstrated in Figure 3.5), 989933 elements were generated. Due to the complex structure of the OBS, unstructured meshes were formed using hexahedral elements.

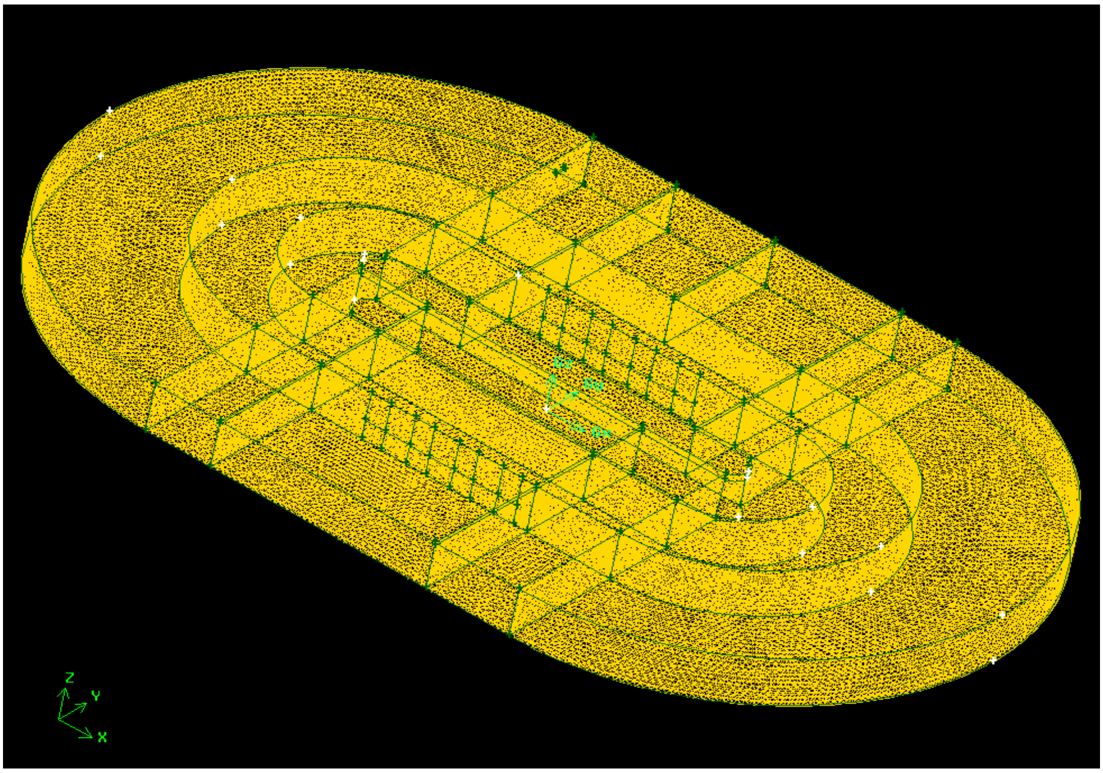


Figure 3.5: 3D grid layout

A grid independency study has been performed for OBS computational model. The model was initially developed using structured quad mesh resulting in 36,808 numbers of elements. This was considered as the starting number of elements in this case when it was firstly meshed. As summarized in Table 3.2, the grid independence test has been conducted for different size of mesh. Each adaptation gives a different number of elements and variation of velocity magnitude.

Table 3.2:2D grid adaptation (m/s)

Grid size (cells)	**36808**	**39553**	**42646**	**92152**	**163684**
Ch1-down	0.325	0.326	0.326	0.336	0.346
Ch1-up	0.342	0.353	0.362	0.362	0.358
Ch2-down	0.161	0.161	0.161	0.16	0.158
Ch2-up	0.172	0.175	0.176	0.18	0.184
Ch3-down	0.045	0.057	0.072	0.052	0.059
Ch3-up	0.103	0.105	0.102	0.093	0.088
Channel 3	0.061	0.063	0.064	0.064	0.063
Channel 2	0.163	0.164	0.165	0.174	0.177
Channel 1	0.298	0.305	0.319	0.326	0.332
Effluent	0.034	0.034	0.033	0.027	0.026
Influent	0.577	0.577	0.577	0.578	0.578

The results obtained show that the average velocity values do not change as the mesh refined to 92152 numbers of elements (as shown in Figure 3.6). Furthermore, once the number of elements was increased to more than 92152, the difference of maximum velocity values for different numbers of elements is very small and can be ignored. For this reason, the mesh with 92152 elements was chosen for further analysis. This is considered sufficient since the variation between subsequent grids were negligibly very small. The outcome of this grid independency study was that the grid that was developed was independent of any effects due to the grid size. This increases the accuracy of the simulation results and errors induced due to the grid are minimized to a large extent.

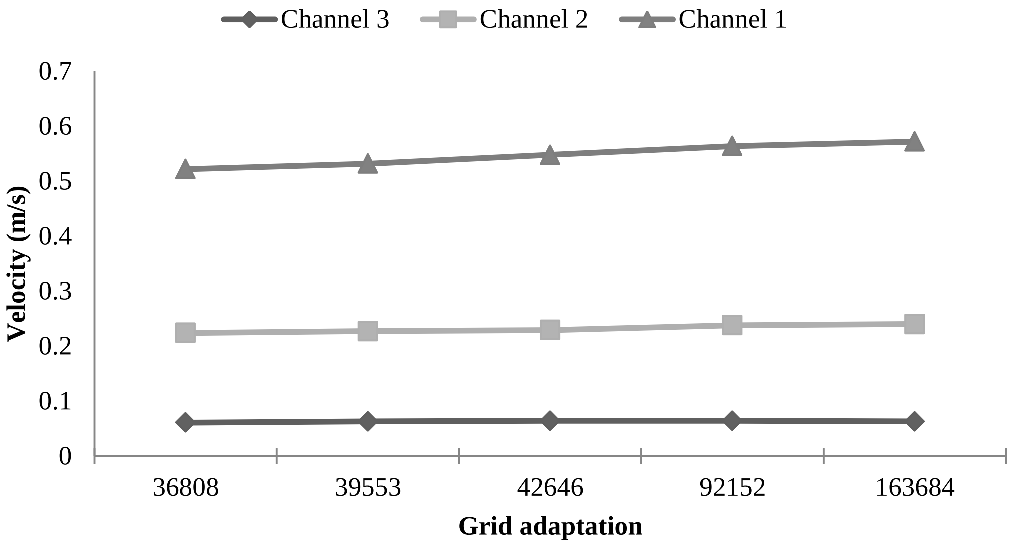


Figure 3.6: 2D Grid Adaptation

The same approach was applied to 3D model, grid independent tests were performed in order to make sure that the selection of grid size is optimal (the details are presented in Table 3.3 and Figure 3.7). Three dissimilar sets of meshes with the number of cells of 88,9303, 98,9933 and 1,077,463 were selected to simulate the velocity magnitudes at the sampling points inside the OBS. As a result, the second set was chosen for all further computations, in view of the low difference of velocity magnitudes simulated with the best possible mesh (989933 cells) and the refined mesh (1077463 cells).

Table 3.3: 3D grid adaptation (m/s)

Grid size (cells)	**889303**	**989933**	**1077463**
Sample 1	0.98	1.04	1.06
Sample 2	0.52	0.53	0.51
Sample 3	0.53	0.59	0.61
Sample 4	0.61	0.60	0.58
Sample 5	0.57	0.59	0.55
Sample 6	0.83	0.81	0.79

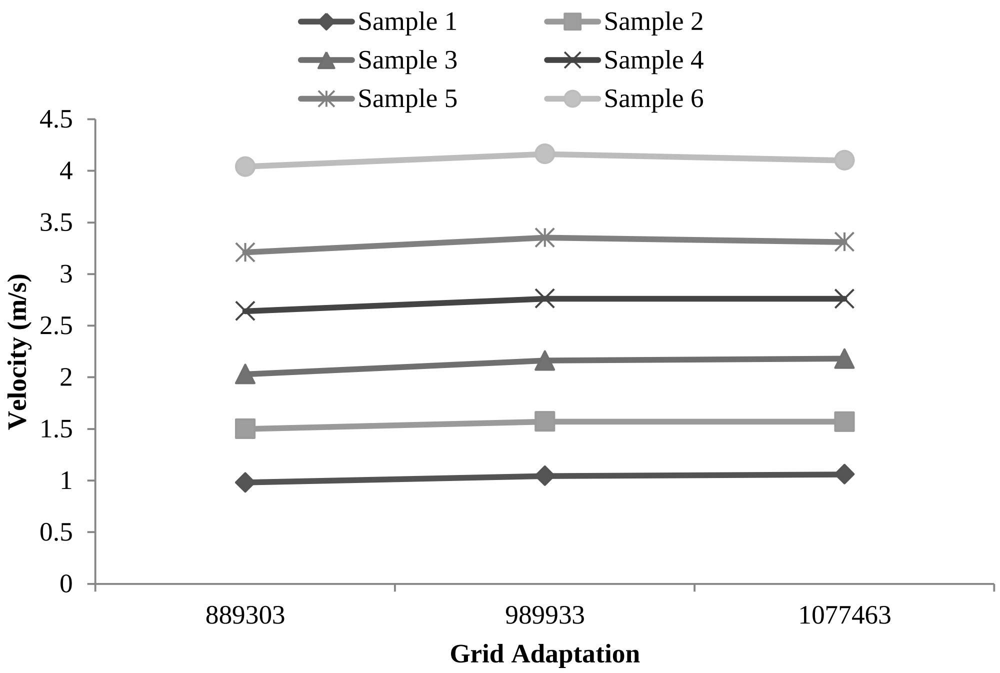


Figure 3.7: 3D grid independency analysis

3.1.2.3 Reynolds number calculation

For the purpose of this research, numerical simulation is carried out for mass flow rates of 580 kg/s. Using the Equation 2.11, the Reynolds number value for the velocity magnitude of 0.581 m/s obtained is more than 2, 5551 798. Therefore, the flow is considered as turbulent for flow rates of 0.581 m/s and above. It is important to determine whether the flow is laminar or turbulent in order to estimate a correct model, so that the results given by CFD will represent the actual condition of OBS.

3.1.2.4 Boundary condition definition

For the 2D CFD model, the boundary conditions are defined as shown in Table 3.2 below.

Table 3.4: 2D boundary condition definition

Name	Type
Influent	Mass-flow-inlet (mass flow rate is 580 kg/s)
Outer, middle and inner channel	Fluid
Penstocks between channels	Interior
Effluent	Pressure Outlet

In 3D CFD model, the boundary condition of the inlet was still applied as a mass flow inlet. This type of boundary condition was applied because it allows the total pressure to differ in response to the interior solution. Aeration shafts were represented by fan model with the definition of the pressure drop values. The pressure drop values were calculated using the parameters of the power input, area of the disc, the density of the fluid and the average flow velocity throughout the ditch. The fan model is actually predicting the amount of flow through the fan. Side walls and bottom walls of OBS were defined as a non-slip wall. The effluent was defined as outflow since the exit flow velocity and pressure are not known due to the complexity of the flow inside the ditch. The penstocks between the channels were presented as interface. The interface was used to unite two faces of different volumes. The details of the geometrical layout and boundary condition of 3D model are given in Table 3.5 and Figure 3.8.

Table 3.5: 3D model boundary condition

Name	Type
Influent	Mass-flow-inlet (mass flow rate is 580 kg/s)
Outer, middle and inner channel	Fluid (water liquid, air)
Aeration shafts	Fan
Penstocks between channels	Interface
Effluent	Outflow

3.1.2.5 Material properties

Standard values of water-liquid properties were used in this research. The density used was 998.2 kg/m^3, while viscosity as 0.001 kg/m.s. In 3D CFD model, two phases of fluid were involved. Water and air properties have been defined as summarized in Table 3.6.

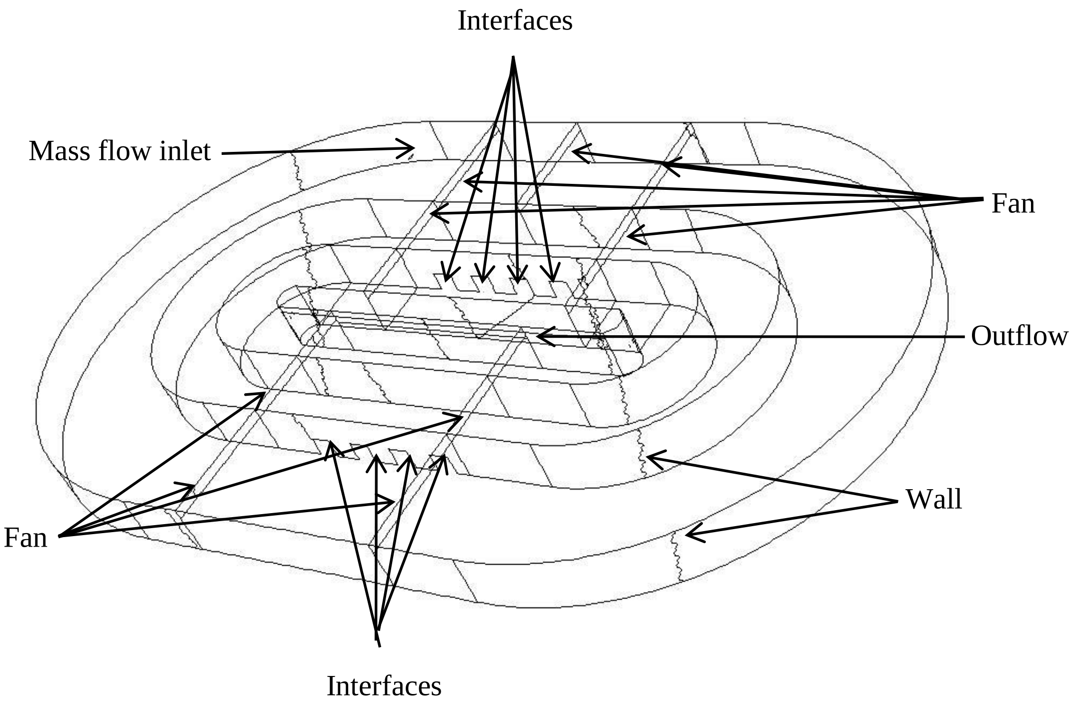


Figure 3.8: Boundary conditions of 3D model

Table 3.6: 3D material properties

Water-liquid (fluid)	
Density	998.2 kg/m3
Viscosity	0.001003 kg/m-s
Molecular Weight	18.0152 kg/kgmol
Air (Fluid)	
Density	1.225 kg/m3
Viscosity	1.7894e-05
Molecular Weight	28.966

3.1.2.6 Solver Setting

Using a computer with a memory capacity of 2 GB, the 2D CFD model took about 36 hours to converge for 86, 000 iterations. Velocity and k-epsilon turbulence residuals decrease until they reached 0.0001 or 1e-4. Table 3.7 shows the solver setting of the 2D model. Continuity decreases below 0.0000001 or 1e-7. Solver setting of the 3D model simulation is summarized in Table 3.8. The segregated solver of Fluent 15.1 was used with the default parameter settings applied. All simulations were performed using Intel Core 2 Quad CPU2.5 GHz processor and 8 GB installed memory (RAM). Each run took more than 72 hr of CPU time to reach steady condition.

The default values of standard k-epsilon turbulence model were used in the simulation. K-epsilon turbulence model is suitable for turbulent flows. Based on the Fluent's user guide, k-epsilon turbulence model has been considered as the simplest 'complete model', which allows the turbulent velocity and length, scales to be independently determined. Derivation of the standard k-epsilon are based on model transport equations for the turbulence kinetic energy (k) and its dissipation rate (ε).

Table 3.7: 2D model case setup

Model	Settings
Space	2D
Time	Steady
Viscous	k-epsilon turbulence model

Table 3.8: 3D Model Case Setup

Models	
Space	3D
Time	Unsteady
Viscous	Standard k-epsilon turbulence model
Wall Treatment	Standard Wall Functions
Coupled Dispersed Phase	Enabled
Solver settings	
Flow Equation	Solved
Volume Fraction Equation	Solved
Turbulence Equation	Solved

In this study, volume of fluid (VOF) approach of the multiphase model was applied.

Discrete phase model was used to get the hydraulic residence time throughout the ditch. In order to estimate the distribution of particles that exit the system, a cluster of particles was injected near the influent region of the channels. Around 1,000 particles were injected into the channels. Every time step corresponded to 0.01 second, which means that in order for the particles to trace 1 sec, it must be subjected to at least 100 time steps. This is necessary to maintain requisite accuracy of the iteration and to avoid, subsequent iterative errors that would magnify disproportionately under higher time steps. The case was set to auto save mode at the rate of 100 sec per case. The simulation would save the case files for every 100 seconds. Initial few cases were manually saved at the rate of 10 sec per case to keep track of the particles. The whole process took about 16 days to complete.

3.2 Model calibration and validation

Sampling and water quality analyses were conducted for the purpose of calibrating and validating the models. Onsite sampling works were performed at Bayan Baru Wastewater Treatment Plant. Samples were collected for 20 days at six points (S1-S6 as described in Figure 3.9) which were identified within the channels to give an indication of the incoming (upstream) and outgoing (downstream) values for each channel.The sampling points were 1m from the channel walls (horizontally) and 0.4m below the water surface (vertically). During actual ditch operation, the water near the aeration disks is affected by a centrifugal force, causing the water surface to be uneven. However, this unevenness is negligible when compared to more than 4m water depth in the channel (Guo et al., 2013). For the pupose of the model validation, velocity values of same six points have been obtained from the CFD results and compared to results of in situ measurements.

Onsite measurements were performed in order to get the idea of the values of some parameters, which were considered significant to the research, either for the purpose of analysing or for the purpose of monitoring. The velocity values were obtained using current meter. Other parameters such as dissolved oxygen (DO), temperature, pH, conductivity, salinity and total dissolved solid were used for monitoring purposes. All these values are obtained using Hanna Instrument (HI) 9828 Multiparameter Water Quality Meter. This instrument has been calibrated in the lab before it was used onsite. In order to ensure that the values given are correct, separate DO meter was also used. With the aim of controlling the data quality, quick calibrations as suggested in the instrument's manual are performed onsite every day before starting the measurement works. The range of DO detection for this instrument is 0.00 to 50.00 mg/L.

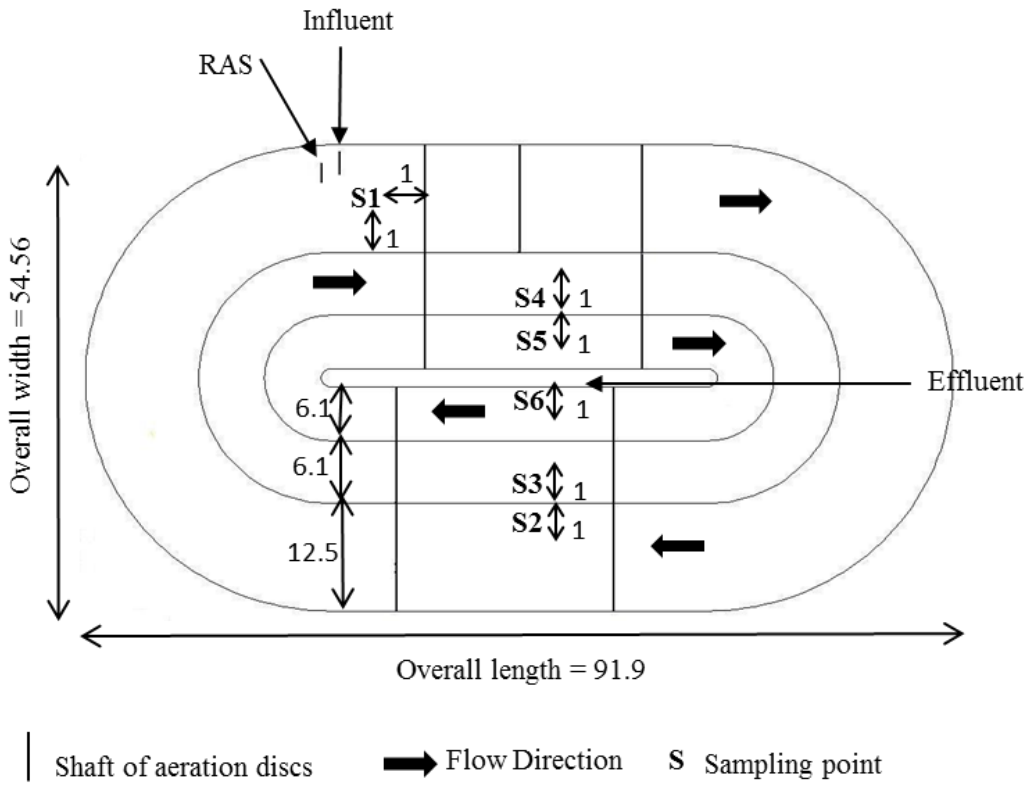


Figure3.9: Plan view sketch (unit: m) of the OBS and sampling points at 4.4m depth of water level

3.2.1 Onsite sampling at Bayan Baru Wastewater Treatment Plant, Malaysia

For the purpose of the research, a full scale Orbal Biological System (OBS) of the Bayan Baru Sewage Treatment Plant (STP) is chosen as a case study. Bayan Baru STP is located in Penang Island, Malaysia (5°18'54.9"N 100°17'40.9"E) as shown in Figure 3.10(Google Maps, 2016). Sewage treatment in Penang is managed by the national sewerage company, IWK. The Bayan Baru STP is being operated to treat the municipal wastewater of the city of Bayan Baru, Malaysia. The wastewater is being treated for two treatment levels, including preliminary and secondary treatment. The preliminary treatment consists of screening, grit removal and grease removal. The secondary treatment includes biological treatment by a US Filter Orbal Biological System and secondary sedimentation. Two OBS basins with a capacity of 50 000 m^3/day corresponding to 200 000 PE peak flow are used. Each has three concentric channels, with wastewater passing through the channels in series (from outer to inner). Each basin uses rotating aeration discs as the mechanical aeration device as shown in Figure 3.11. The amount of oxygen transferred into the ditch is controlled by the speed and immersion of the disc. Figure 3.12 shows the penstock gates, which control the flow volume from one channel to another.

Figure 3.10: Location of Bayan Baru STP

Figure 3.11: Rotating discs as mechanical aeration devices

Figure 3.12: Four penstocks gate between channels

3.2.2 Sample analyses

Besides the in-situ measurements, all the wastewater samples were sent to a third party commercial lab known as Union Lab, which is located in Penang. This lab is an accredited laboratory by the Malaysian government. The first test results produced by the private lab were compared to the results obtained through the lab testing conducted in Universiti Sains Malaysia. The results were found to be compatible (refer to Appendix B for more details). Standard laboratory methods as detailed in Appendix C were conducted by the Union lab in order to determine the BOD, MLVSS, and total nitrogen contents (including TKN, nitrite and nitrate) of the wastewater samples. Detailed on the sample analyses is provided in Appendix D.

3.2.3 Sensitivity Analyses

Sensitivity analyses (SA) were performed in order to determine the impacts of the inputs (independent variables on the outputs (the model dependent variables). For the purpose of this research ‘one factor at a time (OAT) approach was chosen as a method for the sensitivity analysis. OAT approach is a classic and widely used SA method where the input (variable) is perturbed (usually by 1%) and the changes in the output are observed or measured (Degasperi and Gilmore, 2008). Equation 3.1 contains at least three variables need to be analyzed for their sensitivity. These include the variable of Y, f_b and $K_{d.}$.

a= 1.46-1.42Y

$b = 1.42 f_b K_d$

Y= yield coefficient (gVSS produced per gBOD removed) ($gX_{v\,/g}BOD_5$)

$f_{b=}\ f_{b'}/\ [1+(1-\ f_{b'})K_d\,\Theta_c\,]$

f_b =biodegrable fraction of MLVSS generated in a system subjected to a sludge age Θ_c (X_b/X_v)

$f_{b'}$=biodegrable fraction of the VSS immediately after its generation in the system, that is, with Θ_c =0. This value is typically equal to 0.8 (80%)

Θ_c =Sludge age (d)

K_d =endogenous respiration coefficient (d^{-1})

Table 3.9 highlights the value of the variables used in this research and the value of the variables tested for the sensitivity analysis.

Table 3.9: Variable Values

Variable	Value used in this research	Value need to be tested
Y	0.6	0.5, 0.7
K_d	0.08	0.07, 0.09
$f_{b'}$	0.8	0.7, 0.9

Figure 3.13 shows the flow chart of the overall procedures involved in the research.

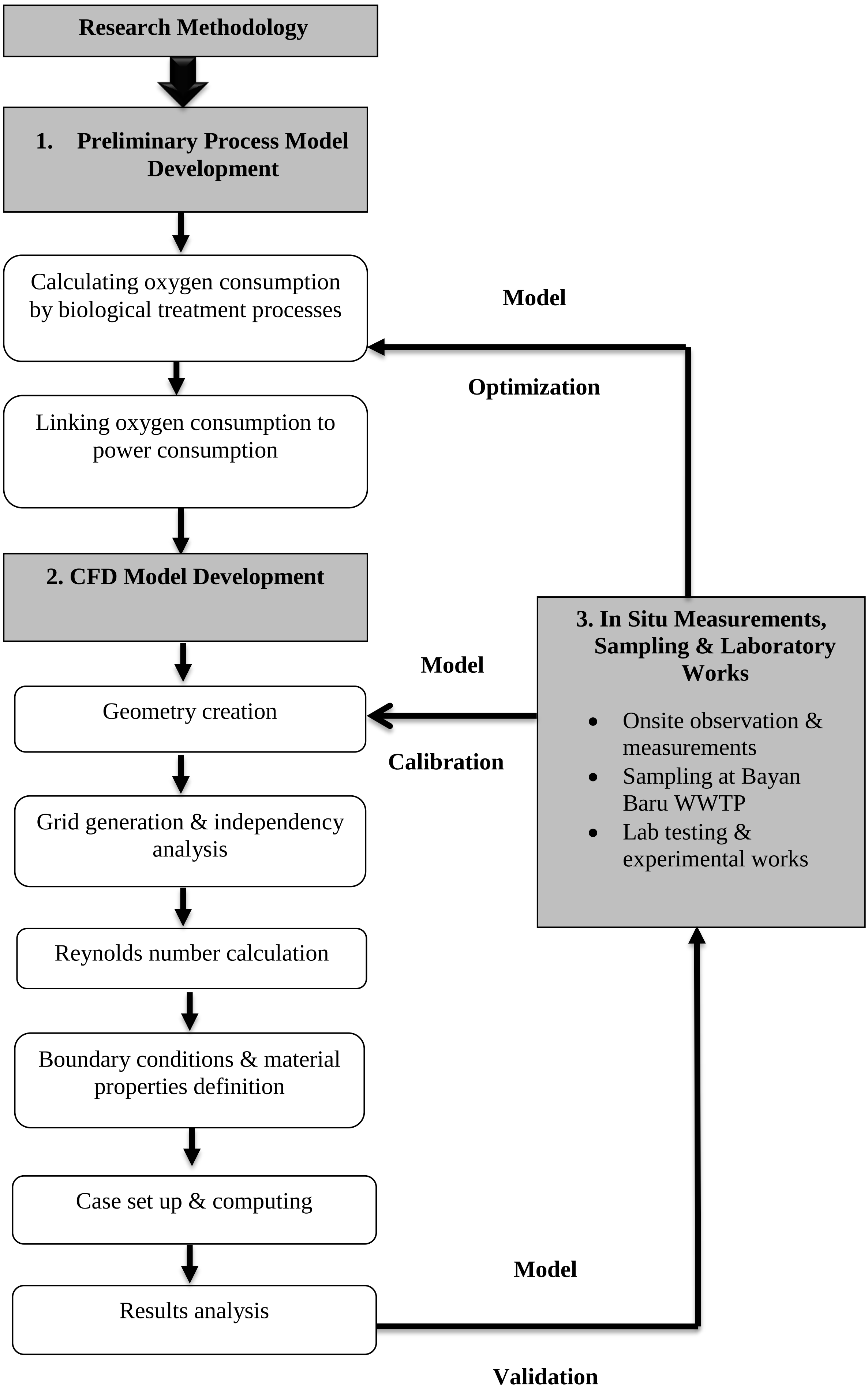


Figure 3.13: Flow chart of the overall research methodology

CHAPTER FOUR

RESULTS AND DISCUSSION

This chapter presents and discusses the results of the preliminary process model and CFD model. Using the developed 3D CFD model, the values of hydraulic residence time are obtained and further calculations are performed in order to get the OUR values. At the end of this chapter, OUR values, which are calculated based on CFD outputs are presented and analyzed. Power consumption related to the current OUR values and power consumption related to the increment of OUR values are also discussed.

4.1 Results of preliminary process model

The overall results of the calculation performed using the preliminary process model is shown in Figure 4.1. Detailed calculations are provided in Appendix F. Based on the graph, OUR values in outer channel are obviously higher than the middle and inner channel. Generally, the range of OUR values for the middle and the inner channel are nearly the same. This is most probably because both channels having more or less the same volume capacity.

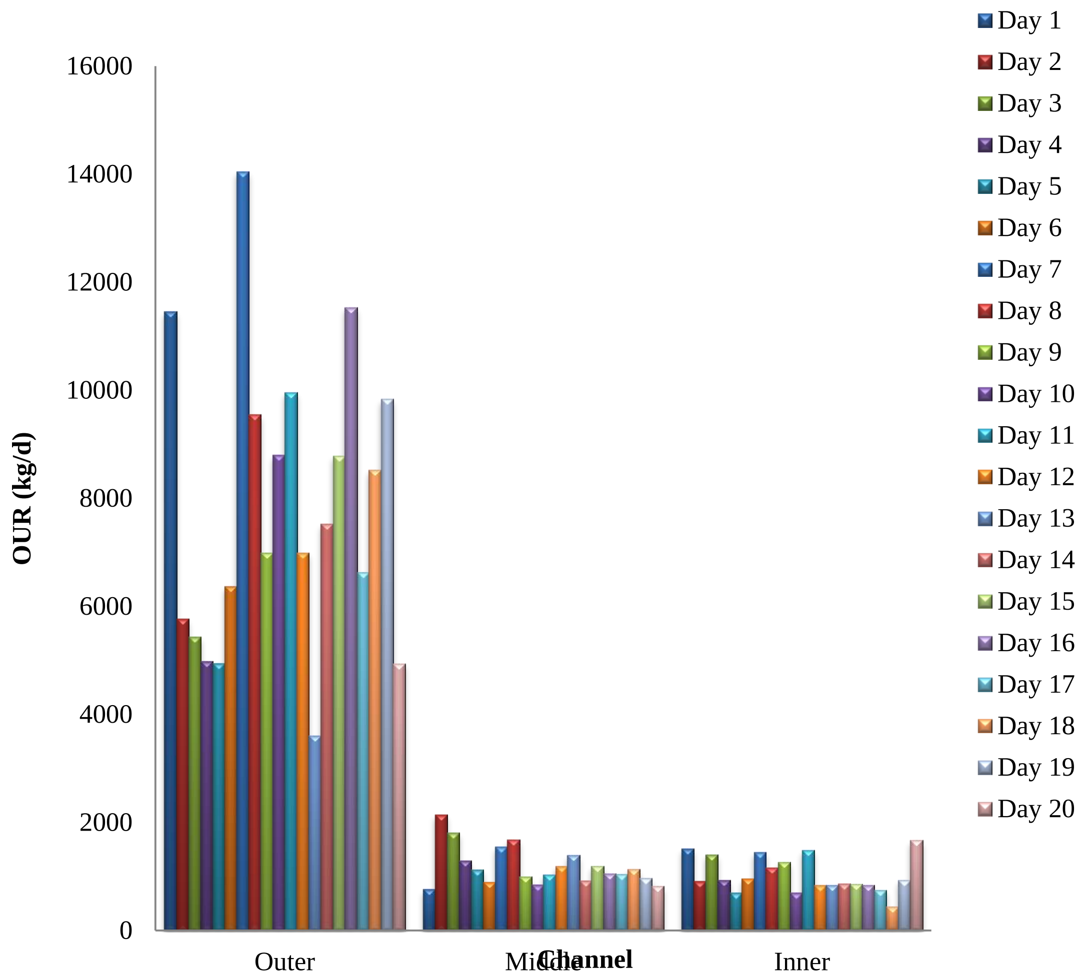


Figure 4.1: OUR values (kg/d) throughout the channels

The outputs given by the preliminary process model is summarized in Table 4.1. Based on the Table 4.1, maximum OUR value for the outer channel is 14049 kg/d, whereas the maximum value for the middle and inner channel is only 2145 kg/d and 1671 kg/d, respectively. Based on the outputs, there is so much variation of OUR values in the channels, especially the outer channel. The main reasons are probably because of the variations in the daily flow and wastewater strength. Minimum OUR value of the outer channel is still high compared to the maximum value of middle and inner channel. The average values of the middle and inner channel are not even half of the average OUR value of the outer channel. The average value of outer channel is calculated as 7836 kg/d, while the average value of the middle and inner channel are calculated as 1194g/d and 1027 kg/d correspondingly.

The results of ANOVA for preliminary process model are given in Table 4.2. Based on the results, the standard deviation of the average OUR value for outer channel is ±2686, whereas the standard deviation of the average OUR values for middle and inner channel are given ±361 and ±331. The average OUR value of the outer channel is roughly seven times higher than the average values of the other two channels. This is parallel with the fact that the outer channel is having a bigger tank volume and provided with a higher number of aeration discs compared to the middle and inner channel. The outer channel is having about 63 % of the total OBS volume and having 204 discs compared to the middle and inner channel which only having 132 numbers of discs. In addition to that, since the BOD and nitrogen levels are higher in the influent to the outer channel, so the ratio of the food to microorganisms is higher and therefore, more oxygen consumed in the outer channel compared to middle and inner channel.

Based on the results, the outer channel takes the highest portion of the overall oxygen consumption, which is 78% compare to the middle channel with only 12% and followed by the inner channel with 10%. This is parallel with the points highlighted by the Siemens Water Technologies (2007), where the actual oxygen demand of the first channel might be as high as 75% of the total amount of oxygen needed by the ditch.

Table 4.1: Summary of OUR values throughout the channel

Channel	Maximum value (kg/d)	Minimum value (kg/d)	Average value (kg/d)
Outer	14049	3612	7836
Middle	2145	764	1194
Inner	1671	439	1027

Table 4.2: ANOVA for preliminary process model

Groups	Count	Sum	Average	Variance	std dev
Outer	20	156727	7836.35	7214907	2686.058
Middle	20	23889	1194.45	130494.8	361.2406
Inner	20	20544	1027.2	110218.6	331.9919

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	6.03E+08	2	3.02E+08	121.3948	2.84E-21	1.655749
Within Groups	1.42E+08	57	2485207			
Total	7.45E+08	59				

4.1.1 Results of sensitivity analyses

The results of the calculation based on the preliminary process model were used to test the different values of Y, K_d and $f_{b'}$. Using the excel spread sheet, the outputs given by the different input values of these three variables were observed. Appendix F summarized the outputs of the calculation.

Figures4.2, 4.3 and 4.4 reveal how the tested values of variable Y K_d and $f_{b'}$ compared to the variables which used in this research. The t-test was used to investigate the difference between the average values relative to the spread or variability of their values. The t-test provides the probability that the null hypothesis is true. In this case, the null hypothesis is that there is no significant difference between the average values when different Y values (ranges from 0.5 to 0.8) were applied. The null hypothesis is true if only the t-test value is greater than 5 %. The details of the t-test calculation are attached in the Appendix E. The summary of the t-test values (in percentage) for each tested variables in comparison to the original values used in this research are shown in Table 4.3.

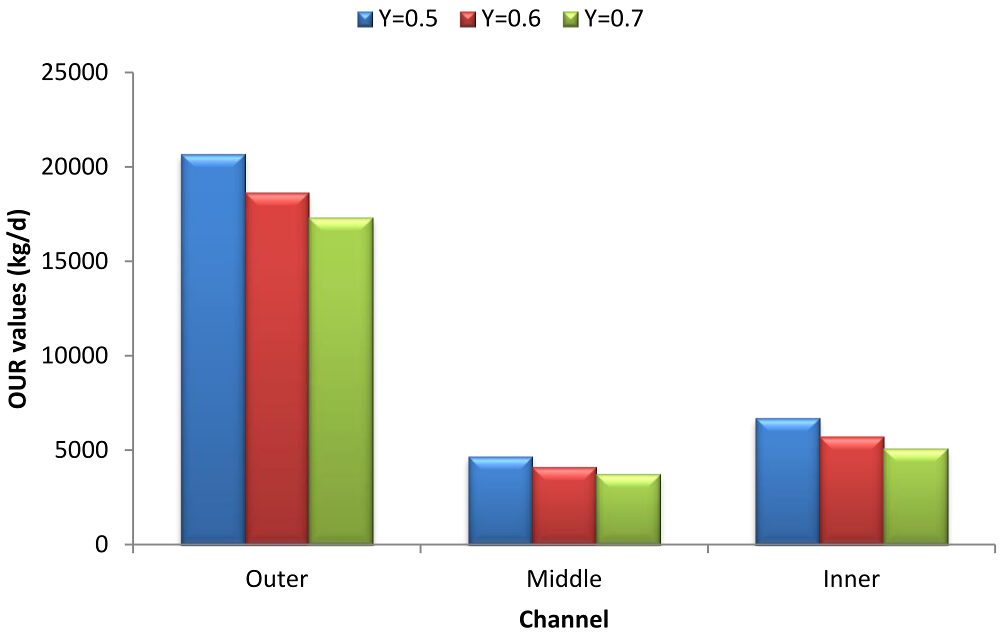


Figure .4.2: Average OUR values for different Y values

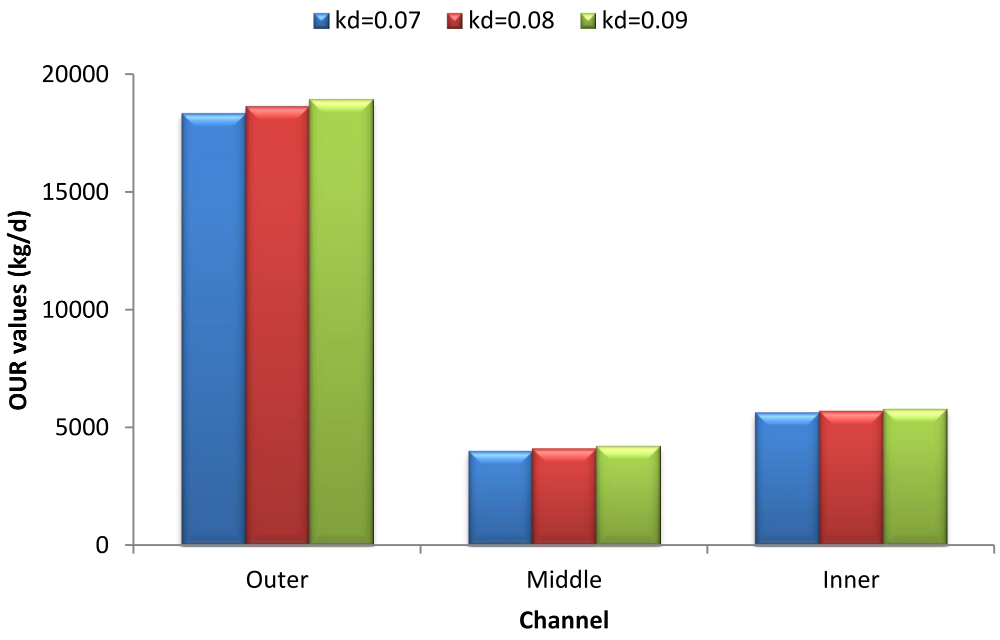


Figure .4.3: Average OUR values for different K_d values

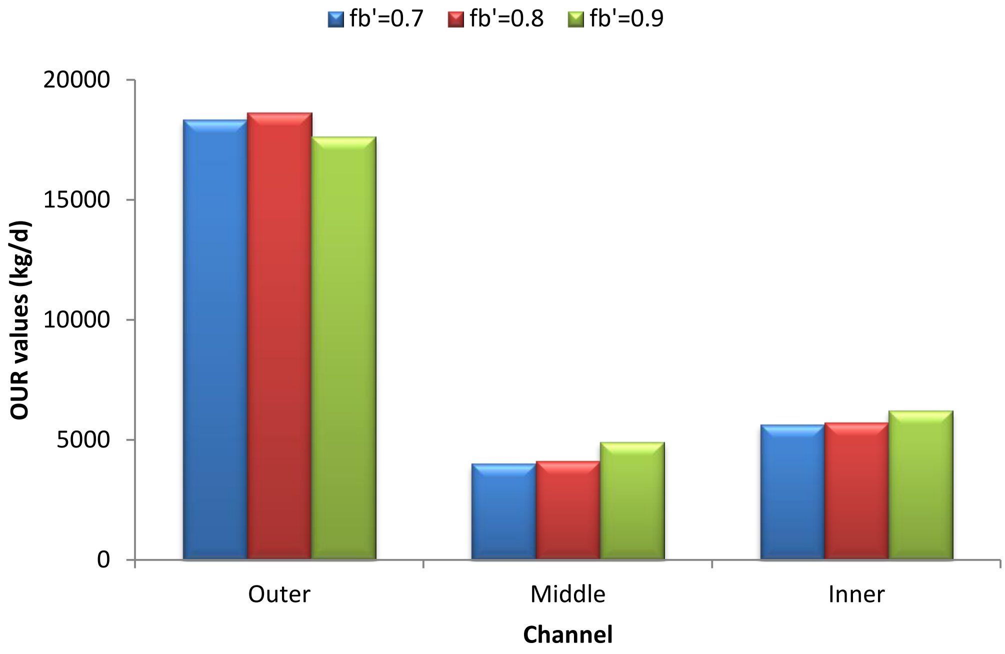


Figure 4.4: Average OUR values for different $f_{b'}$ values

Table 4.3: T-test values

Variable	Value need to be tested	T-test value for the outer channel	T-test value for the middle channel	T-test value for the inner channel
Y	0.5, 0.7	41.1% 53.3%	61.8% 69.8%	42.2% 52.1%
K_d	0.07 0.09	89.2% 89.3%	91.0% 91.0%	93.8% 93.8%
$f_{b'}$	0.7 0.9	89.2% 78.8%	91.0% 82.1%	93.8% 87.7%

Based on the results, there is no significant difference between the averages. In another word, the results validate the use of the variable values, which were applied in this research.

4.2 Results of the 2D CFD model

The modelling of the flow through the OBS channel involved certain assumptions of flow. These assumptions are required to simplify the model and to overcome the computational limitations. Firstly, as mentioned earlier, only 2D model was developed. This is considered sufficient to resolve the physics of the flow for a symmetrical channel. Secondly, steady state simulations were carried out and time variations of the flow were neglected. Over a period of time, the flow is expected to reach steady state, and time variations can be neglected. Thirdly, the water was assumed to be of single phase and the impurity that constitutes the usual wastewater was neglected. In general, the water entering the OBS consists of impurities in the form of suspended particulates, or other fluid impurities. In order to consider these impurities, it comes under the domain of multiphase flow regimes which requires defining the accurate description of their properties and their composition, which is a difficult scenario considering the extent of OBS and varying nature of impurities that enter the OBS channel. Hence, at this stage it is adequate to assume it as a single phase flow with fluid properties defined as water.

4.2.1 Results of basic CFD model without incorporation of disc aerator

Basic CFD model is a 2D CFD model, which was developed without incorporating the disc aerator or which also known as the mixer. Understanding the basic flow without the mixers will lead to a better understanding on which shafts should been operated to optimize the system. Basic flow model was developed in order to get the overall idea of the fluid flow pattern throughout the channels without any impact from the mixers. The purpose of doing this is to obtain a clearer picture of the function of each shaft in every channel since it is known that the aeration system such as mechanical aerators do not create uniform conditions of hydraulic flow patterns throughout the reactor (Daigger and Littleton, 2014). This basic CFD model was used as a reference for further studies in case of investigating the number of shaft that should be operated in order to optimize the OBS. Figure 4.5 shows the velocity contours of the flow inside the OBS system without the incorporation of disc aerator. The outer channel exhibits high velocity magnitude compared to the middle and inner channel.

As shown by Figure 4.5, the flow velocity decreases substantially as the flow enters middle and inner channel and it seems that the flow starts to lose its momentum. The flow velocity is heterogeneous throughout the ditch, with the outer velocity being greater than the inner velocity. (Guo et al., 2013). Without the support from the mixers, the flow is not able to maintain its velocities right from the outer channel to the middle and inner channel. In the real OBS, the velocity values are much higher compared to the velocity values given by the basic CFD model. This is possibly because of the missing role played by the mixers. Many authors have highlighted that the flow inside the oxidation ditch is agitated by the moving parts such as rotating discs(Yang et al., 2011, Lei and Ni, 2014, Karpinska and Bridgeman, 2016)

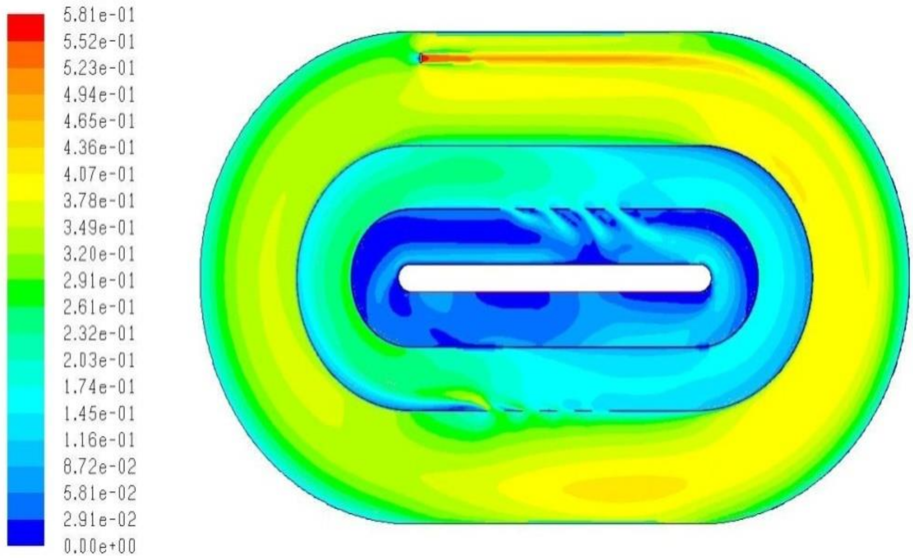


Figure 4.5: Contours of velocity magnitude (m/s) for CFD model without disc aerator

The values of average velocity obtained for each channel are shown in Figure 4.6. The average velocity value for outer channel is 0.33 m/s, the middle channel is 0.17 m/s and the inner channel is 0.06 m/s. There is about 47 percent decrease in the flow velocity as the flow enters the middle channel from outer channel and there is 63.2 percent drop when the water flows from the middle channel into inner channel. The simulation results have indicated and emphasized the role of aeration discs not only as mixers but also as flow regulators that maintain the requisite flow inside the OBS channels. These two functions of the aeration discs are the key points that should be taken into consideration in every decision to be made in term of optimization of the system.

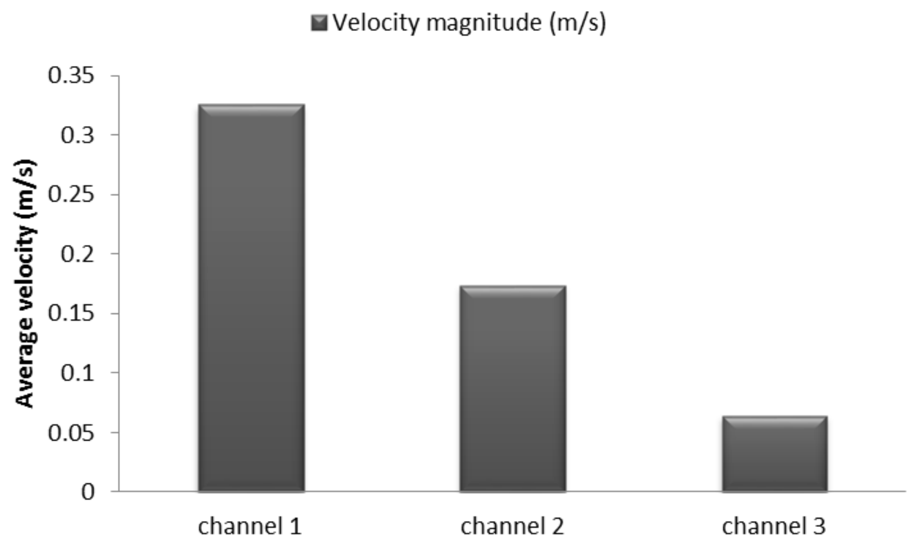


Figure 4.6: Average Velocity Magnitudes for CFD Model without Mixers

Figure 4.7shows the vector plot coloured by velocity magnitude for CFD model without the incorporation of mixers. As shown in Figure 4.8, vector plots indicate the presence of reverse flow in the inner channel. The flow is almost exhausted and continues to lose its impetus as it flows into the inner channel. Since the effectiveness of the OBS system lies in efficient mixing of oxygen, the flow across all the channels is required to have sufficient momentum to facilitate mixing of the water in order to provide enough oxygen to the system. In the present case without the use of mixers, that objective cannot be fulfilled. Hence, the use of appropriate disc aerator is necessary for an efficient mixing of water and oxygen as well as maintaining the required momentum of flow. This is precisely the reason why in real OBS the mixers are incorporated. As mentioned by Guo and his colleagues (2013) the flow velocity and dissolved oxygen concentration in the channels of OBS were affected by the rotary discs inside the ditch.

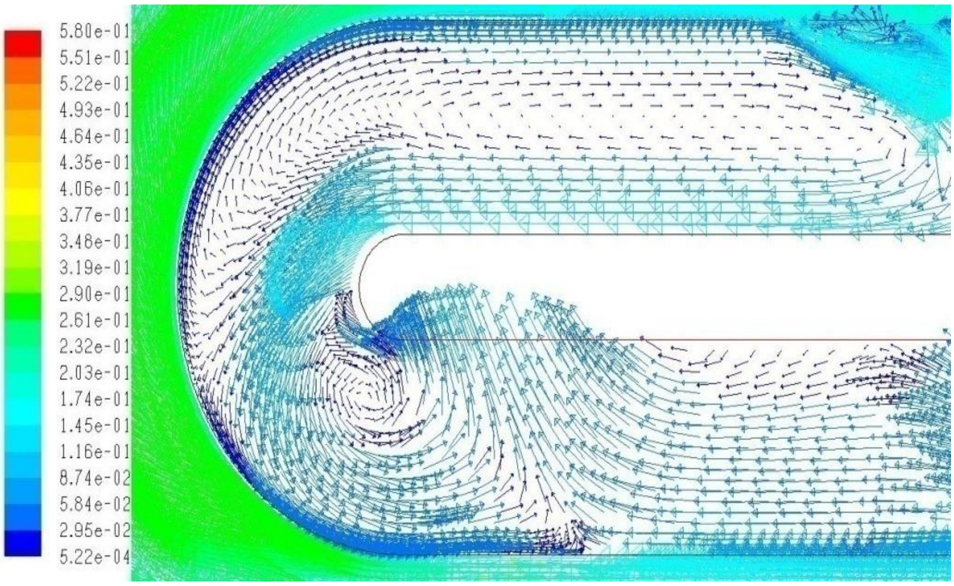


Figure 4.7: Velocity vectors coloured by velocity magnitude for the case without mixers

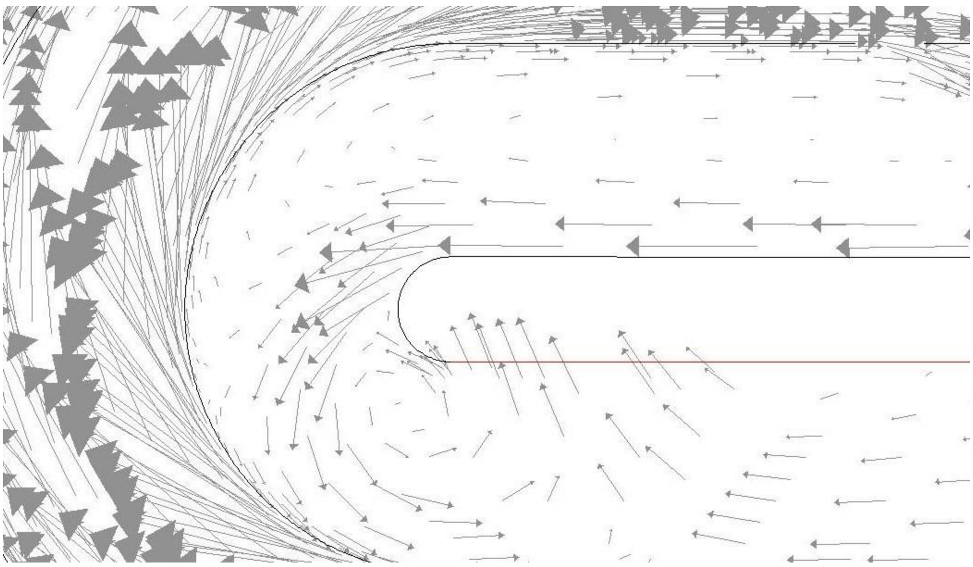


Figure 4.8 : Velocity vectors inside the inner channel (without mixers)

4.2.2 Results of CFD model with the incorporation of discs aerator

CFD studies have also been carried out to ascertain the influence of mixers on the flow inside the channels. The geometrical construction of the model was similar to the one without mixers except that mixers or aeration discs were incorporated in the model at certain locations as shown in Figure 4.9.

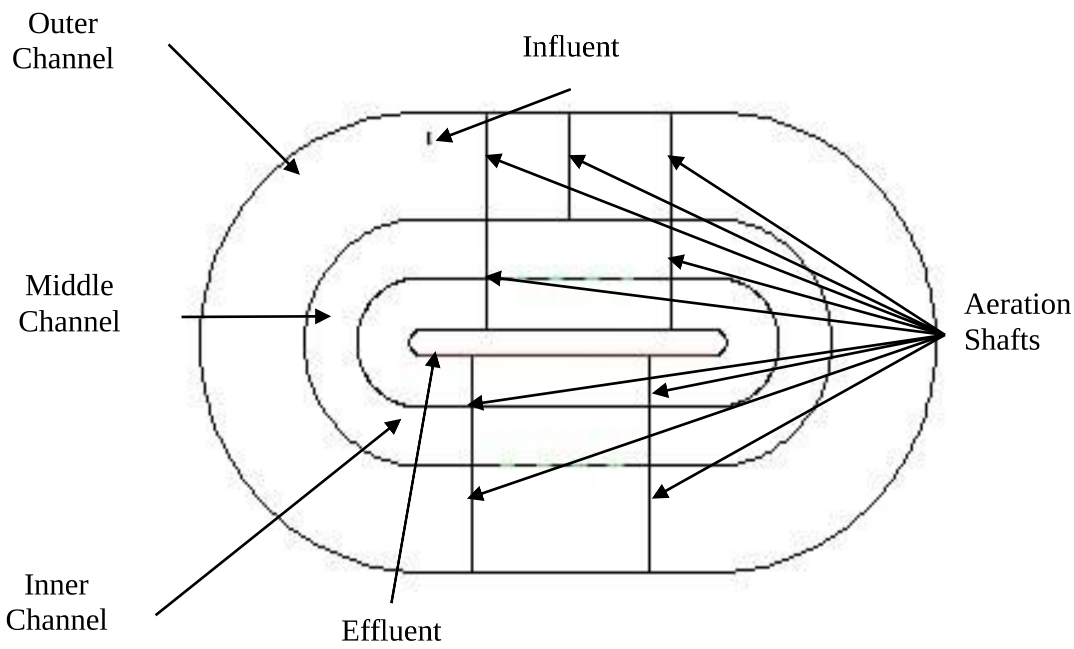


Figure 4.9: Geometry Layouts of OBS including the Mixers

The aeration discs were defined as fan with the calculated pressure drop values. The pressure drop values were calculated based on parameters such as density of the fluid, the average flow velocity in the ditch, the power input and area of the disc (Yang et al., 2011, Xie et al., 2014).

Based on Figure 4.10, there are significant increments of the velocity values inside the OBS after the incorporation of aeration discs. As observed in the case without mixers, the flow momentum could not be maintained in the OBS especially inside the inner channel. However, after the incorporation of mixers, there are major increases in the velocity of the flow. The values of velocity near to mixers show sudden jump in velocity values and therefore the velocity distributions across the channels are also higher.

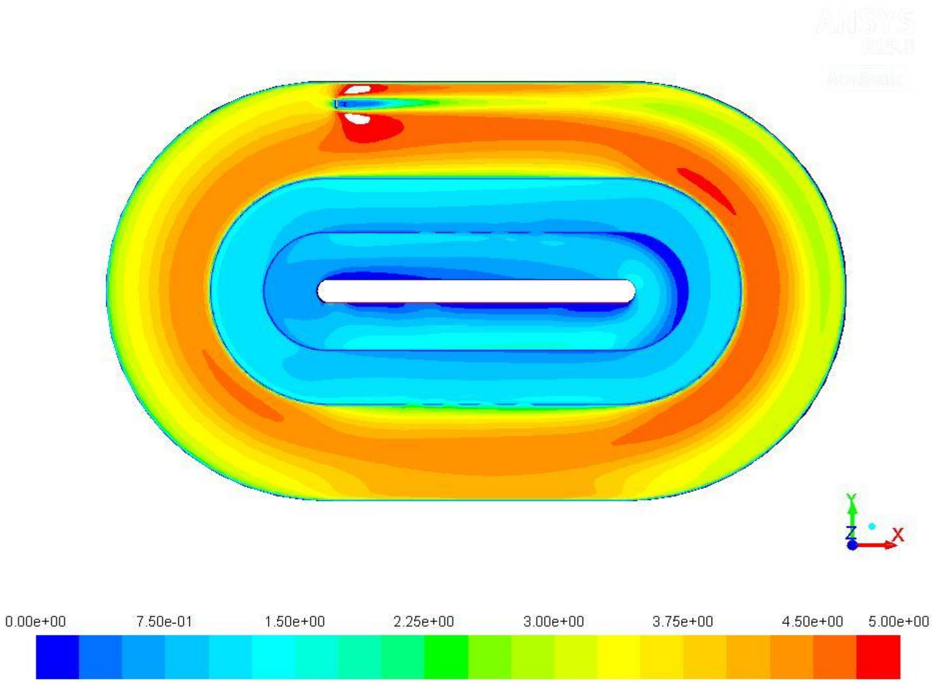


Figure 4.10: Contours of velocity magnitudes with the incorporation of aeration discs

The vectors plot in Figure 4.11 also shows that the flow reversal inside all the channels was avoided and flow continuity established due to the incorporation of aeration discs. The difference of velocity vectors between the previous case without the incorporation of the mixers (Figure 4.7 & 4.8) and the case with the incorporation of the mixers (Figure 4.11) can be clearly seen. As mentioned by Guo and his co-authors (2013), the variations in flow velocity in the longitudinal (along the channel) and transverse directions were closely related to the spinning speeds of the rotary discs(Guo et al., 2013) . Even in the critical part of the inner channel, where the flow almost lose its motion, the water still can flow in the right direction as it happens onsite. This ensures flow continuity inside the OBS system.

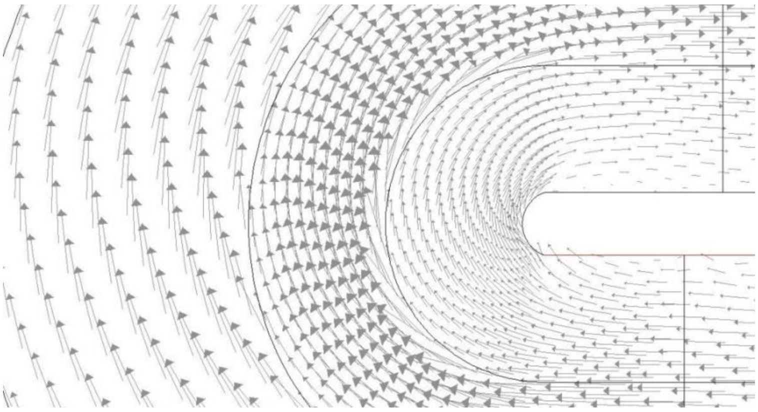


Figure 4.11: Velocity vectors

The results have shown that with the incorporation of the aeration discs, the real flow conditions inside OBS can be characterized and represented by the developed CFD model. It can be clearly seen that, in the case without the incorporation of mixers, the flow has already deteriorated and lost its impulse, whereas in the case with the incorporation of mixers, it demonstrates higher flow magnitude throughout the OBS.

4.2.2.1 Average velocity magnitude at sampling points

The results given by 2D CFD model have shown that CFD approach is applicable to study the flow of OBS. These results have created the potential of using CFD model for further investigation of the current operating system. Nevertheless, there are some limitations of 2D model, which needs to overcome in case of further exploration and analysis. In this case, the 2D model has simplified the entire flow into a pipe flow notion. Only steady state simulations were carried out and time variations of the flow were neglected. The water was assumed to be of single phase and the impurity that constitutes the usual wastewater was neglected. Based on Figure 4.12, most of the velocity magnitudes given by 2D CFD model are too high compared to the values obtained onsite. Most of the values given by 2D model over predicted the actual values, except the value at the upstream of the inner channel, where the results given by 2D CFD model has under predicted the velocity values measured onsite. The variation of the values given by 2D CFD model compared to the values measured onsite may be caused by the action of not modelling the depth dimension. Because of this reason, a better model that taking into account the depth of the tank is modelled. 3D CFD model is needed in order to get a better flow representation.

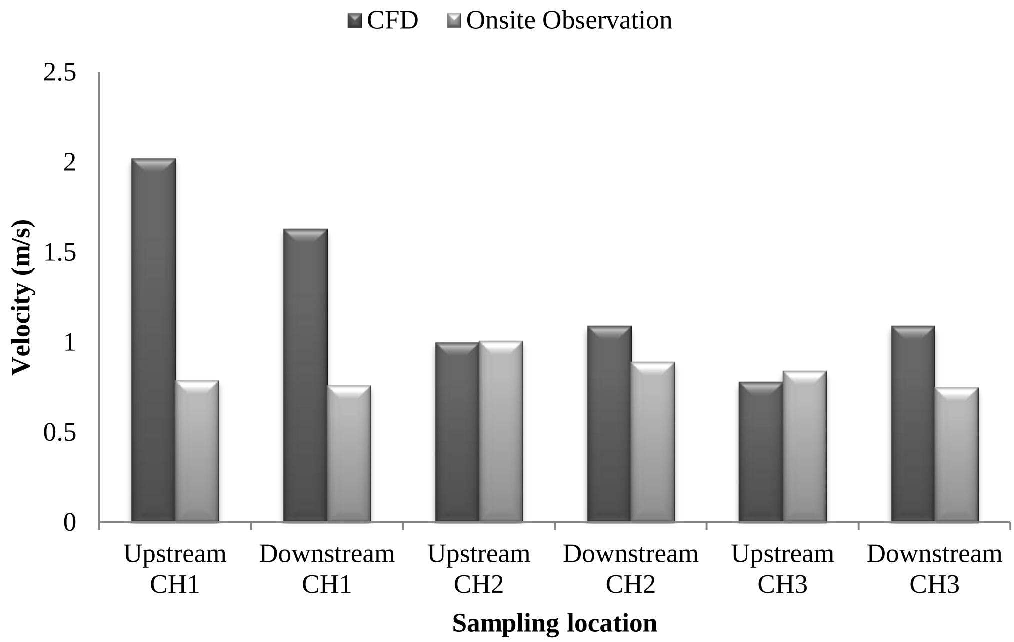


Figure 4.12: Comparison of the velocity magnitude given by 2D CFD model and onsite observation

4.3 Results of 3D CFD model

3D CFD model was developed in order to improve the application of CFD approach to investigate the flow pattern and the distribution of hydraulic residence time throughout the ditch. For 3D model, unsteady state simulations or also known as transient model were carried out to take into consideration the time variations inside the ditch. Two phase model (liquid-gas) was developed and an open channel system was applied. As stated by Xie et al. (2014), compared to the single phase model simulation, the relative error between simulation results and field data in two phase model can be reduced to about 3% (Xie et al., 2014). Material properties for both water and air were also defined to be included within the system. A reason why solid was not chosen to be incorporated in the simulation because 99% of the wastewater is water and only 0.1% of it contains impurities such as suspended solids substance and colloidal (Hadad and Ghaderi, 2015) .

The 3D model was developed on the foundation of2D model. The 3D model of the waste water treatment and its analysis is a significant improvement over the 2D model. In order to realistically capture the flow characteristics as well as the HRT, 3D models provide promising alternative. Besides, the open channel flow modelling can capture the actual flow process. On the contrary the 2D model simplifies the entire flow assumption into a pipe flow notion. This is avoided in case of the open channel flow where the top surface of the tank is exposed to the atmosphere. This implies that, the atmospheric conditions act on the water tank model, and therefore, mixing process facilitated by mixers can be clearly visualized.

4.3.1 Geometry and salient features of 3D model

3D geometrical model of the OBS is shown in Figures 4.13 and4.14. Various salient features have been incorporated in the model. These include the computational boundaries such as the influent, penstocks between the channels, free surface and the effluent. The model has also included 5 aeration shafts in outer channel, 4 shafts in middle and inner channel.

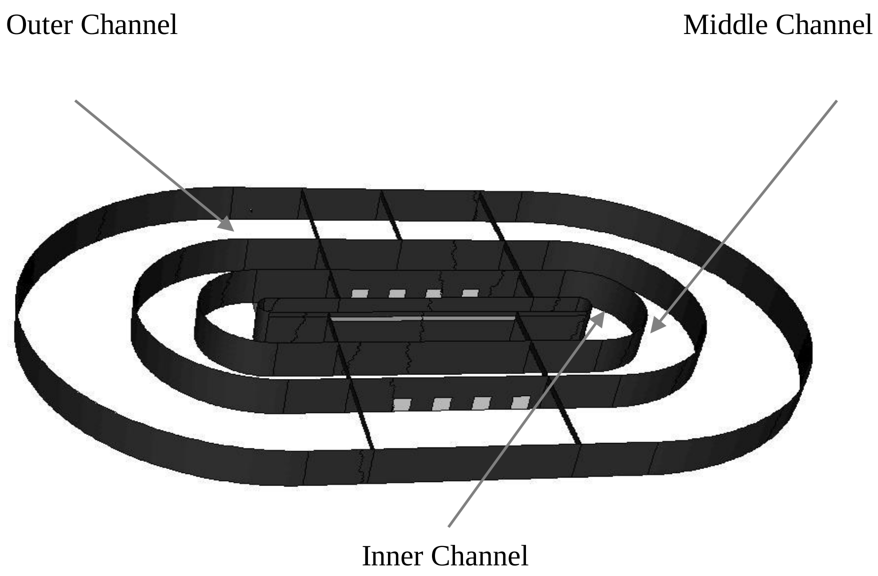


Figure 4.13: Geometry Layout of 3D model (channels of OBS)

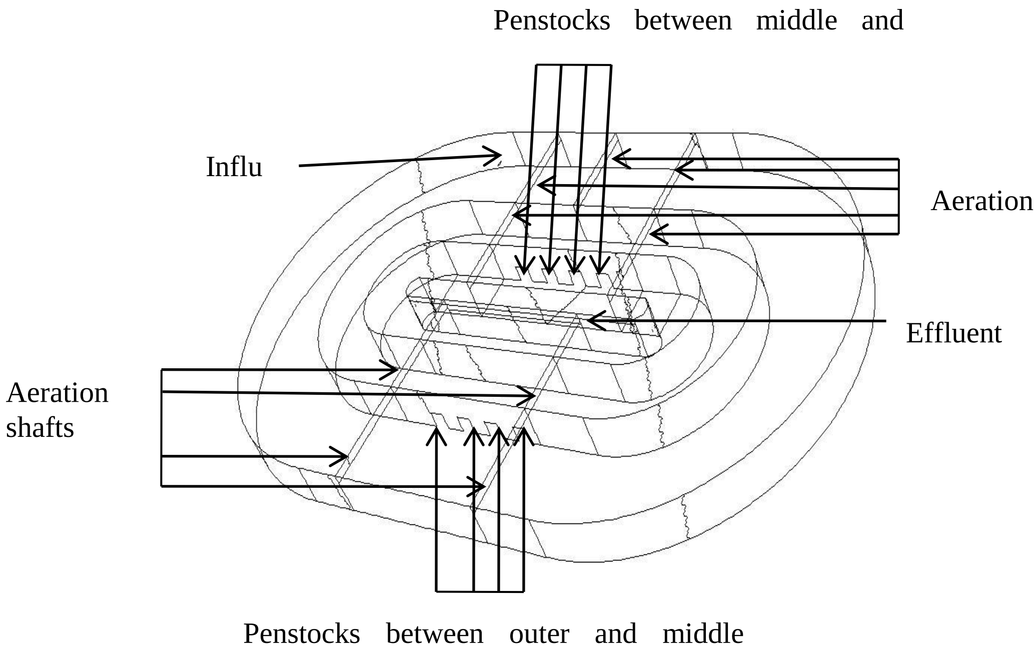


Figure 4.14: Geometry Layout of 3D model

4.3.2 Volume fraction of air and fluid (water) across the ditch

Open channel flow modelling which takes into consideration the influence of atmospheric air, also provides the data of the volume fractions of fluid (water) and air. Figure 4.15 and Figure 4.16 describe the distribution of two phases (air and water) involved in this study. The sectional contours obtained at the centre of the OBS show the distribution of air and its influence on the water due to the aeration discs. This was not possible using the simplified 2D model. The 3D model is able to provide more detailed information on the effects of the aeration devices as the mixers inside the OBS. Variation of volume fractions are in fact supporting the reality of the occurrence of anoxic and aerobic conditions throughout the OBS. The volume fractions have indicated the real conditions of OBS as an open channel system.

Figure 4.15 describes the fraction of water in the OBS. The distribution is represented in the array of colors with blue representing less or zero volume of water and red indicating the maximum portion of the composition to be water. Thus, it can be inferred from this figure that water composition is maximum at the bottom of the tank as expected, whilst it is lowest on the top of the tank. This is because the top portion of the tank is exposed to atmospheric air on account of the open channel flow modelling. This was possible only with 3D CFD model. The 2D model would fail to account this fundamental notion of mixing of air which was the purpose for which the oxidation ditch was built for. Most works found in literature assume a 3D channel without the open channel system. This renders the flow to be identical to that of the inside of a closed duct and therefore would not accurately capture the realistic flow assumption of OBS.

Secondly, it can also be observed that due to the mixers, the air composition is extended to at least half of the tank's depth as can be inferred from Figure 4.16. Based on Figure 4.16, it can be seen that the volume fractions of the air are higher at the surface level compared to the bottom of OBS. This also demonstrates that the oxygen mixing is facilitated by the mixers and the mixers are effective in increasing the composition or volume fraction of oxygen throughout the ditch. The figure shows that oxygen penetration is almost negligible at the bottom of the OBS. These important findings are made possible due to the advanced CFD model developed incorporating the open channel flow system.

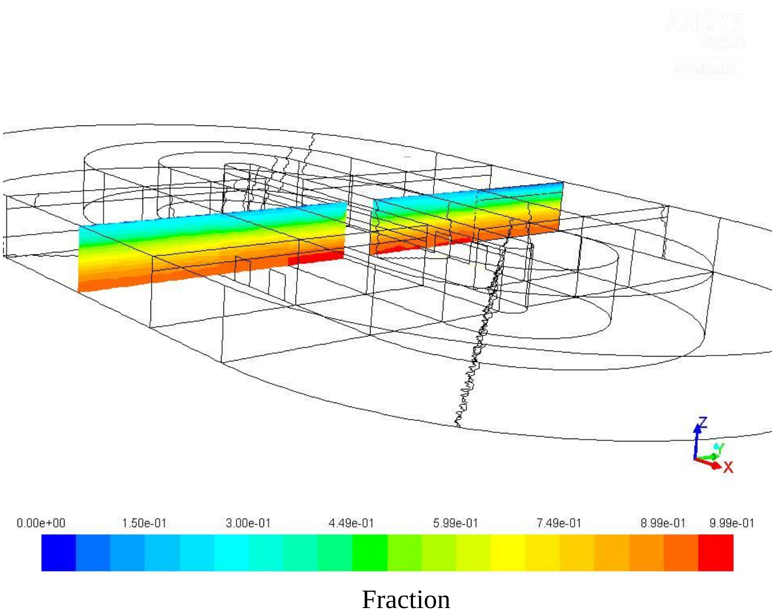


Figure 4.15 : Contours of volume fraction of water

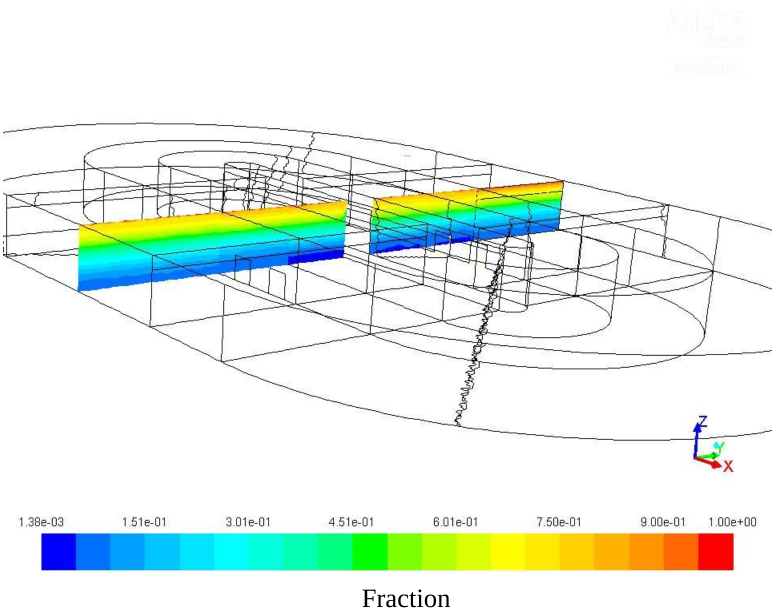


Figure 4.16: Contours of volume fraction of air

4.3.3 Pressure distribution across the channel

The following Figure 4.17 depicts the pressure contours across the OBS. The figure was captured at a height of 4m. The pressure drop across the mixers is observed (as shown in Figure 4.18). The drop in pressure is a reflection of the aerator discs rotational speed, which accelerates the flow of water, and contributes to the mixing processes inside the OBS. As shown by Figure 4.19, pressures observed at different depths have also changed. This is due to the fact that, disc diameter penetrates only the top surface of the ditch and therefore the plane surface below the aerator disc will have different pressure distribution than observed along the plane having the aerator discs.

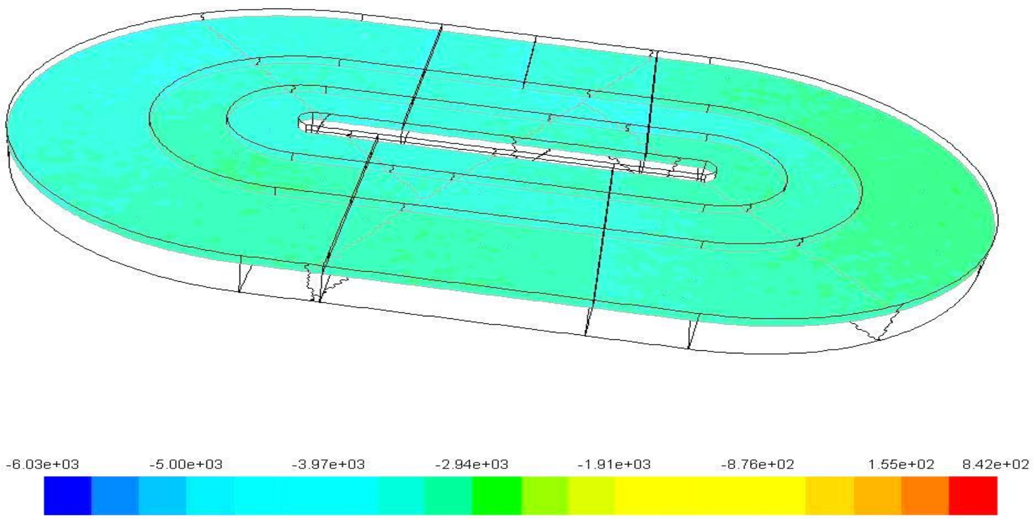


Figure 4.17: Contours of pressure throughout OBS

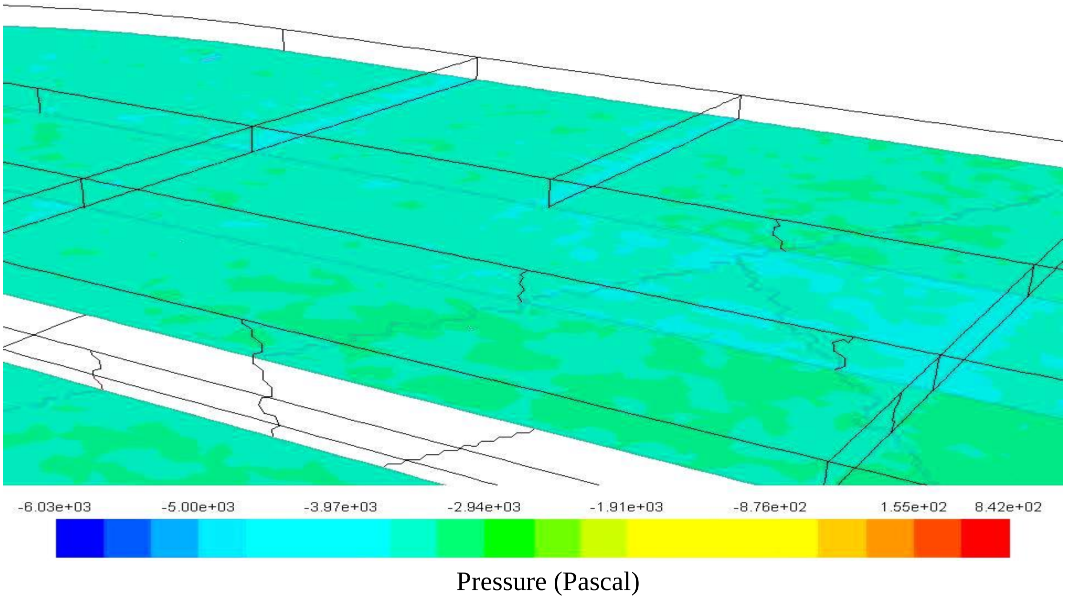


Figure 4.18 : Contours of pressure near to aeration shafts

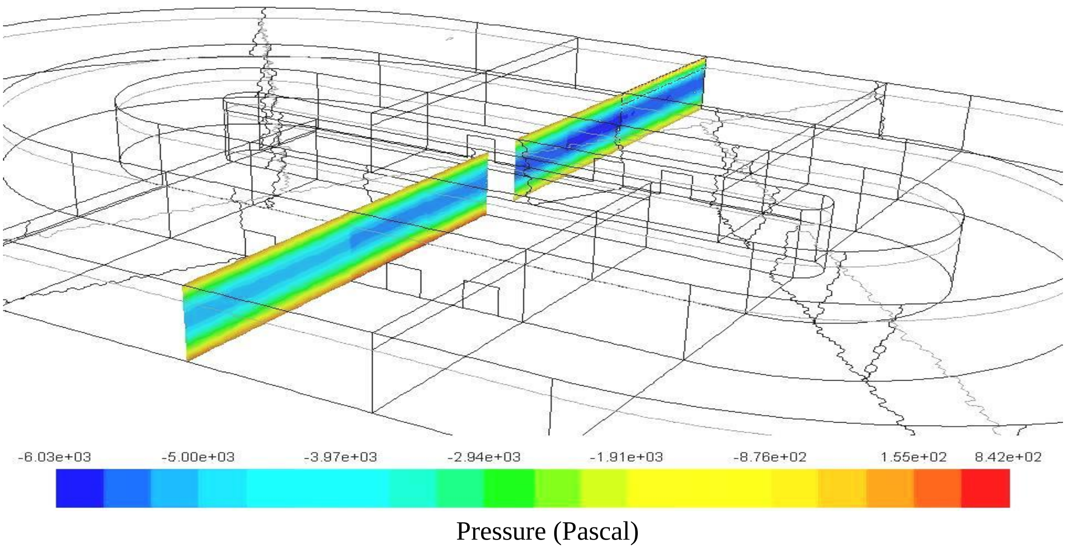


Figure 4.19 : Contours of pressure at different depth

4.3.4 Flow analysis of 3D OBS Model

Figure 4.20shows the contours of velocity magnitude as observed on the top surface of the ditch. It is evident from this figure that, the maximum flow velocity is at the mixers which helps maintain the flow in the aerator disk. Figure 4.21 to Figure 4.23 show the flow distribution at various depths inside the ditch. This is main advantage with 3D CFD modelling. The flow features can be obtained at any depth or location in the mixing tank. Further to this observation, Figure 4.24 provides a cross sectional views of velocity profiles at different locations near the shafts. It can be clearly seen that the velocity distribution varies from a maximum on the top surface to the minimum on the bottom of the tank. The first half of the tank shows high velocity gradient inferring better mixing due to the action of the shafts. The vector plot as can be seen in Figure 4.25 depicted the right vectors of the flow. The flow’s vectors near to the aeration shafts (refer to Figure 4.36) and vectors of the flow through the penstocks (pictured in Figure 4.27) have also indicated that the model has successfully represented the OBS. They represent flow of water from the outer channel into the middle channel as well as from the middle channel to the inner channel.

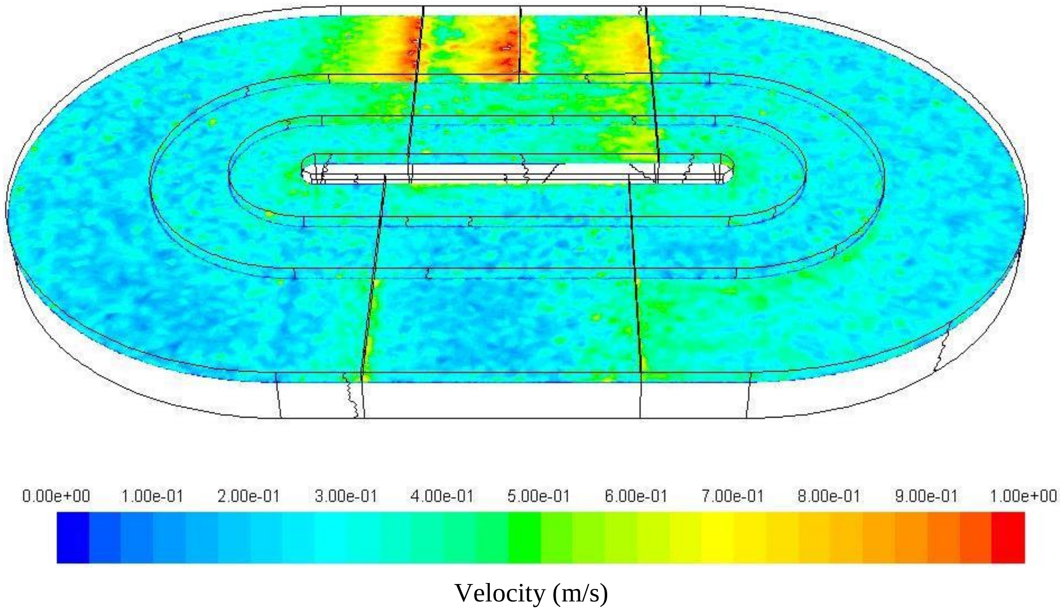


Figure 4.20: Contours of velocity magnitude at 4meter depth

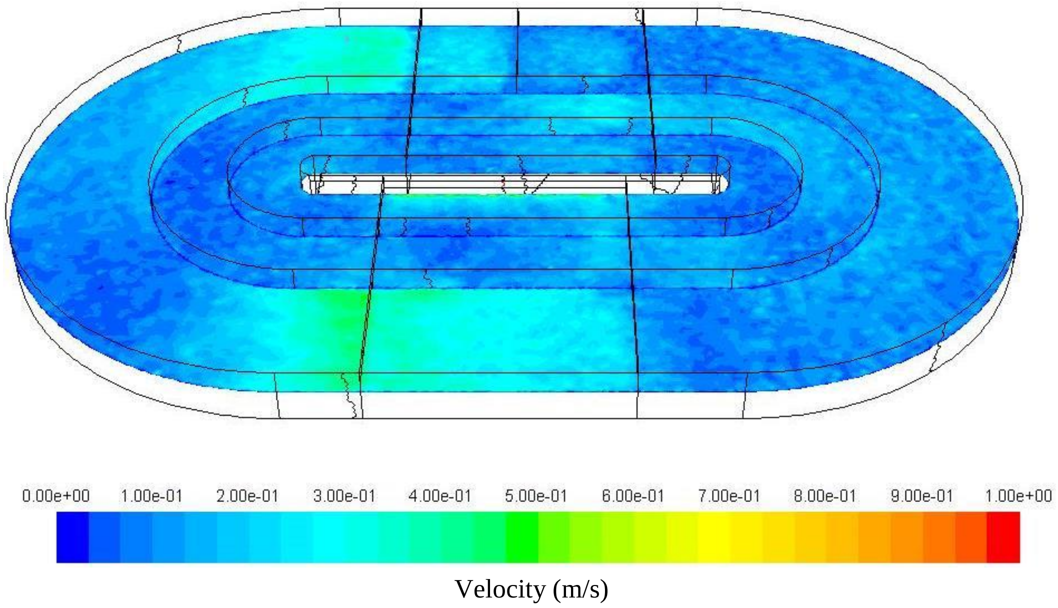


Figure 4.21: Contours of velocity magnitude at 3meter depth

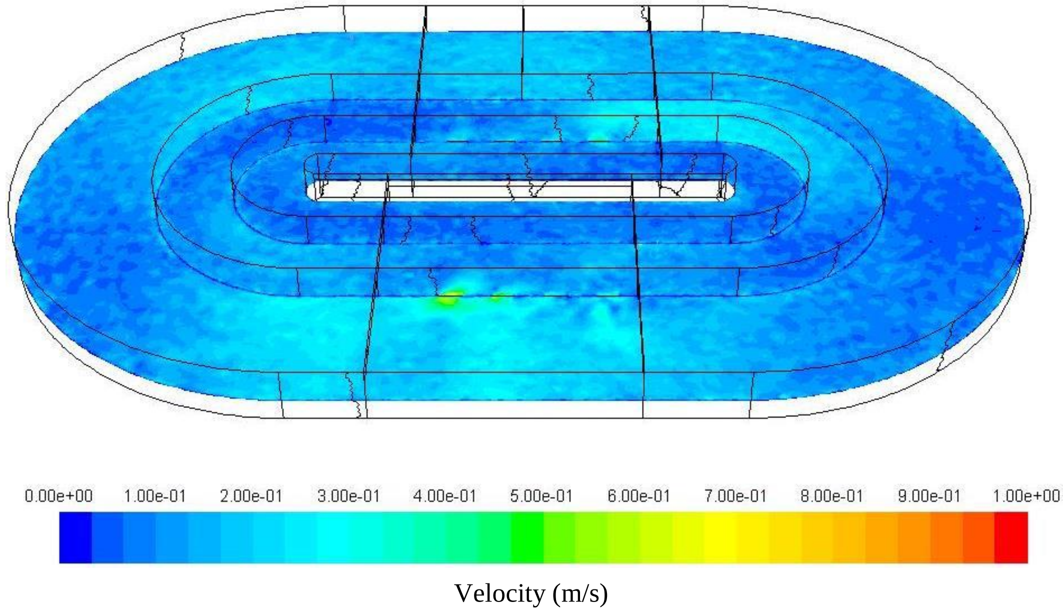


Figure 4.22: Contours of velocity magnitude at 2meter depth

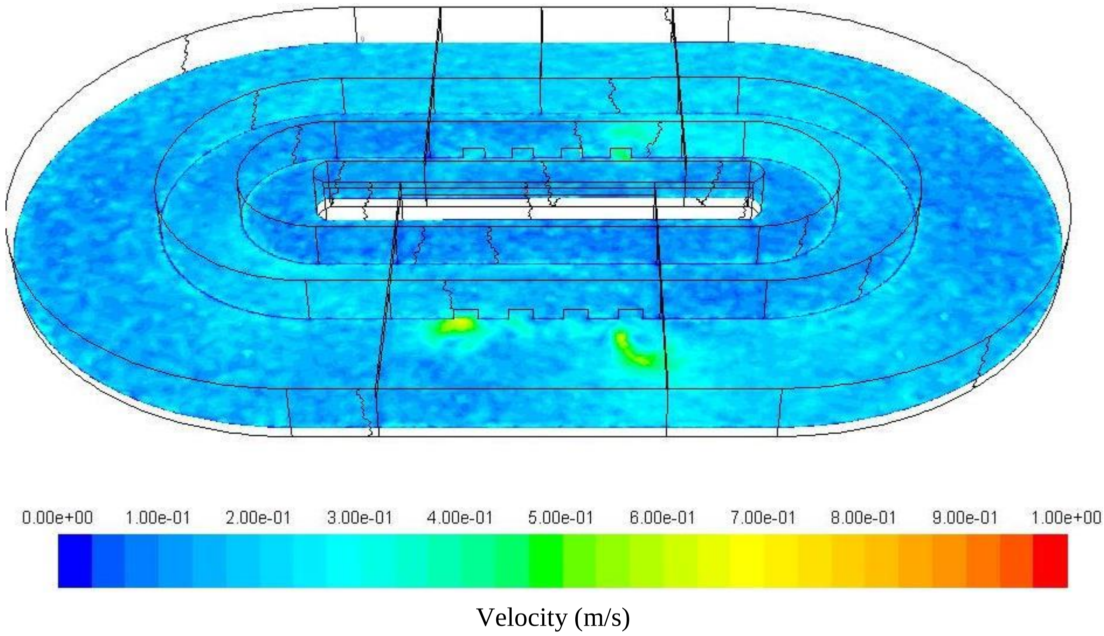
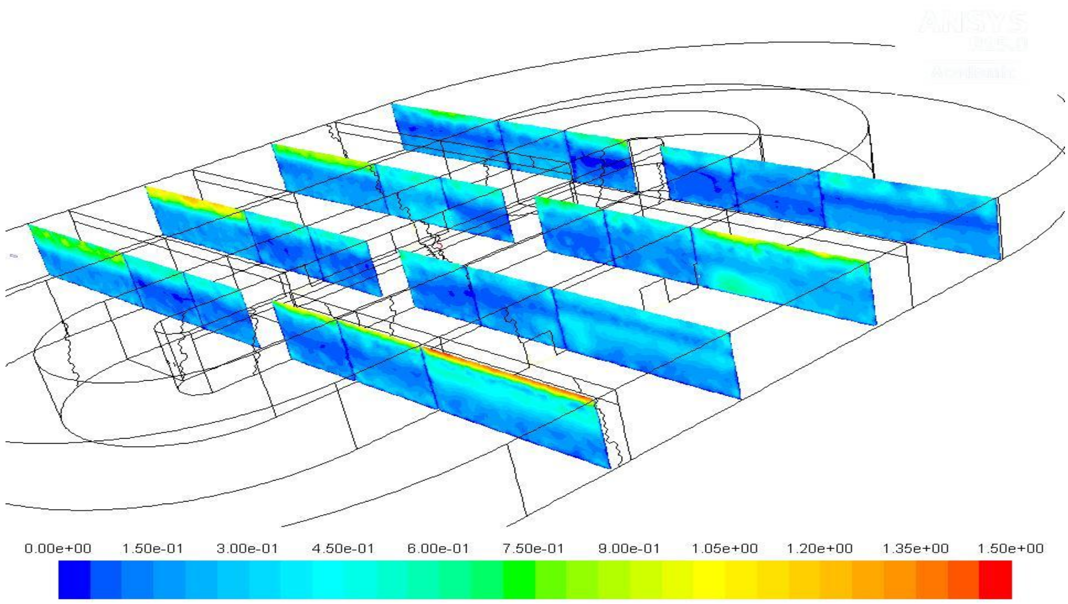


Figure 4.23: Contours of velocity magnitude at 1 meter depth



Velocity (m/s)

Figure 4.24: Contours of velocity magnitude at cross section near to shafts' location

In contrast to the 2D velocity, the maximum velocity obtained was much lesser in the 3D simulation. The scenario is due to the influence of the 3D movement of the fluid, where there are variations in flow velocity throughout the ditch in both vertical and horizontal directions. Furthermore, in 3D model, transverse directions were influenced by the spinning speeds of the aeration discs inside the ditch (Guo et al., 2013), where the rotational plane of the mixers tends to disturb the overall velocity flow of the ditch.

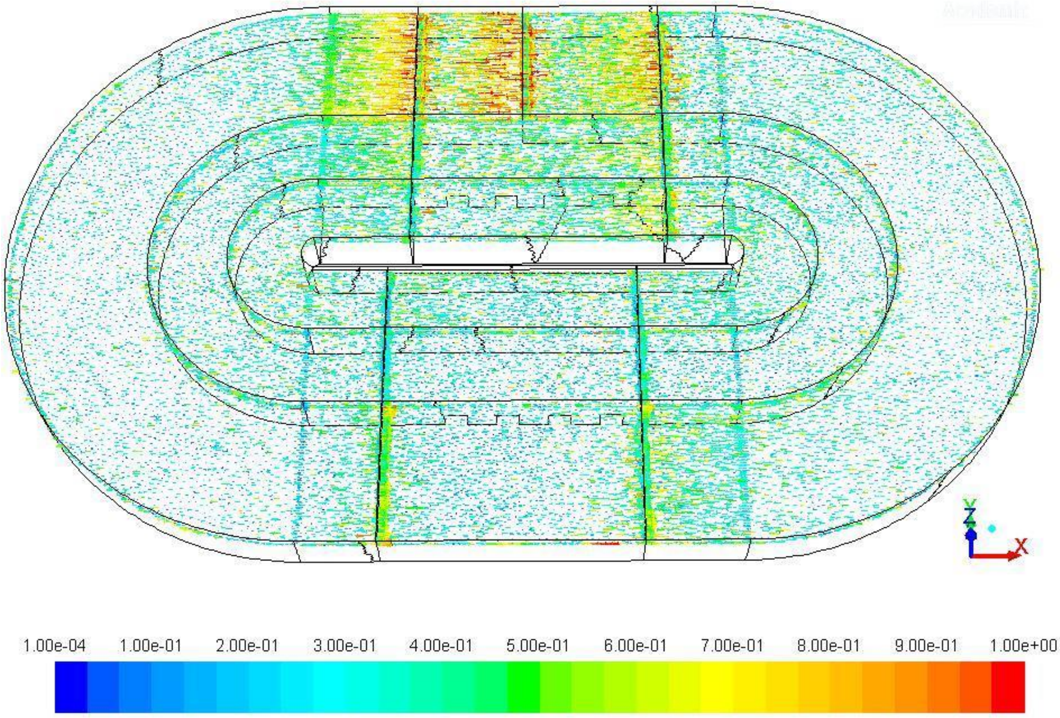


Figure 4.25: Vectors coloured by velocity magnitude at 4 meter depth

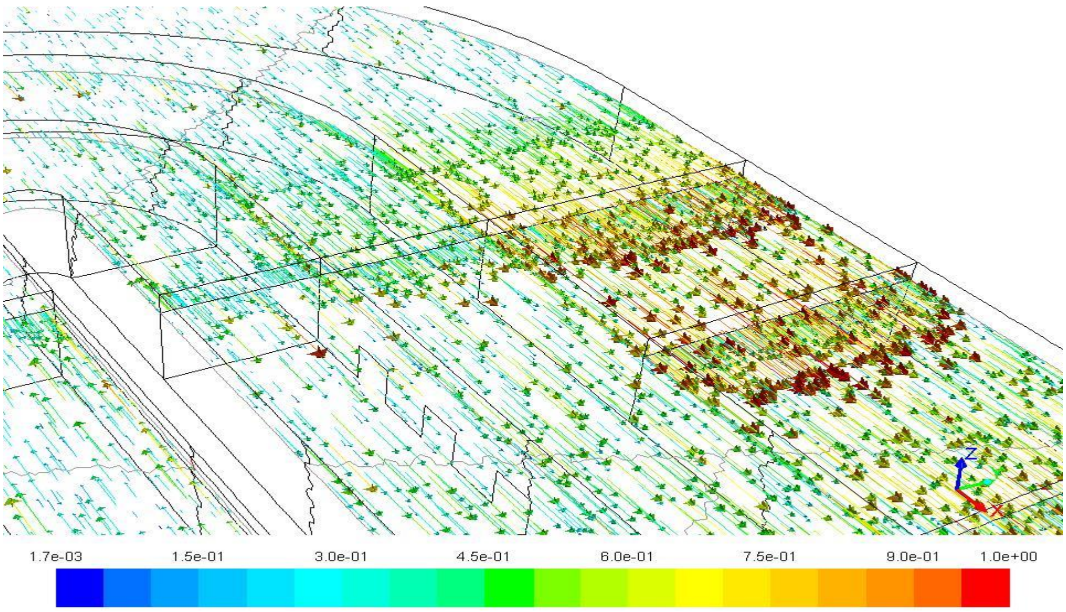


Figure 4.26: Vectors coloured by velocity magnitude at 4 meter depth (near to aeration shafts' location)

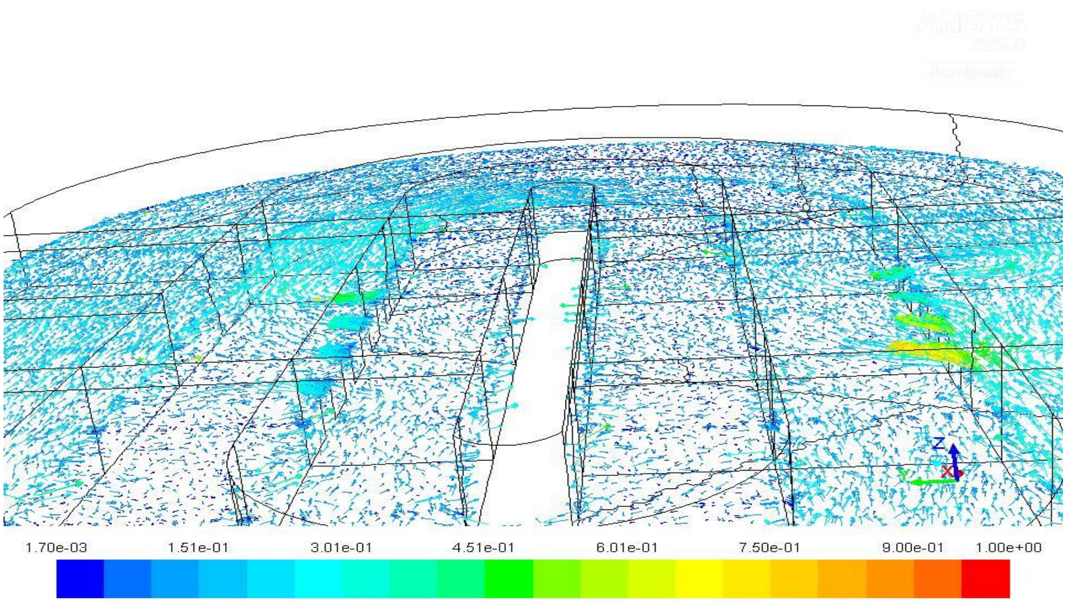


Figure 4.27: Vectors coloured by velocity magnitude at 4 meter depth (near to penstocks' location)

Velocity magnitudes at each sampling point have been extracted and transferred into line graph as demonstrated in Figure 4.28. Based on the simulation outputs, the average flow velocity throughout the ditch was 0.76 m/s. Maximum velocity value was 0.851, while the minimum value was 0.591. Velocity magnitudes all over the ditch were found to be heterogeneous with the highest velocity value revealed at the downstream of the outer channel. This may be related to the highest number of aerations discs inside the outer channel and also caused by the operation of aeration discs at the curve bend before the sampling point of the downstream of outer channel. Downstream of the inner channel has shown the lowest velocity value among all the sampling locations. This may be due to the inertia force and centrifugal force, where the fluid has been dragged towards the weirs and energy loss has occurred (Yang et al., 2010b). The variations of the flow velocity which caused high and low velocity zones inside the ditch have been pointed out by previous workers

(Lei and Ni, 2014, Guo et al., 2013, Yang et al., 2011, Yang et al., 2010b). The overall results have shown an acceptable match to the actual onsite measurements (as shown in Figure 4.29), thereby validating our 3D CFD model. The average deviation between flow velocities simulated by the CFD model and the actual data is 13%. Thus, this advanced CFD model can now be used for more detailed study on hydraulic behaviour of OBS.

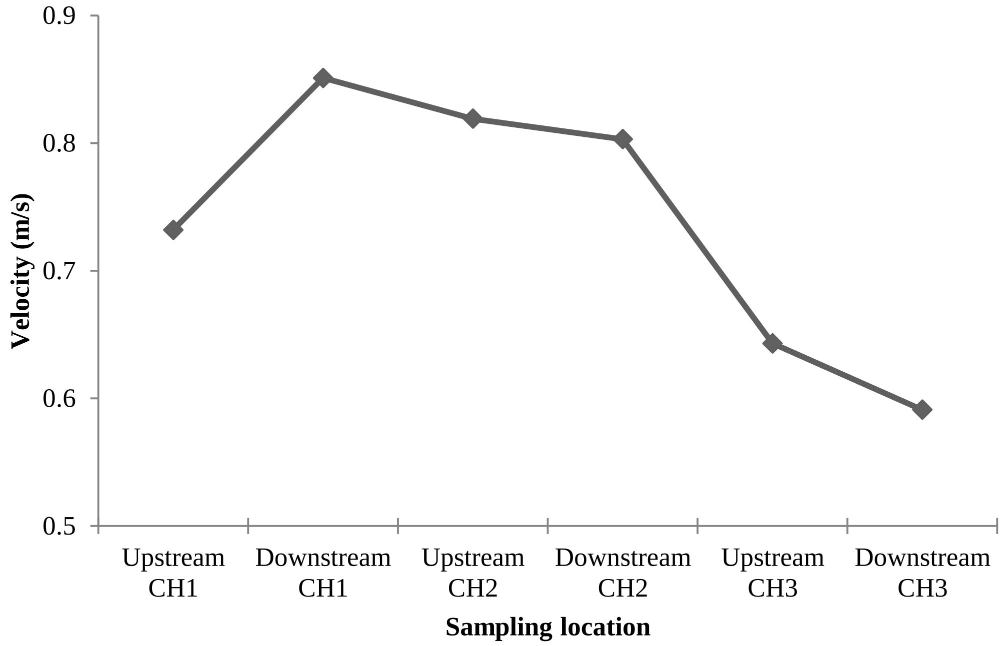


Figure 4.28: Velocity magnitudes given by 3D CFD model

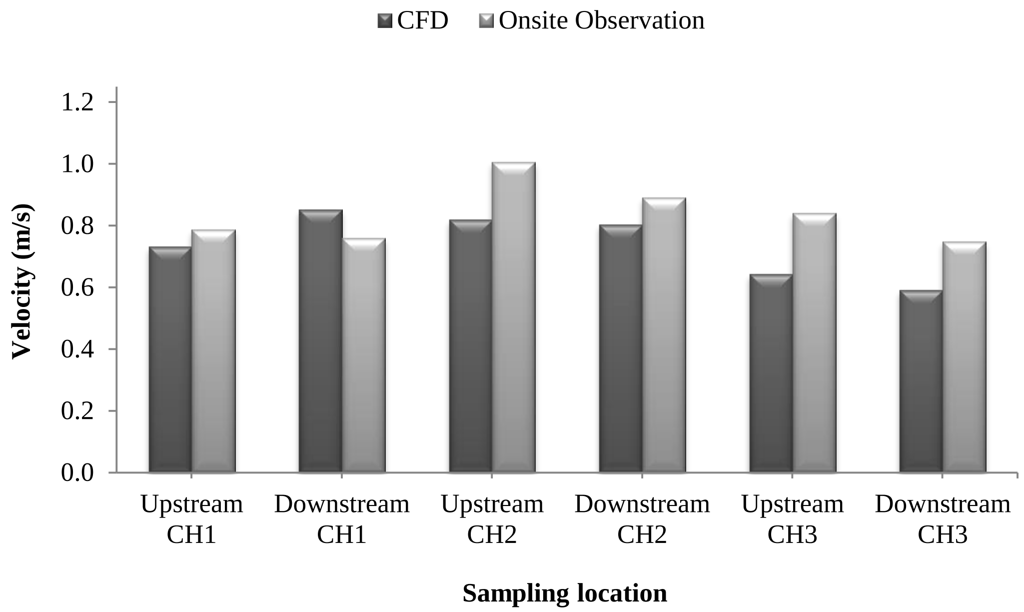


Figure 4.29: Comparison of the velocity magnitude given by 3D CFD model and onsite observation

4.3.4.1 Velocity profiles at different depth

The simulation outputs have also established the overall velocity magnitude at various depths of the OBS as shown in the Figure 4.30. The average velocity at the top (4m height) and bottom (1m height) of the OBS were 0.825m/s and 0.795m/s, respectively. It can be noticed that, the values of velocity at various depths were slightly varied. The velocity at the top of the ditch was higher than the bottom part. However, the velocity values at 1m, 2m and 3m were not much different. This may be attributable to the consistent acceleration produced by the aeration discs. This means that the current submergence of the aerations discs has given a good mixing pattern where the velocity values at 1m, 2m and 3m are almost consistent.

Generally, the solids build up at locations where the velocities are relatively low (Xie et al., 2014). The higher velocities at the bottom of the ditch may possibly help to keep solids in suspension. Since the mixers were producing a good mixing pattern where the velocities in vertical directions were not much varied, maybe another aspect that can be focussed on is the speed of the aeration discs. The speed of the aeration discs can be increased to prevent the sludge settling at the bottom of the ditch. As pointed out in the previous research, increasing the rotational speed of the aeration discs will decrease the variations of the flow speed (Guo et al., 2013). Onsite measurement makes it almost impossible if not difficult to gather velocity distribution at different depth of wastewater level. Now, using the 3D CFD model, the velocity profiles at different depth of the system are easily measurable.

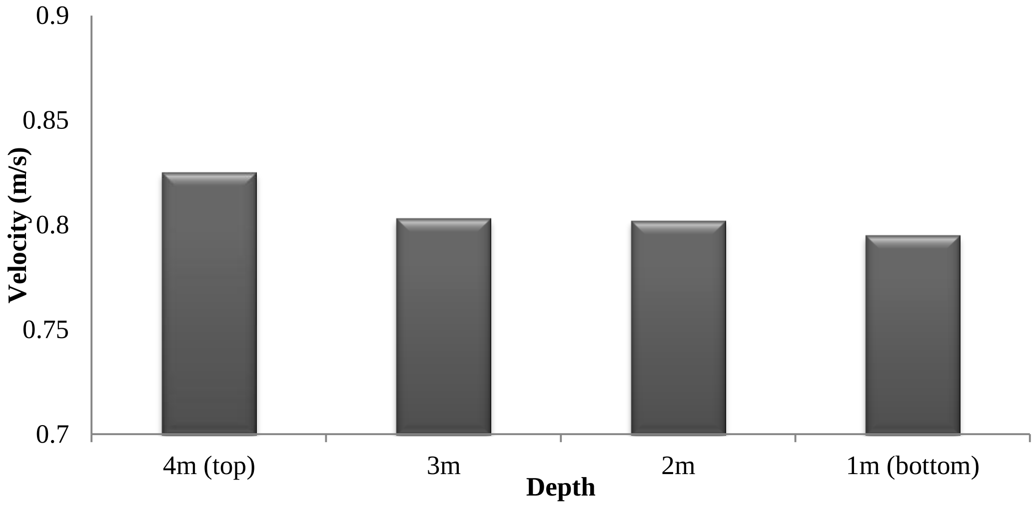


Figure 4.30: Average velocity magnitude at different depth

In addition to the velocity profile and vector plots, CFD can be used to extract the vorticity profiles observed at different depths. Vorticity is referring to the local fluid element rotation and in this case can attribute to the action of rotating shafts. This is very important in mixing which is the objective of the wastewater tank used in this study. Figure 4.31 to Figure 4.36 present the vorticity magnitude at various depths and cross sections inside the oxidation ditch. It is evident from this figure that the vorticity magnitude is at maximum near the vicinity of the shafts which indicates that shafts provide better mixing of the fluid in the ditch. The vorticity magnitude decreases substantially with depth and is found predominant at the penstock and the area attached to the wall curvature.

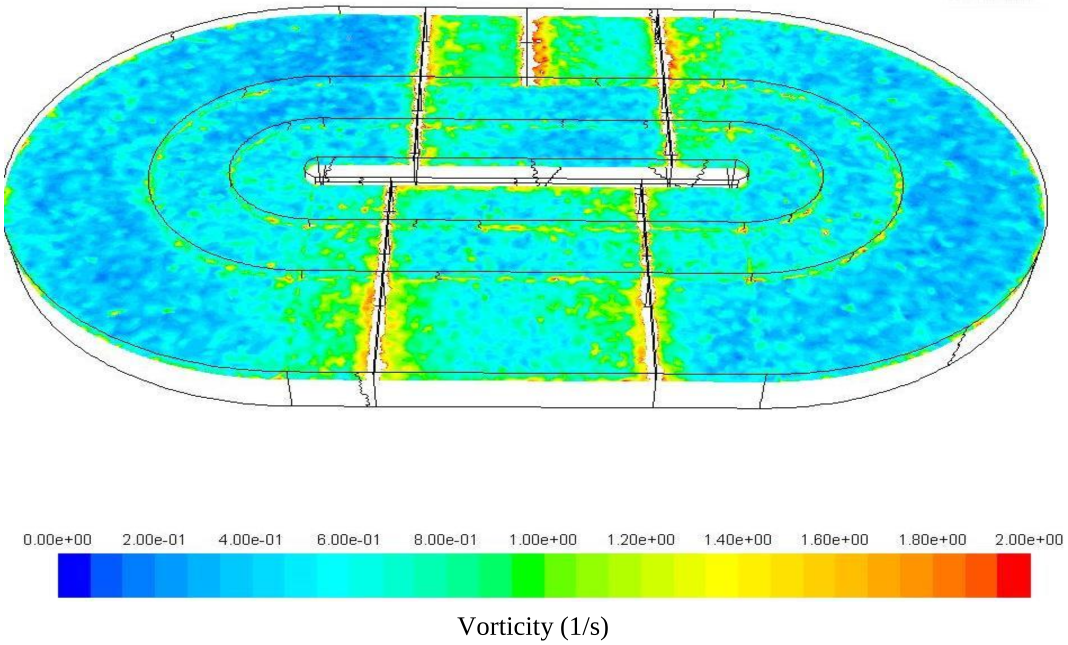


Figure 4.31: Contours of vorticity magnitude at 4 meter depth

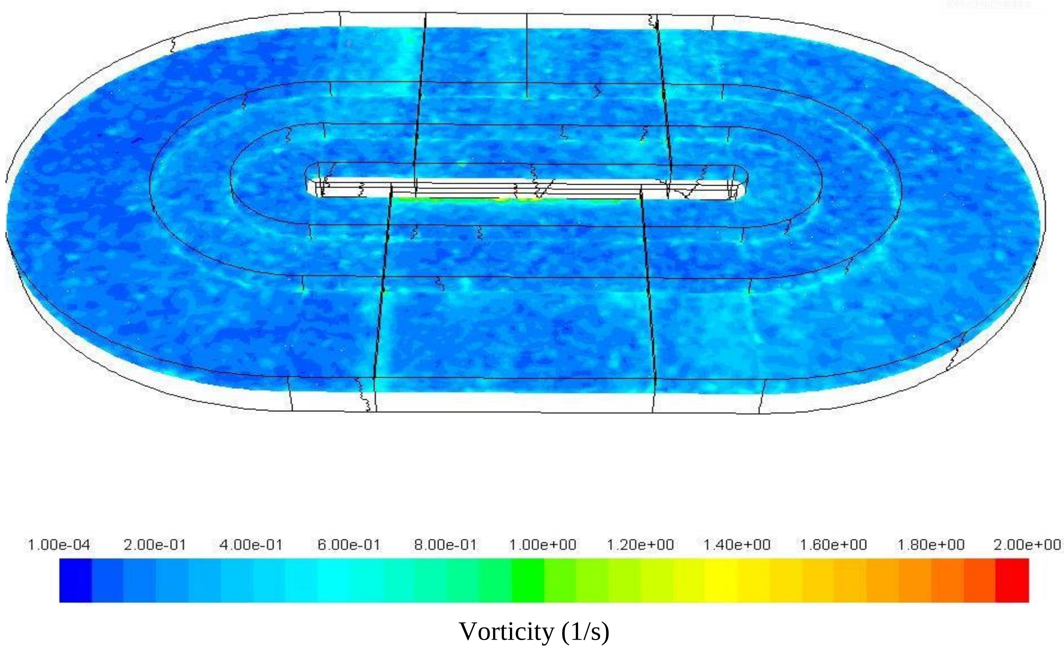


Figure 4.32: Contours of vorticity magnitude at 3 meter depth

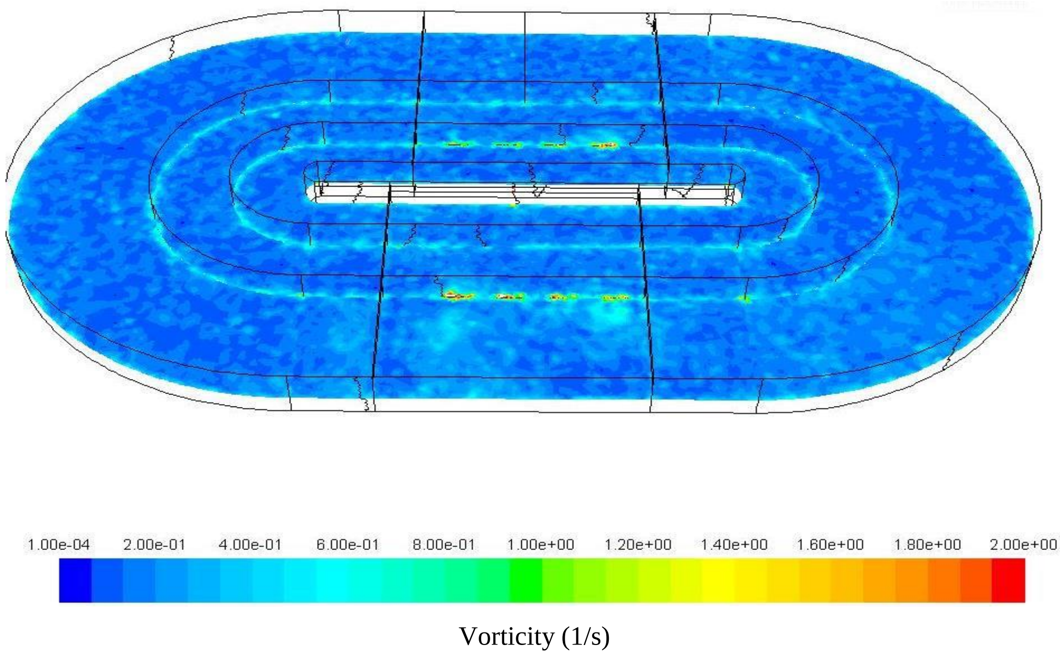


Figure 4.33: Contours of vorticity magnitude at 2 meter depth

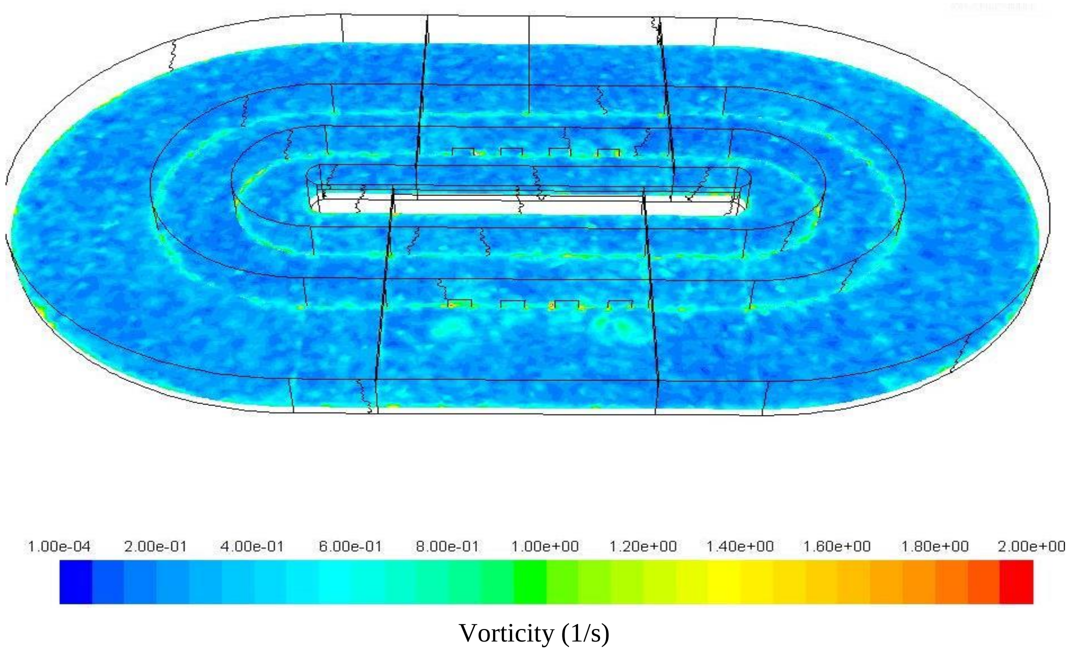


Figure 4.34: Contours of vorticity magnitude at 1 meter depth

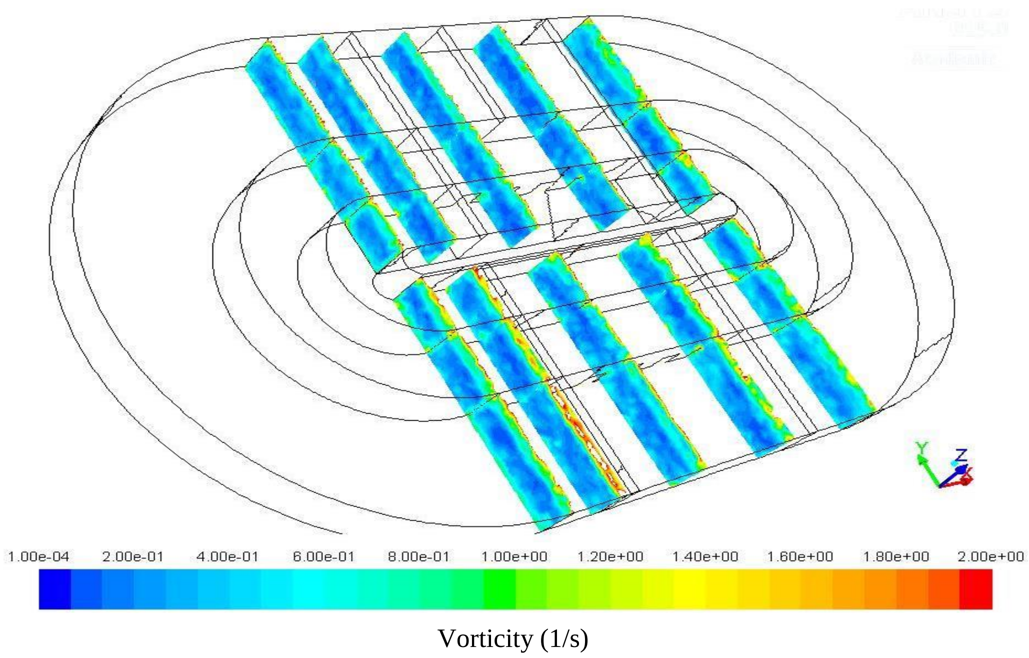
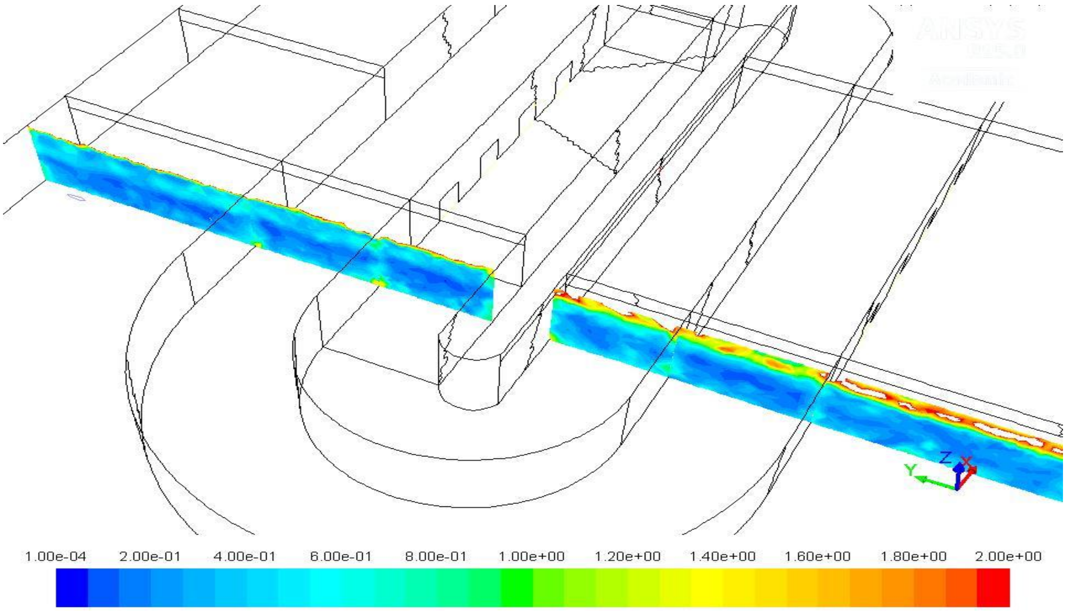


Figure 4.35: Contours of vorticity magnitude at 4 meter depth (at different cross sections)



Vorticity (1/s)

Figure 4.36: Contours of vorticity magnitude at 4 meter depth (at cross sections focused on the differences of vorticity given by the shafts)

4.3.5 Hydraulic Residence Time from CFD Model

A main factor, which makes CFD enhanced than the preliminary process model, is its aptitude to visualize the distribution of the HRT throughout the channels. Another major issue with respect to theoretical estimation is that the water that exits the OBS, returns again to the OBS channels and this process repeats itself for several times. Because of that reason, it is difficult to estimate the value of HRT as well as the OUR values in each channel, especially when using the conventional mathematical calculations. Particle tracking using CFD is a useful tool to simulate HRT in aeration tank (Karpinska and Bridgeman, 2016, Le Moullec et al., 2008) such as OBS.

HRT values given by CFD are shown in Table 4.4. The values were obtained by injecting particles, which had properties same as water, into the OBS simulation system. Around 1,000 particles were injected to determine the distribution of hydraulic residence times throughout the channel. According to previous researchers, a significant residence time distribution can be achieved on condition that enough particles trajectories are calculated. (Le Moullec et al., 2008). Number of particles being tracked is subjected to computer memory limitations and computational time limitations.

Karpinska and Bridgeman (2016) have also highlighted that this approach is the most expensive, involving long computational times and demanding a large number of CPUs (Central Processing Units). In the case of a full-scale OBS, tracking too many particles will consequence in great computational time. Due to computer memory limitations and computational time limitations, particles tracking approach is not often being applied in the simulation of wastewater treatment. Therefore, the simulation approach and results of this research will become a value added to wastewater treatment modelling.

Table 4.4: Summary of HRT values based on CFD model

Channel	Maximum HRT value (minutes)	Average HRT value (minutes)	Minimum HRT value (minutes)	Standard Deviation (minutes)
Outer	301	177	74	43
Middle	251	125	14	49
Inner	199	71	5	39

4.3.6 Volume Flow Rate and estimation of OUR values

More specific HRT values given by CFD simulation have been utilized to get the volume flow rate for the outer, middle and inner channels. The volume flow rates for each channel are calculated as given in Table 4.5.

Table 4.5: Volume flow rate for each channel

Channel	Volume Flow Rate (m^3/d)
Outer	52 447
Middle	22 431
Inner	21 504

Using the calculated volume flow rate, OUR values for each channel are calculated using Equation 3.1 as introduced earlier in chapter three. The calculation was performed using the data of 20 sampling days. The overall results of the calculation performed using the outputs given by CFD model are revealed in Figure 4.37. Detailed calculations are attached in Appendix G. Based on the results of ANOVA (refer to Table 4.6), the maximum OUR value for the outer channel is 35,818kg/d, whereas the maximum value for the middle and inner channel is only 6,729 kg/d and 6,749 kg/d, respectively. The average value of outer channel is calculated as 21,109 ± 7909 kg/d, while the average value of the middle and inner channel are calculated as 2,456 ± 1620 kg/d and 3,035± 1681kg/d correspondingly. For the purpose of further analysis, the average OUR values are used to represent OUR value of each channel.

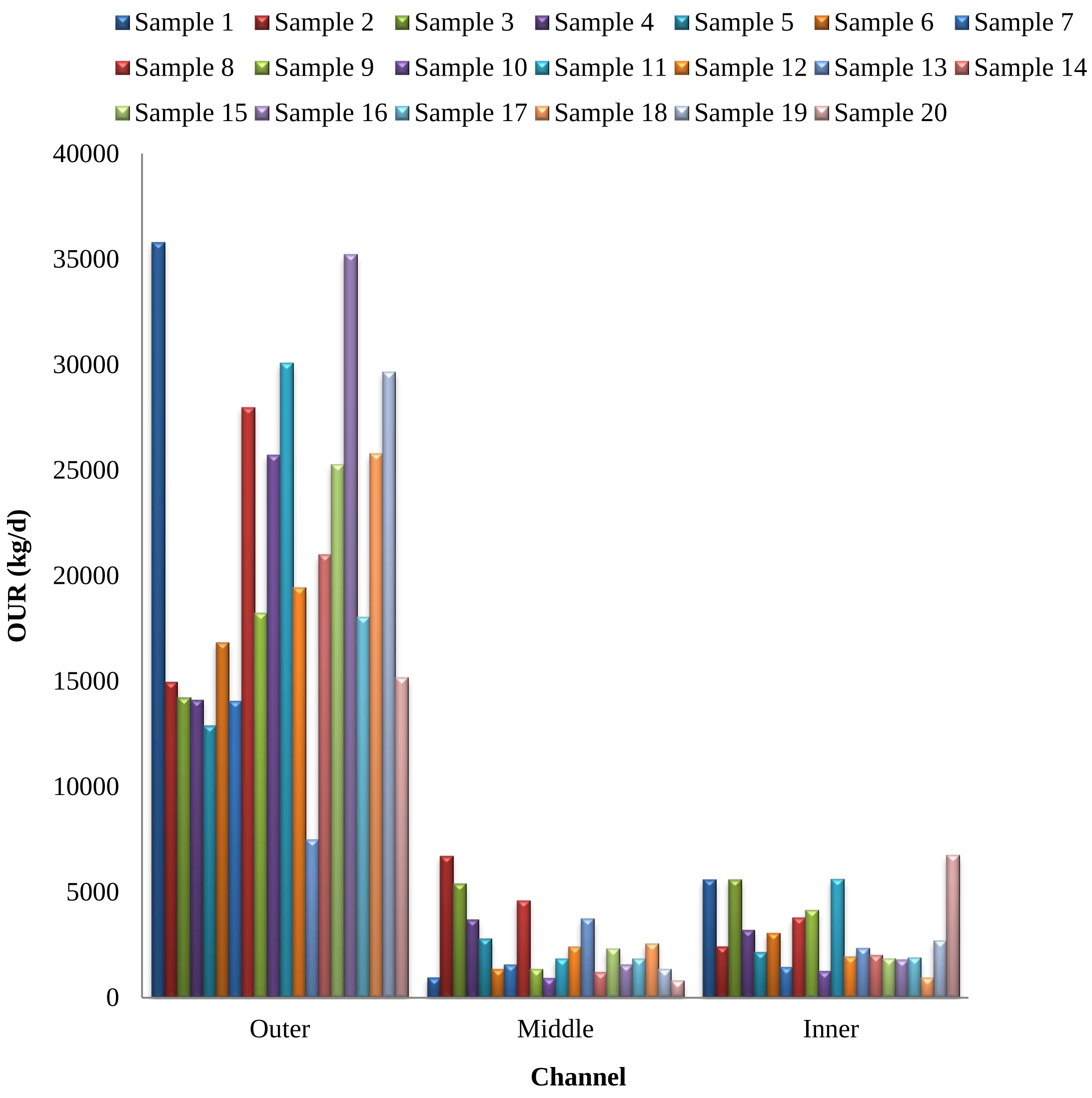


Figure 4.37: OUR values (kg/d) throughout the channels

Using the average OUR values, a pie chart has been plotted as shown in Figure 4.38. The chart shows that the outer channel takes the highest portion of the overall oxygen consumption, which is 79 % compare to the middle channel with only 9 % and followed by the inner channel with 12 %. Based on the literature, the actual oxygen demand of the outer channel might be as high as 75% of the total oxygen consumption. Since the outer channel is where the raw wastewater and RAS first enter the OBS, the majority of the process takes place in this channel. In comparison to the middle and inner channel, most of the nitrification process happens in outer channel, while the inner channel is just keeping the channel in a polish mode to eliminate any remaining BOD and ammonia (Evoqua Water Technologies, 2015)before the flow exits the system.

Table 4.6: ANOVA for CFD model

SUMMARY					
Groups	Count	Sum	Average	Variance	std dev
Outer	20	422170	21108.5	62552186.4	7908.994017
Middle	20	49115	2455.75	2623398.09	1619.69074
Inner	20	60701	3035.05	2825174.05	1680.825407

ANOVA						
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	4499401773	2	2.25E+09	99.2504026	2.70242E-19	1.655749
Within Groups	1292014412	57	22666920			
Total	5791416185	59				

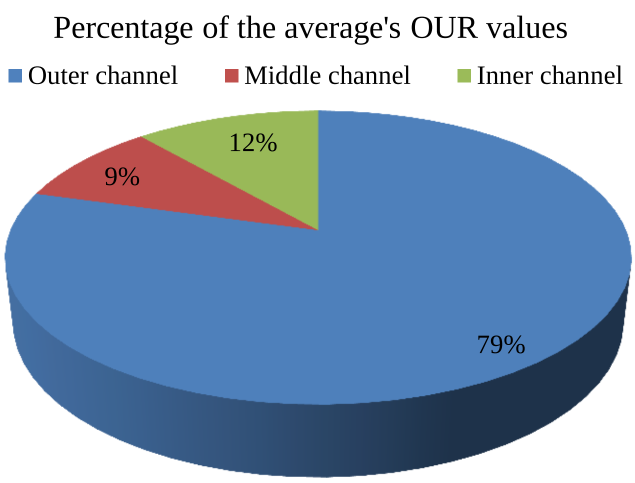


Figure 4.38: Percentage of OUR values

4.3.7 The effect of different operating shafts on the performance of OBS

Not only velocity profiles at different depth, CFD application also provided quantified velocity values at different sampling locations inside the OBS. The main advantage of CFD application is that, parametric studies can be carried out and their performance can be assessed without having to do any experimental modifications. It is well known that the OBS are energy drains and the operations cost involved are prohibitively large. Furthermore, it consumes a lot of power in order to operate the 9 mixing shafts inside the oxygen ditch. Therefore, in this study, parametric evaluation of the oxygen consumption is carried out by shutting down operational shafts to study their effect on the performance of the OBS. Study on aerators could evade the condition of back mixing and could resolved the problem of sludge deposit and the low velocity in the bend which due to the attenuation of the drive power. Since the aeration devices are crucial components of OBS, different numbers of operating shaft have been studied.

In this work six cases representing different operating conditions of the aeration shafts were set up as described in Table 4.7. Case 1 represents the CFD model with the second shaft (shaft 2) of the outer channel being turned off. Since the flow entering the first channel was supported by the first aeration shaft (shaft 1) and then the flow momentum was kept by shaft 3, so by turning off the second shaft it is hypothesised that it may not have a major impact on the overall flow of the outer channel.

Case 2 involved shutting down of shaft 6 of the inner channel, whereas case 3 involved shutting down of shaft 9 of the inner channel. In general, the case 1, case and case 3 represent shutting down one channel each of the outer, middle and inner channel respectively. Whereas case 4 involves blanking 2 shafts (shaft 2 of the outer channel and shaft 6 of the middle channel). Similarly, case 5 blanks out shaft 2 of the outer channel and shaft 9 of the inner channel. The last case studied was case 6 which involved shutting down 3 shafts (shaft 2 of channel 1, shaft 6 of channel 2 and shaft 9 of channel 3), thereby allowing only 6 shafts to operate out of a total of 9 shafts. Shaft 6 and shaft 9 were chosen because of their location at the downstream of their own channel (as shown in Figure 4.39).

Table 4.7 : Six cases of different operating condition

Channel	**Shaft**	**Case1**	**Case2**	**Case3**	**Case4**	**Case5**	**Case6**
Channel 1	Shaft 1	√	√	√	√	√	√
	Shaft 2	**X**	√	√	**X**	**X**	**X**
	Shaft 3	√	√	√	√	√	√
	Shaft 4	√	√	√	√	√	√
	Shaft 5	√	√	√	√	√	√
Channel 2 &3	Shaft 6	√	**X**	√	**X**	√	**X**
	Shaft 7	√	√	√	√	√	√
	Shaft 8	√	√	√	√	√	√
	Shaft 9	√	√	**X**	√	**X**	**X**
Total shafts in operation		**8**	**8**	**8**	**7**	**7**	**6**

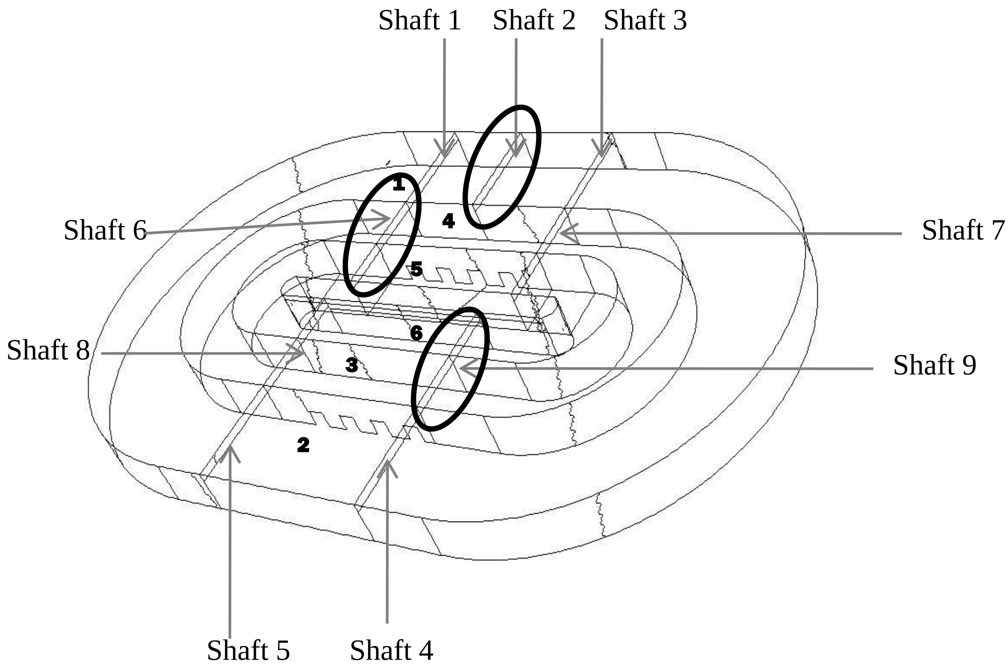


Figure 4.39 : Shafts locations

The comparison of flow velocities for different operating conditions is given in Figure 4.40. In the real case, where all the aerators were running, the mean velocity throughout the ditch was 0.74. This is the highest mean velocity throughout the ditch. This scenario is expected due to the acceleration caused by all the mixers in the channels. Since the aeration discs are also keeping the water flowing along the ditch and flowing from one channel to another, the case with all the mixers turned on is expected to show the highest velocities among all the cases. In case 1, when shaft 2 inside the outer channel was turned off, the mean velocity has dropped by 7.4%. The least reduction of mean velocity (7%) was demonstrated by case 2, where shaft 6 inside the middle and inner channel were turned off. Among all the cases, the highest reduction of mean velocity for about 13.6% was found in case 6. In this case one shaft from outer channel and 2 shafts from middle and inner channel were turned off. Minimum flow velocity in all cases was not less than 0.4 m/s. The minimum flow velocity obtained for each case is still considered acceptable to support the overall mechanisms of OBS.

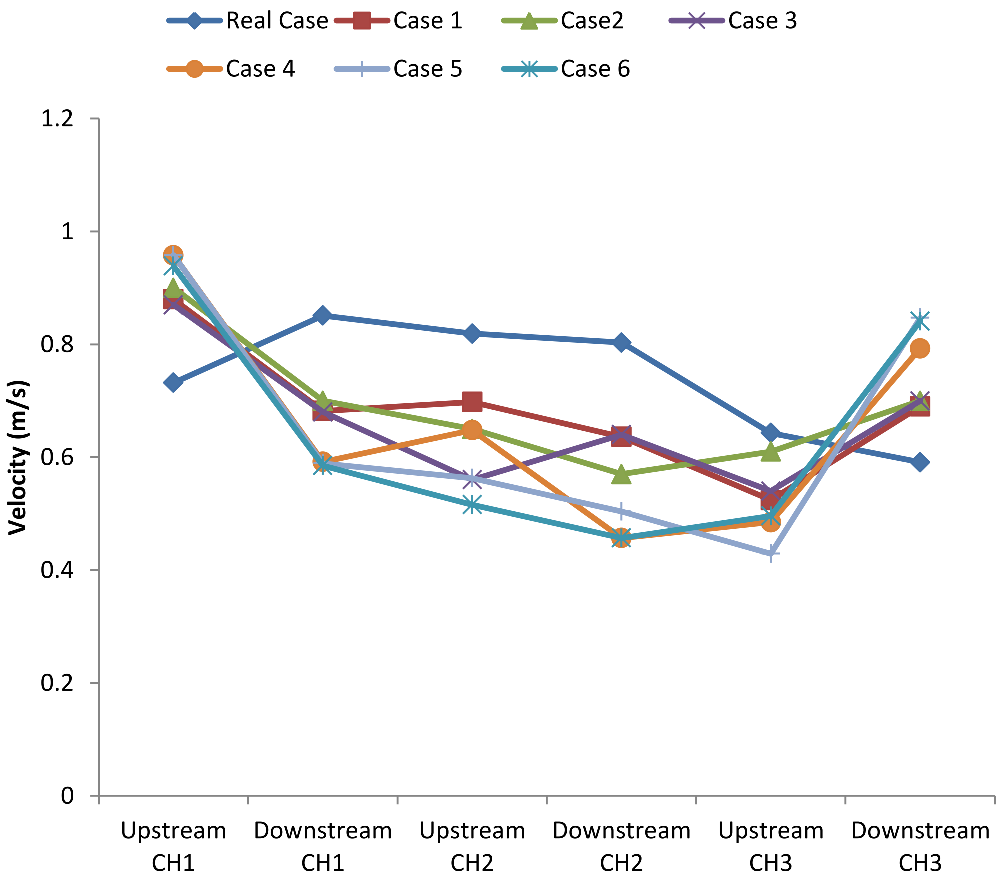


Figure 4.40 : Velocity values for different operating conditions

4.3.7.1 Hydraulic Residence Time for different operating conditions

The summary of HRT values for different operating conditions of OBS is given in Table 4.8. Based on the results, the highest HRT value is shown in case 1 with 334 minutes, while the lowest HRT value is 262 minutes in case 3.

Table 4.8: Summary of HRT values for different operating conditions

Case	HRT values (minutes)
Case 1	334
Case 2	310
Case 3	262
Case 4	282
Case 5	275
Case 6	317

4.3.7.2 OUR values and power consumption for different operating conditions of OBS

OUR values for different operating conditions of aeration devices operational system is given in Figure 4.41.Based on the graph, the highest OUR value is 28,732 which is in case 3. Case 6 has shown the lowest OUR value which is 24, 346 kg/d. Among all the cases, case 6 is having the closest OUR value to the real case where all the shafts of aeration discs are turned on. This shows that even though in case 6 three shafts have been turned off, the oxygen consumption of the ditch can still be satisfied.

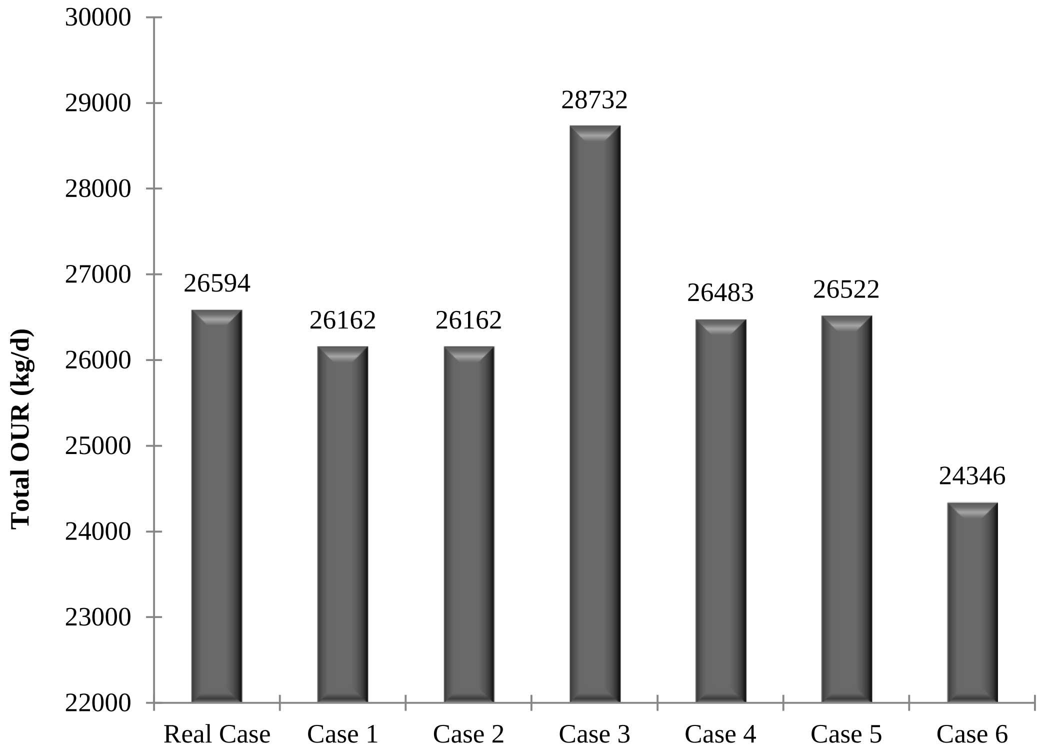


Figure 4.41: Summary of OUR (kg/d) values for different operating conditions

Figure 4.54 shows the summary of power consumption for different operating conditions of OBS. The graph shows that the highest amount of power is consumed in case 3 with the value of 1330 kW. Case 6 has shown the lowest power consumption which is 1127 kW.

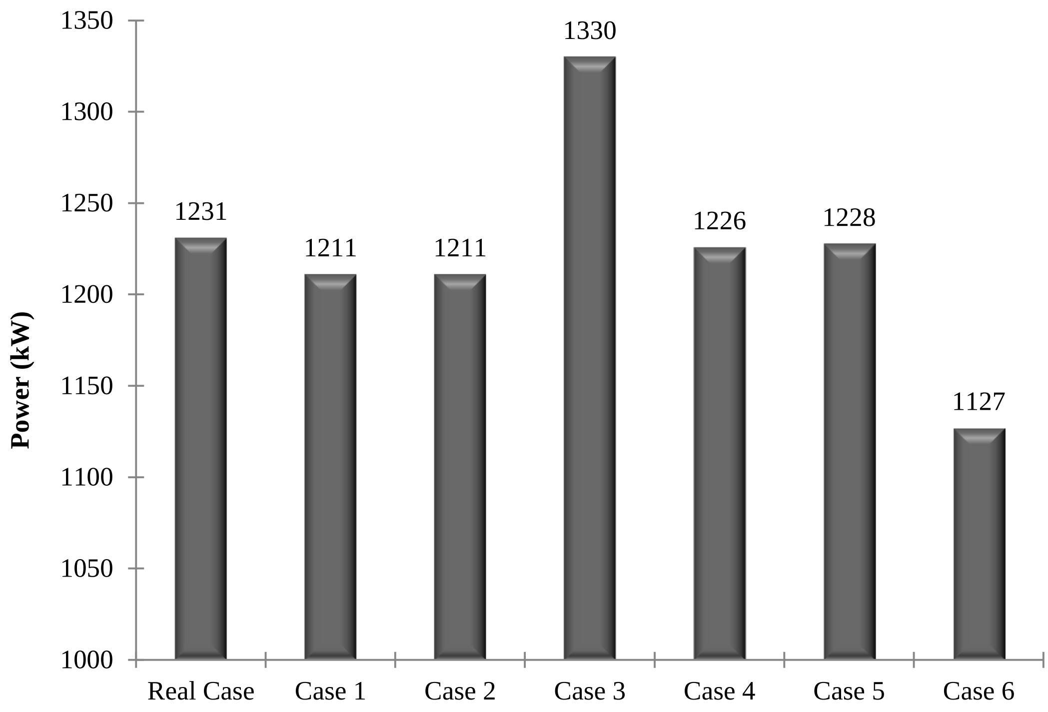


Figure 4.42: Summary of power consumption (kW) for different operating conditions

4.4. Impact of more stringent effluent water quality targets on energy consumption by the OD

Before analysing the impact of more stringent effluent quality targets on energy consumption by the OD, power consumption associated to the current oxygen utilization rate (OUR) were calculated. Power consumptions are calculated based on OUR values and in process oxygenation efficiency for mechanical aeration devices. OUR values are calculated in the previous sections using more specific HRT values for each channel as given by CFD. The overall results of the calculation performed using the outputs given by CFD model are shown in Figure 4.43.

Figure 4.55 describes how much power is consumed to attain the current effluent discharge standards imposed by the government. The results are demonstrating the power consumption of the OBS when the system is being operated to perform BOD, and SS reduction to an acceptable level as listed in the Third Schedule of the Environmental Quality Act 1974, under the Environmental Quality (Sewage and Industrial Effluents) Regulations 1979, regulations 8 (1), 8 (2) and 8 (3). Outer channel is obviously the highest consumer of the power among the channels. Aeration discs in the outer channel consume an average of 977 kW of electricity for its 21109 kg/d of oxygen consumption. Based on the results, outer channel consumed 79% of the total power consumption of the OBS, while the middle and inner channel consumed 9% and 11% respectively. This is parallel with the fact that the majority of the reactions occur in the outer channel. Most of the nitrification process takes place in this channel.

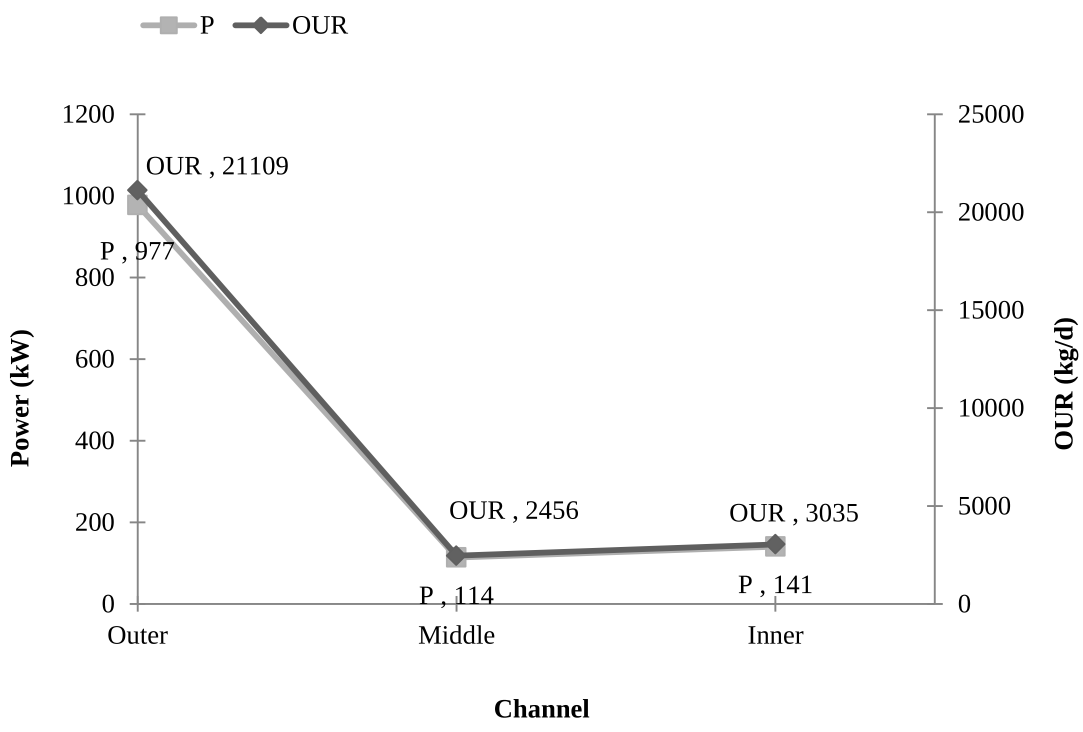


Figure 4.43: Power consumption of the current OUR values

4.4.1. Amount of power consumption associated with improved effluent quality

Untreated wastewater contains a significant amount of biodegradable organic matter, suspended solids and ammoniacal nitrogen loads. Significant amount means they may have adverse effects on the quality of the country's watercourses. Currently, all STPs in Malaysia are enforced to produce final effluents having BOD_5 and SS values less than or equal to the average values as shown in Table 4.9. However, based on the pressure received by wastewater treatment industry, in future these effluent standards are expected to be more stringent. This will probably increase the energy consumption of the treatment system. It is important to study the impact of more stringent effluent standards to ascertain that the decision made will not cause significant unfavourable effects.

Once ammoniacal nitrogen reduction becomes a compulsory task, operators of the treatment plants have to make sure that their treatment systems are surrounded by an adequate environment, which can support the process. The most important criterion is to fulfill the oxygen demand for the nitrification process. Nitrifying bacteria present in the wastewater will consume the oxygen dissolved in the wastewater to convert ammonia into nitrite and then into nitrate. Increasing of oxygen utilization indicates that more oxygen is supplied into the system in order to achieve the targets of the treatment process. Supplying more oxygen will most likely increase the power consumption by the aeration devices. The question here how much more power will be consumed for more stringent effluent quality targets? In order to answer this question, we at least need to know how to relate the current OUR of the ditch and the overall power consumption in order meet the current requirement. Then, we will able to predict what will be the OUR and power consumption in case of more strengthen effluent standard will be imposed in the future.

Amount of the current power consumption is the reflection of the current effluent standard requirement. The effluent quality at this site has to meet with the Standard B of the Environmental Quality (Sewage and Industrial Effluents) Regulations 1979 under Environmental Quality Act (EQA) 1974. In order to meet the existing effluent requirement, OUR values as shown in Figure 4.37 are needed and the amount of power as shown in Figure 4.43 is consumed.

Table 4.9: Effluent Permitted Level of BOD_5, and SS (Department of Environment, 2009)

Parameters	Unit	Maximum Permitted Value
BOD_5 at 20˚C	mg/L	50
Suspended Solids	mg/L	100

In future, in case more stringent effluent water quality standard will be imposed, the amount of the OUR as well as the power consumption of the treatment system will also increase. The estimation of the OUR values can be performed using formulae 3.1 as used in this dissertation. For each 5% increment of the current OUR values will cause an increasing of power consumption. The ranges of power consumption that are related to the increment of OUR values are demonstrated in Figure 4.44 to, Figure 4.49.

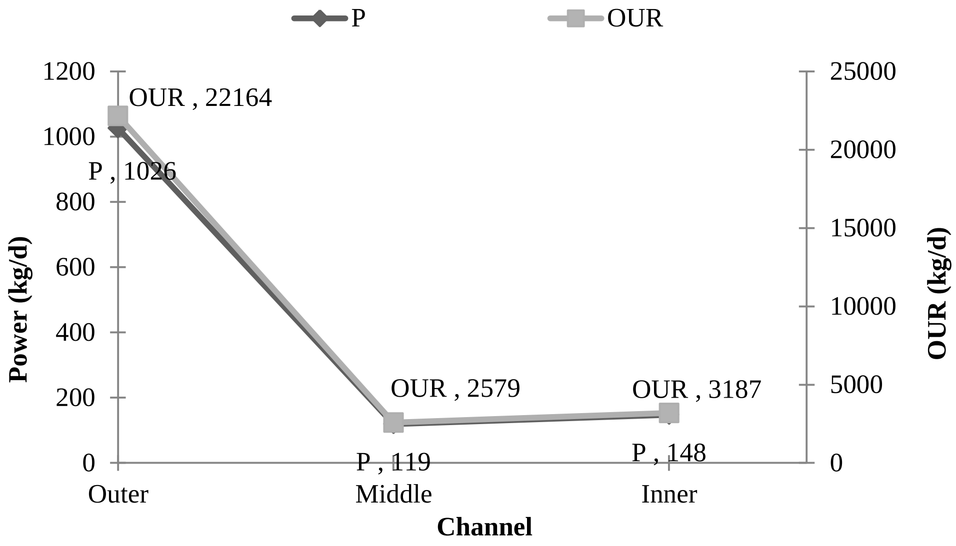


Figure 4.44: Power consumption of 5 % increment of current OUR

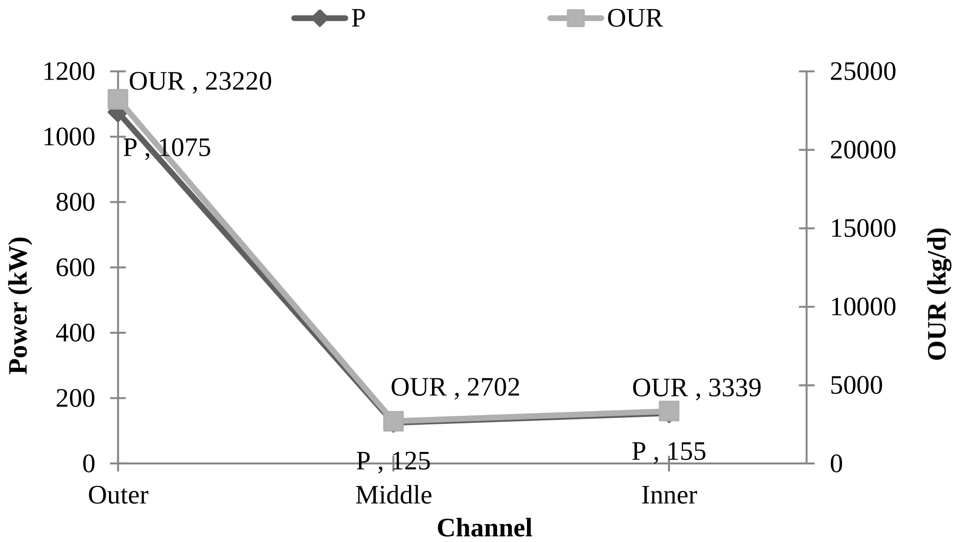


Figure 4.45: Power consumption of 10 % increment of current OUR

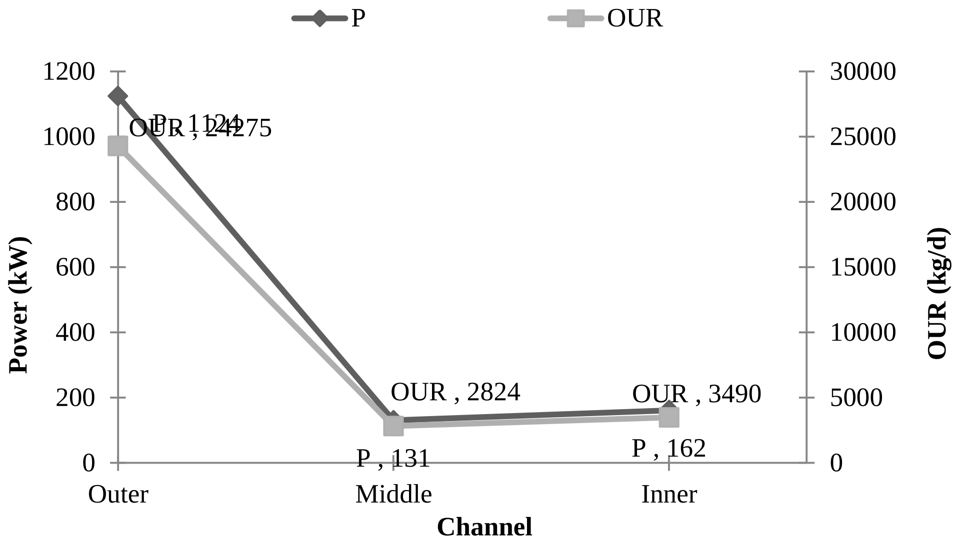


Figure 4.46: Power consumption of 15 % increment of current OUR

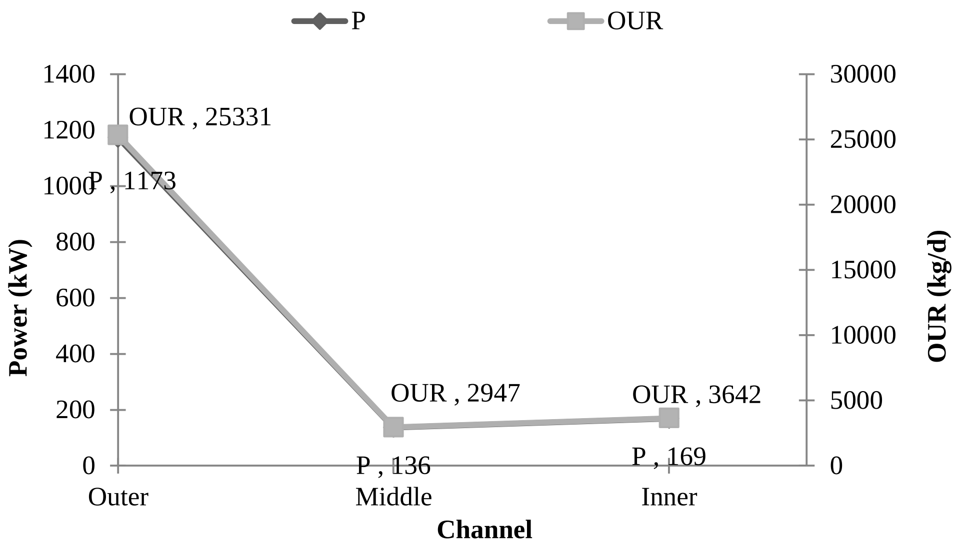


Figure 4.47: Power consumption of 20 % increment of current OUR

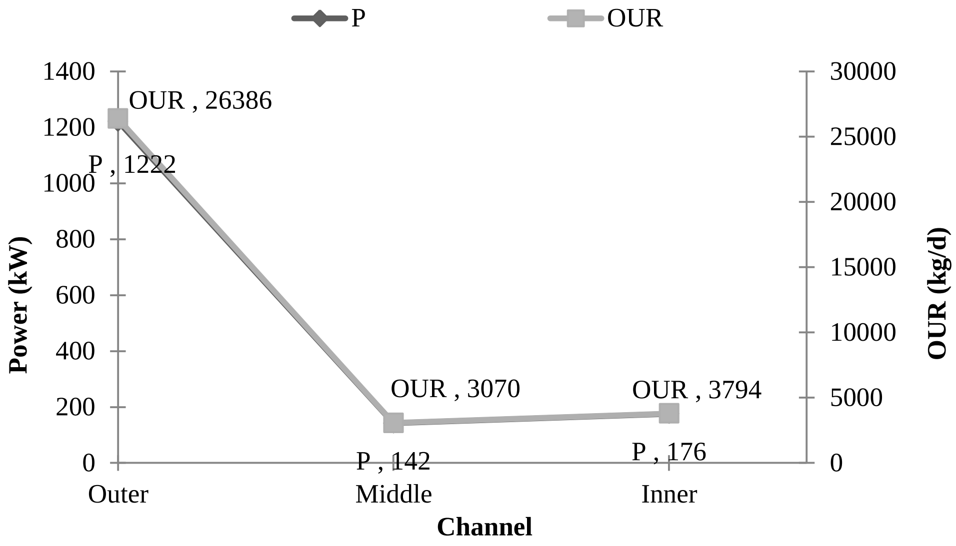


Figure 4.48: Power consumption of 25 % increment of current OUR

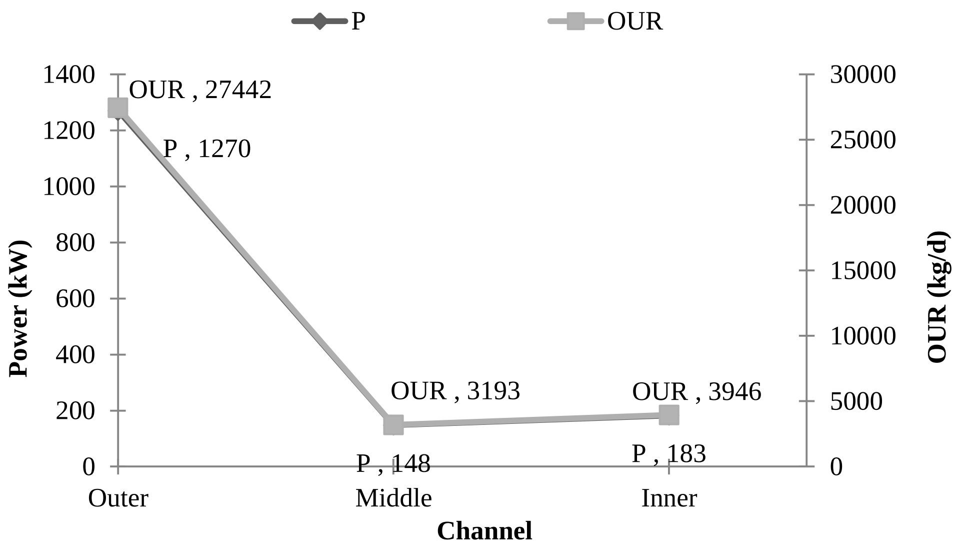


Figure 4.49: Power consumption of 30 % increment of current OUR

The increment of power consumption of the OBS can be clearly seen (as shown in Figure 4.50), especially for the outer channel. The total estimated power consumption for 30% increment of OUR values is 1601 kW. Compared to the power consumption of the current OUR values, roughly it will be an additional of 370kW in an hour. This will cost an additional of 3, 241MW of power in a year for only the biological treatment system. This still does not include the power consumption of the treatment process of the overall wastewater treatment system.

Generally, wastewater treatment system is able to reduce their energy costs more than 30% through the measurement of energy efficiency and modifications of treatment process (Stillwell et al., 2010a). However, the reduction of energy consumption may vary from one treatment plant to another depending on some factors such as type of aeration process or aeration devices involved in the treatment. Furthermore, other factors such as energy resources and energy costs can also be the reasons of variation in energy saving between some countries.

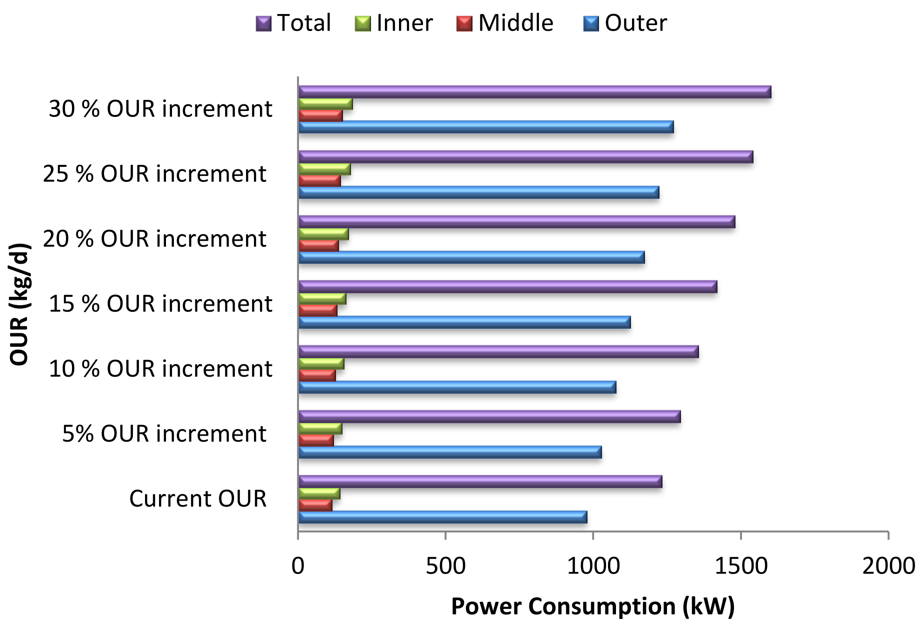


Figure 4.50: Power consumption for different increment of current OUR

The 3D computational analysis provides an opportunity to measure the average length of time that the wastewater remains in the OBS, or particularly in each channel of OBS. In general, the 3D CFD simulation is the best possible alternative towards obtaining the distribution of OUR values throughout the ditch. Using the information of the OUR values, the power consumption which is associated to these values has also been computed.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1. The relationship between oxygen utilization rate (OUR) and biological reactions in each channel of OBS. has been studied through the application of mathematical formulae as a preliminary process model. The results given by preliminary process model show that the average value of OUR for the outer channel is 7836 ±2686 kg/d, while the average value for the middle and inner channel are 1194±361 kg/d and 1027 ±331 kg/d, respectively. Based on the calculations performed using the preliminary process model, the outer channel takes the highest portion of the overall oxygen consumption, which is 78% compare to the middle channel with only 12% and followed by the inner channel with 10%. Simplifying assumptions made for the preliminary process model development (e.g. constant values of HRT throughout each channel) have affected the accuracy of describing the real conditions of the system. There is a need for more advance model in order to achieve more insight into the complex hydraulic flow patterns inside the OBS

2. A 3D multiphase and an open channel CFD-based model have been developed to simulate the hydrodynamics in an Orbal Biological System. The study shows that CFD application has the ability to provide a better

look inside the OBS including in the aspects of flow representation, flow distribution for different operating conditions and distribution of hydraulic residence time all through the system. Even though the developed 2D CFD model was able to present the overall flow pattern for the real OBS but it has some limitations in terms of providing more precise distribution of velocity and hydraulic residence time. The developed 3D two phase CFD model has shown a good representation of the real OBS. The flow pattern given by this model is indicating the right flow pattern and velocity distribution of the system. Velocity profiles given by 3D CFD model is well matched the velocity values obtained through in-situ measurements. The average deviation between flow velocities simulated by the CFD model and the actual data is only 13%. From the model it can be seen that the velocity magnitudes are decreasing when the depth of the ditch is increasing. CFD can be used to observe the pressure profiles at different depths. The pressure values are increasing when the depth of the ditch is increasing. Through the particles tracking method (discrete phase model), the distribution of residence time throughout the whole channel was obtained.

3. The developed CFD-based model has been used to assess the current operational performance (related to aerations shafts) of the OBS at the Bayan Baru Sewage Treatment Plant. The existence of the aeration devices in the OBS is not only for the aeration process but it is also important in terms of keeping the required flow momentum and avoiding the reversed flow. Without the function of the discs aerators, there is

about 47 % decrease in the flow velocity as the flow enters the middle channel from outer channel and there is approximately 63% decline when the water flows from the middle channel into inner channel. In the case without the incorporation of mixers, the flow has already deteriorated, whereas the case with the incorporation of mixers, demonstrates higher flow magnitude throughout the OBS. With the incorporation of the aeration discs within the OBS, the velocity values close to the mixers have increased and therefore the velocity distributions across the channels are also higher. Number of shafts being operated affects the total amount of power consumption in the ditch. It is found that turning off the combination of shaft 2 (outer channel) and shaft 6 (middle and inner channel) is the best option for less power consumption but still having almost the same average OUR value (26,483 kg/d) like the real case (26,594 kg/d). This means that even though these shafts have been turned off, but the biological reactions inside the ditch are still carried on as the biological reactions when all the shafts are turned on. This may be due to the similar HRT values between this two operating conditions. Based on the scenario, the findings show that with a lesser power consumption (when shaft 2 and 6 turned off), the current wastewater quality which satisfying the Standard B of effluent standards can still be accomplished.

4. Using the calculated OUR values, power consumption and its implications of effluent quality targets have been evaluated. The results have shown how much power is associated to the current OUR values that are explaining the total use of oxygen in order to fulfil the current

requirement of effluent quality standard. Based on the results, outer channel consumed 79% of the total power consumption of the OBS, while the middle and inner channel consumed 9% and 12% respectively. In future, we can easily know the ranges of power consumption in case if the OUR values have increased at certain percentage as revealed in this study. If the current OUR value which is 21109 kg/d increases for about 30%, it will cost an additional 3,241 MW of electricity in a year (almost RM 1,092,284 per year).

5.2 Recommendations

The future scope of the present work may be summarized as below:

1. Further investigations can be done to study depth-averaged velocity in vertical and horizontal directions and develop models using analytical and numerical approaches which are more convenient than conventional methods

2. It is recommended that the model be applied for parameter studies as a design tool in helping to select optimal arrangements of aeration discs throughout the OBS.

3. The model can be used for optimizing purposes, where the study on the optimum number of shafts should be operated onsite can be conducted.

4. The model can be utilized to study the effect of more stringent effluent quality standards not only in term of monetary perspective but also in term of environmental costs. These will include the effect on the carbon footprint.

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APPENDICES

Appendix A

Basin Dimension and OBS' Design Parameter

Parameter	Units	Value
No. of channels per basin		3
Channel depth	m	5.08
Inner channel width	m	6.10
Middle channel width	m	6.10
Outer channel width	m	12.50
Wall thickness	m	0.36
Radius of Center Island	m	1.52
Length of short axis straight section	m	0.00
Length of long axis straight section	m	37.34
Overall width	m	54.56
Overall length	m	91.90
Volume per basin	m^3	17 344
Flow	m3/day	50 000
MLSS	mg/L	4000
Hydraulic Residence Time	hr	18
Sludge age	day	17

Process Split

Parameter	Units	Inner Channel	Middle Channel	Outer Channel
Volume split	%	15.5	21.6	62.9
Process split	%	18.0	22.0	60.0
Selected no. of shafts per basin	-	4	4	5
No. of discs per shaft	-	33	33	68
No. of discs provided per basin	-	132	132	340

Discs

Parameter	Units	Design conditions	Maximum Output	Largest drive out of service
Speed	rpm	49	56	56
Immersion	cm	43	53	53
No. of discs in service per basin	-	604	604	536

Appendix B

Comparison of private lab data with USM lab data

USM lab data

Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)
S1	1520	2680	280
S2	890	2560	210
S3	1240	2720	220
S4	1440	2880	220
S5	1590	3690	265
S6	1130	3100	253

Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)
S1	1613	2770	316
S2	774	2600	235
S3	1380	2700	260
S4	1365	2870	259
S5	1613	3720	276
S6	1283	3120	259

T-test

BOD test (g/m3)

USM	Union lab
1520	1613
890	774
1240	1380
1440	1365
1590	1613
1130	1283

T-test p= 0.831330878 $p>0.05$

Statistically no difference between these two data sets.

MLVSS (g/m3)

USM	Union lab
2680	2770
2560	2600
2720	2700
2880	2870
3690	3720
3100	3120

T-test p= 0.918354882 $p>0.05$

Statistically no difference between these two data sets.

TKN (g/m3)

USM	Union lab
280	316
210	235
220	260
220	259
265	276
253	259

T-test p= 0.134993586 $p>0.05$

Statistically no difference between these two data sets.

Appendix C

Method used for sample analyses

Lab Test	Method
BOD (5 days @ $20^{o}C$)	APHA 5210 & 4500-O G
Total Kjeldahl Nitrogen	APHA 4500-N_{org} B
Nitrate	HACH 8039
Nitrite	HACH 8153
Mixed Liquor Volatile Suspended Solids	APHA 2540 E

Appendix D

Sample analyses

Day 1

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	1613	2770	316.3	316.3	10909
	Effluent	S2	774	2600	234.9	234.9	10909
Middle	Influent	S3	1380	2700	259.7	259.7	3746
	Effleunt	S4	1365	2870	258.5	258.5	3746
Inner	Influent	S5	1613	3720	276.2	276.2	2688
	Effluent	S6	1283	3120	258.5	258.5	2688

Day 2

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	923	2930	159.3	159.3	10909
	Effluent	S2	540	1470	151.7	151.7	10909
Middle	Influent	S3	1350	3170	206.5	206.5	3746
	Effleunt	S4	1080	3160	147.5	147.5	3746
Inner	Influent	S5	1133	3250	282.1	282.1	2688
	Effluent	S6	1005	3210	277.4	277.4	2688

Day 3

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	720	2700	257.3	257.3	10909
	Effluent	S2	675	1410	138.1	138.5	10909
Middle	Influent	S3	1500	2970	283.3	283.3	3746
	Effleunt	S4	1223	2940	259.7	259.7	3746
Inner	Influent	S5	1140	3000	270.3	270.3	2688
	Effluent	S6	768	3040	263.2	263.7	2688

Day 4

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	762	1930	184.1	184.1	10909
	Effluent	S2	498	1370	135.7	135.7	10909
Middle	Influent	S3	953	2290	212.4	212.4	3746
	Effleunt	S4	741	2420	206.5	206.9	3746
Inner	Influent	S5	753	2530	247.9	248.3	2688
	Effluent	S6	690	2030	195.9	196.4	2688

Day 5

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	828	2480	218.3	218.3	10909
	Effluent	S2	606	1670	173.5	173.5	10909
Middle	Influent	S3	795	2480	265.6	265.6	3746
	Effleunt	S4	720	2750	236.0	236.0	3746
Inner	Influent	S5	912	2170	247.9	247.9	2688
	Effluent	S6	807	2700	237 .2	237 .2	2688

Day 6 (on surface)

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	645	3090	286.8	286.8	10909
	Effluent	S2	600	1240	141.6	141.9	10909
Middle	Influent	S3	711	2910	277.4	277.4	3746
	Effleunt	S4	690	2990	269.1	269.1	3746
Inner	Influent	S5	666	3050	269.1	269.1	2688
	Effluent	S6	507	2620	256.1	256.1	2688

Day 6 (at 2m depth)

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	1086	3110	290.3	290.6	10909
	Effluent	S2	687	3000	283.3	283.3	10909
Middle	Influent	S3	780	2860	291.2	291.2	3746
	Effleunt	S4	633	2940	265.6	265.6	3746
Inner	Influent	S5	912	3030	274.1	274.1	2688
	Effluent	S6	774	2640	270.6	270.6	2688

Day 7 (on surface)

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	873	2440	228.4	228.4	10909
	Effluent	S2	744	1250	139.3	140.2	10909
Middle	Influent	S3	804	2520	245.5	245.5	3746
	Effleunt	S4	744	2680	243.2	243.2	3746
Inner	Influent	S5	876	2800	261.5	261.5	2688
	Effluent	S6	840	2800	248.9	248.9	2688

Day 7 (at 2m depth)

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	882	2780	251.2	252.4	10909
	Effluent	S2	867	2370	220.4	220.4	10909
Middle	Influent	S3	837	2730	252.4	253.6	3746
	Effleunt	S4	750	2620	220.4	220.4	3746
Inner	Influent	S5	858	2770	259.2	259.2	2688
	Effluent	S6	786	2860	259.2	259.2	2688

Day 8

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	1103	3250	276.7	276.7	10909
	Effluent	S2	995	1800	174.4	174.6	10909
Middle	Influent	S3	998	3230	300.8	300.8	3746
	Effleunt	S4	735	2970	294.8	294.8	3746
Inner	Influent	S5	1058	3360	300.8	300.8	2688
	Effluent	S6	840	3190	291.1	291.1	2688

Day 9

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	810	3500	334.5	334.5	10909
	Effluent	S2	603	1530	233.5	233.5	10909
Middle	Influent	S3	990	3360	324.8	324.8	3746
	Effleunt	S4	990	3410	312.8	312.8	3746
Inner	Influent	S5	1080	3660	330.9	330.9	2688
	Effluent	S6	818	3480	328.4	328.4	2688

Day 10

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	780	3010	288.7	288.7	10909
	Effluent	S2	632	1490	150.4	151.0	10909
Middle	Influent	S3	765	3150	300.8	300.8	3746
	Effleunt	S4	758	3040	300.8	300.8	3746
Inner	Influent	S5	735	3140	317.6	317.6	2688
	Effluent	S6	690	3000	315.2	315.2	2688

Day 11

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	1050	2930	288.7	288.7	10909
	Effluent	S2	564	1270	150.4	151.4	10909
Middle	Influent	S3	915	3050	300.8	300.8	3746
	Effleunt	S4	885	3180	283.9	283.9	3746
Inner	Influent	S5	1088	3480	300.8	300.8	2688
	Effluent	S6	840	3220	252.6	252.6	2688

Day 12

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	804	2910	288.7	288.7	10909
	Effluent	S2	507	1970	200.9	200.9	10909
Middle	Influent	S3	768	3190	300.8	301.0	3746
	Effleunt	S4	657	3090	298.4	298.6	3746
Inner	Influent	S5	738	3310	316.4	316.4	2688
	Effluent	S6	681	2930	300.8	301.1	2688

Day 13

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	606	2780	269.5	269.5	10909
	Effluent	S2	570	2490	222.6	222.8	10909
Middle	Influent	S3	717	2740	268.5	269.1	3746
	Effleunt	S4	546	2660	249.0	249.3	3746
Inner	Influent	S5	573	2820	257.5	257.5	2688
	Effluent	S6	480	2760	240.6	240.7	2688

Day 14

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	672	3080	294.8	294.8	10909
	Effluent	S2	465	1420	160.0	160.4	10909
Middle	Influent	S3	816	3220	300.8	300.8	3746
	Effleunt	S4	789	3100	300.8	300.8	3746
Inner	Influent	S5	786	3390	315.2	315.2	2688
	Effluent	S6	720	3380	300.8	300.8	2688

Day 15

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	873	3230	291.1	291.4	10909
	Effluent	S2	519	1330	161.2	161.6	10909
Middle	Influent	S3	849	3310	300.8	300.8	3746
	Effleunt	S4	810	3370	276.7	276.7	3746
Inner	Influent	S5	810	3520	332.1	332.1	2688
	Effluent	S6	762	3250	316.4	316.4	2688

Day 16

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	888	3190	292.2	292.2	10909
	Effluent	S2	387	850	102.8	103.2	10909
Middle	Influent	S3	897	3460	321.4	321.4	3746
	Effleunt	S4	867	3170	314.4	314.4	3746
Inner	Influent	S5	951	3410	292.2	292.2	2688
	Effluent	S6	861	3220	292.2	292.2	2688

Day 17

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	555	2940	261.8	261.8	10909
	Effluent	S2	393	1170	143.7	144.5	10909
Middle	Influent	S3	600	3100	280.5	280.5	3746
	Effleunt	S4	590	2930	257.1	257.1	3746
Inner	Influent	S5	567	2760	286.3	286.3	2688
	Effluent	S6	509	2920	268.8	268.8	2688

Day 18

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	798	2480	287.5	287.5	10909
	Effluent	S2	471	1410	136.7	137.3	10909
Middle	Influent	S3	599	2800	330.7	330.7	3746
	Effleunt	S4	590	2660	286.3	286.3	3746
Inner	Influent	S5	575	1780	315.5	315.5	2688
	Effluent	S6	527	1550	315.5	315.5	2688

Day 19

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	978	2910	307.3	307.3	10909
	Effluent	S2	585	1610	139.1	139.1	10909
Middle	Influent	S3	993	3280	292.2	292.2	3746
	Effleunt	S4	957	3430	292.2	292.2	3746
Inner	Influent	S5	1257	3080	292.2	292.2	2688
	Effluent	S6	1242	2870	239.6	239.6	2688

Day 20

Channel		Sample Point	BOD (g/m^3)	MLVSS (g/m^3)	TKN (g/m^3)	TN (g/m^3)	V (m^3)
Outer	Influent	S1	747	1320	288.6	288.6	10909
	Effluent	S2	596	1180	183.5	183.5	10909
Middle	Influent	S3	1020	3120	315.5	315.5	3746
	Effleunt	S4	1020	2990	315.5	315.5	3746
Inner	Influent	S5	1002	3480	314.4	314.4	2688
	Effluent	S6	593	2980	292.2	292.2	2688

Appendix E

OUR of the current process model

Day 1

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	839	2770	81.4	81.4	11462
Middle	3746	4995	15	2700	1.2	1.2	764
Inner	2688	3584	330	3720	17.7	17.7	1519

Day 2

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	383	2930	7.6	7.6	5769
Middle	3746	4995	270	3170	59	59	2145
Inner	2688	3584	128	3250	4.7	4.7	916

Day 3

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	45	2700	119.2	118.8	5436
Middle	3746	4995	277	2970	23.6	23.6	1811
Inner	2688	3584	372	3000	6.6	6.6	1405

Day 4

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	264	1930	48.4	48.4	4982
Middle	3746	4995	212	2290	5.9	5.5	1292
Inner	2688	3584	63	2530	52	51.9	932

Day 5

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	222	2480	44.8	44.8	4946
Middle	3746	4995	75	2480	29.6	29.6	1128
Inner	2688	3584	105	2170	10.7	10.7	700

Day 6

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	14545	45	3090	145.2	144.9	6377
Middle	3746	4995	21	2910	8.3	8.3	897
Inner	2688	3584	159	2835	13	13	956

Day 7

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	129	2440	89.1	88.2	14049
Middle	3746	22431	60	2520	2.3	2.3	1557
Inner	2688	21504	36	2800	12.6	12.6	1455

Day 8

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	518	3250	102.3	102.1	9556
Middle	3746	4995	263	3230	6	6	1687
Inner	2688	3584	218	3360	9.7	9.7	1161

Day 9

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	207	3500	101	101	6992
Middle	3746	4995	0	3410	12	12	997
Inner	2688	3584	262	3660	2.5	2.5	1268

Day 10

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	348	3010	138.3	137.7	8801
Middle	3746	4995	7	3150	0	0	847
Inner	2688	3584	45	3140	2.4	2.4	703

Day 11

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	486	2930	138.3	137.3	9961
Middle	3746	4995	30	3050	16.9	16.9	1035
Inner	2688	3584	248	3480	48.2	48.2	1484

Day 12

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	297	2910	87.8	87.8	6998
Middle	3746	4995	111	3190	2.4	2.4	1190
Inner	2688	3584	57	3310	15.6	15.3	845

Day 13

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	36	2780	46.9	46.7	3612
Middle	3746	4995	171	2740	19.5	19.8	1394
Inner	2688	3584	93	2820	16.9	16.8	836

Day 14

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	207	3080	134.8	134.4	7528
Middle	3746	4995	27	3220	0	0	926
Inner	2688	3584	66	3390	14.4	14.4	869

Day 15

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	354	3230	129.9	129.8	8792
Middle	3746	4995	39	3310	24.1	24.1	1191
Inner	2688	3584	48	3520	15.7	15.7	862

Day 16

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	501	3190	189.4	189	11536
Middle	3746	4995	30	3460	7	7	1057
Inner	2688	3584	90	3410	0	0	836

Day 17

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	162	2940	118.1	117.3	6630
Middle	3746	4995	10	3100	23.4	23.4	1043
Inner	2688	3584	58	2760	17.5	17.5	752

Day18

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	327	2480	150.8	150.2	8524
Middle	3746	4995	9	2800	44.4	44.4	1141
Inner	2688	3584	48	1780	0	0	439

Day 19

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	393	2910	168.2	168.2	9836
Middle	3746	4995	36	3280	0	0	968
Inner	2688	3584	15	3080	52.6	52.6	935

Day 20

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	14545	151	1320	105.1	105.1	4940
Middle	3746	4995	0	3120	0	0	819
Inner	2688	3584	409	3480	22.2	22.2	1671

Appendix F

Sensitivity Analysis

Comparison of different Y value for the outer channel

	Y=0.5	Y=0.6	Y=0.7
	34972	29591	26004
	15437	12912	11228
	11889	11611	11426
	13235	11576	10470
	13089	11556	10533
	14712	14417	14221
	13348	12465	11877
	25666	22472	20343
	20263	18597	17487
	26075	23589	21931
	31399	27878	25531
	16646	14923	13774
	7797	7513	7324
	20706	19233	18251
	25986	23426	21720
	35504	31958	29594
	17924	16754	15975
	26016	23672	22110
	29324	26581	24752
	13334	12381	11746
Mean	20666	18655	17315
STDEV	8193	7079	6373
Ttest	41.1%		53.3%

Comparison of different Y value for the middle channel

	Y=0.5	Y=0.6	Y=0.7
	1277	1181	1117
	14169	12389	11202
	10992	9282	8142
	7732	6400	5512
	5572	5054	4708
	2071	1933	1842
	2894	2483	2210
	9377	7756	6675
	1995	1995	1995
	1077	1027	993
	3282	3065	2920
	4216	3572	3142
	9182	7831	6931
	1805	1613	1485
	4265	3983	3795
	2534	2322	2180
	3100	3028	2980
	4685	4620	4577
	2117	1866	1698
	819	819	819
Mean	4658	4111	3746
STDEV	3722	3134	2752
Ttest	61.8%		69.8%

Comparison of different Y value for the inner channel

	Y=0.5	Y=0.6	Y=0.7
	12577	10460	9049
	5186	4342	3779
	12514	10217	8686
	6192	5796	5533
	4878	4152	3669
	6711	5670	4976
	2743	2496	2332
	8035	6691	5795
	11457	9350	7944
	2394	2073	1858
	13618	11821	10624
	3342	3011	2791
	5740	5005	4516
	4155	3685	3372
	3692	3345	3114
	3827	3190	2766
	4054	3635	3356
	2056	1712	1482
	5289	5185	5115
	15158	12577	10856
Mean	6681	5721	5081
STDEV	4096	3355	2875
Ttest	42.2%		52.1%

Comparison of different K_d value for the outer channel

	Kd=0.07	Kd=0.08	Kd=0.09
	29289	29591	29893
	12592	12912	13231
	11317	11611	11906
	11365	11576	11786
	11285	11556	11826
	14080	14417	14754
	12199	12465	12732
	22118	22472	22827
	18216	18597	18979
	23260	23589	23917
	27559	27878	28198
	14605	14923	15240
	7210	7513	7816
	18897	19233	19569
	23074	23426	23778
	31610	31958	32306
	16434	16754	17075
	23402	23672	23943
	26263	26581	26898
	12237	12381	12525
Mean	18351	18655	18960
STDEV	7056	7079	7102
Ttest	89.2%		89.3%

Comparison of different K_d value for the middle channel

	Kd=0.07	Kd=0.08	Kd=0.09
	1080	1181	1282
	12270	12389	12508
	9171	9282	9393
	6314	6400	6485
	4961	5054	5147
	1824	1933	2042
	2389	2483	2578
	7635	7756	7877
	1867	1995	2123
	909	1027	1145
	2951	3065	3179
	3452	3572	3691
	7729	7831	7934
	1493	1613	1734
	3859	3983	4107
	2192	2322	2452
	2912	3028	3144
	4516	4620	4725
	1743	1866	1989
	702	819	936
Mean	3998	4111	4224
STDEV	3136	3134	3132
Ttest	91.0%		91.0%

Comparison of different K_d value for the inner channel

	Kd=0.07	Kd=0.08	Kd=0.09
	10360	10460	10560
	4254	4342	4429
	10137	10217	10298
	5728	5796	5864
	4094	4152	4211
	5594	5670	5746
	2421	2496	2571
	6601	6691	6782
	9251	9350	9448
	1988	2073	2157
	11728	11821	11915
	2922	3011	3100
	4930	5005	5081
	3594	3685	3776
	3251	3345	3440
	3099	3190	3282
	3561	3635	3709
	1664	1712	1759
	5102	5185	5267
	12483	12577	12670
Mean	5638	5721	5803
STDEV	3349	3355	3361
Ttest	93.8%		93.8%

Comparison of different f_b value for the outer channel

	fb=0.7	fb=0.8	fb=0.9
	29289	29591	30196
	12592	12912	13551
	11317	11611	12200
	11365	11576	11997
	11285	11556	12097
	14080	14417	15092
	12199	12465	12998
	22118	22472	23181
	18216	18597	19361
	23260	23589	24245
	27559	27878	28517
	14605	14923	15558
	7210	7513	8120
	18897	19233	19905
	23074	23426	24131
	31610	31958	32654
	16434	16754	17396
	23402	23672	24213
	26263	26581	27216
	12237	12381	12669
Mean	18351	18655	19265
STDEV	7056	7079	7126
Ttest	89.2%		78.8%

Comparison of different fb value for the middle channel

	fb=0.7	fb=0.8	fb=0.9
	1080	1181	1383
	12270	12389	12626
	9171	9282	9504
	6314	6400	6571
	4961	5054	5240
	1824	1933	2151
	2389	2483	2672
	7635	7756	7998
	1867	1995	2251
	909	1027	1263
	2951	3065	3294
	3452	3572	3811
	7729	7831	8036
	1493	1613	1855
	3859	3983	4231
	2192	2322	2581
	2912	3028	3260
	4516	4620	4830
	1743	1866	2111
	702	819	1052
Mean	3998	4111	4336
STDEV	3136	3134	3130
Ttest	91.0%		82.1%

Comparison of different fb value for the inner channel

	fb=0.7	fb=0.8	fb=0.9
	10360	10460	10660
	4254	4342	4516
	10137	10217	10379
	5728	5796	5932
	4094	4152	4269
	5594	5670	5822
	2421	2496	2647
	6601	6691	6872
	9251	9350	9546
	1988	2073	2242
	11728	11821	12008
	2922	3011	3189
	4930	5005	5157
	3594	3685	3867
	3251	3345	3534
	3099	3190	3374
	3561	3635	3784
	1664	1712	1807
	5102	5185	5350
	12483	12577	12764
Mean	5638	5721	5886
STDEV	3349	3355	3355
Ttest	93.8%		87.7%

Summary of sensitivity analysis of Y, $K_{d\ and}$ f_b values

OUR value when Y=0.5

Channel	Outer	Middle	Inner
Minimum	7797	819	2056
Average	20666	4658	6681
Maximum	35504	14169	15158

OUR value when Y=0.7

Channel	Outer	Middle	Inner
Minimum	7324	819	1482
Average	17315	3747	5081
Maximum	29594	11202	10856

OUR value when K_d =0.07

Channel	Outer	Middle	Inner
Minimum	7210	702	1664
Average	18351	3999	5639
Maximum	31610	12270	12483

OUR value when K_d =0.09

Channel	Outer	Middle	Inner
Minimum	7816	936	1759
Average	18960	4224	5804
Maximum	32306	12508	12670

OUR value when $f_{b'}$ =0.7

Channel	Outer	Middle	Inner
Minimum	7210	702	1664
Average	18351	3999	5639
Maximum	31610	12270	12483

OUR value when $f_{b'}$ =0.9

Channel	Outer	Middle	Inner
Minimum	8120	1052	1807
Average	17644	4904	6222
Maximum	32654	12626	12764

Appendix G

OUR of CFD model

Day 1

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	839	2770	81.4	81.4	35818
Middle	3746	22431	15	2700	1.2	1.2	956
Inner	2688	21504	330	3720	17.7	17.7	5609

Day 2

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	383	2930	7.6	7.6	14972
Middle	3746	22431	270	3170	59	59	6729
Inner	2688	21504	128	3250	4.7	4.7	2436

Day 3

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	45	2700	119.2	118.8	14229
Middle	3746	22431	277	2970	23.6	23.6	5413
Inner	2688	21504	372	3000	6.6	6.6	5607

Day 4

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	264	1930	48.4	48.4	14123
Middle	3746	22431	212	2290	5.9	5.5	3706
Inner	2688	21504	63	2530	52	51.9	3208

Day 5

Channel	V (m^3)	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	222	2480	44.8	44.8	12898
Middle	3746	22431	75	2480	29.6	29.6	2796
Inner	2688	21504	105	2170	10.7	10.7	2157

Day 6

Channel	V (m^3	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	45	3090	145.2	144.9	16843
Middle	3746	22431	21	2910	8.3	8.3	1365
Inner	2688	21504	159	2835	13	13	3063

Day 7

Channel	V (m^3	Q (m^3/d)	(So-Se) (g/m^3)	MLVSS (g/m^3)	ΔTKN (g/m^3)	ΔTN (g/m^3)	OUR (kg/d)
Outer	10909	52447	129	2440	89.1	88.2	14049
Middle	3746	22431	60	2520	2.3	2.3	1557
Inner	2688	21504	36	2800	12.6	12.6	1455

Day 8

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	518	3250	102.3	102.1	27988
Middle	3746	22431	263	3230	6	6	4617
Inner	2688	21504	218	3360	9.7	9.7	3802

Day 9

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	207	3500	101	101	18245
Middle	3746	22431	0	3410	12	12	1355
Inner	2688	21504	262	3660	2.5	2.5	4162

Day 10

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	348	3010	138.3	137.7	25743
Middle	3746	22431	7	3150	0	0	921
Inner	2688	21504	45	3140	2.4	2.4	1260

Day 11

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	486	2930	138.3	137.3	30085
Middle	3746	22431	30	3050	16.9	16.9	1852
Inner	2688	21504	248	3480	48.2	48.2	5627

Day 12

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	297	2910	87.8	87.8	19443
Middle	3746	22431	111	3190	2.4	2.4	2423
Inner	2688	21504	57	3310	15.6	15.3	1951

Day 13

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	36	2780	46.9	46.7	7492
Middle	3746	22431	171	2740	19.5	19.8	3749
Inner	2688	21504	93	2820	16.9	16.8	2359

Day 14

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	207	3080	134.8	134.4	21016
Middle	3746	22431	27	3220	0	0	1208
Inner	2688	21504	66	3390	14.4	14.4	2019

Day 15

Channel	V (m^3	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	354	3230	129.9	129.8	25272
Middle	3746	22431	39	3310	24.1	24.1	2318
Inner	2688	21504	48	3520	15.7	15.7	1859

Day 16

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	501	3190	189.4	189	35248
Middle	3746	22431	30	3460	7	7	1580
Inner	2688	21504	90	3410	0	0	1803

Day 17

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	162	2940	118.1	117.3	18055
Middle	3746	22431	10	3100	23.4	23.4	1846
Inner	2688	21504	58	2760	17.5	17.5	1912

Day 18

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	327	2480	150.8	150.2	25799
Middle	3746	22431	9	2800	44.4	44.4	2559
Inner	2688	21504	48	1780	0	0	955

Day 19

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	393	2910	168.2	168.2	29675
Middle	3746	22431	36	3280	0	0	1345
Inner	2688	21504	15	3080	52.6	52.6	2708

Day 20

Channel	V (m^3)	Q (m3/d)	(So-Se) (g/m3)	MLVSS (g/m3)	ΔTKN (g/m3)	ΔTN (g/m3)	OUR (kg/d)
Outer	10909	52447	151	1320	105.1	105.1	15186
Middle	3746	22431	0	3120	0	0	819
Inner	2688	21504	409	3480	22.2	22.2	6749

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