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<th>Comparison of intermittently aerated sequencing batch reactors (IASBRs) and conventional sequencing batch reactors (cSBRs) in wastewater treatment</th>
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<td>Henry, Liam Garry</td>
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COMPARISON OF INTERMITTENTLY AERATED SEQUENCING BATCH REACTORS (IASBRs) and CONVENTIONAL SEQUENCING BATCH REACTORS (cSBRs) IN WASTEWATER TREATMENT

by

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Professor of Civil Engineering: Padraic E. O’Donoghue

A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy

January 2014
To my parents Liam and Marie, and my grandparents Bill, Kay, Garry and Ena
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Finally, I am grateful to the Environmental Protection Agency for their financial support (Ref.: 2009-PhD-ET-7).
Abstract

Increasingly stringent discharge standards, increased water usage, variations in wastewater characteristics and climate change have resulted in increased difficulty in achieving cost effective wastewater treatment. The overall aim of this study was to compare the performance of an intermittently aerated sequencing batch reactor (IASBR) technology and a conventional sequencing batch reactor (cSBR) technology under various scenarios in terms of operation effectiveness and flexibility.

During this research, laboratory-scale IASBR and cSBR systems were constructed in the Environmental Engineering Laboratories at the National University of Ireland, Galway to treat synthetic domestic wastewater and real municipal wastewater. In addition mathematical modelling was used to further investigate and compare the IASBR and cSBR operations. The specific objectives of this research included: (1) to compare the treatment effectiveness of IASBR and cSBR operations; (2) to develop predictive GPS-X simulation models capable of predicting the efficiency of cSBR and IASBR systems in the treatment of synthetic domestic and real municipal wastewaters; and (3) to develop a predictive GPS-X simulation model to assess the IASBR technology in slaughterhouse wastewater treatment, in comparison to the cSBR technology.

Synthetic domestic wastewater was treated to observe and compare the effectiveness of the IASBR and cSBR operations at two aeration rates, 0.8 L air/min and 1 L air/min, for a low strength wastewater. The laboratory results show that at the aeration rate of 1 L air/min nitrification was improved but denitrification and phosphorus removal declined due to the increased dissolved oxygen concentrations present in the react period. The cSBR data was then utilised to validate a predictive GPS-X simulation model. The mathematical modelling shows that the IASBR technology can offer safer levels of treatment than the cSBR technology. The heterotrophic biomass concentration was higher in the cSBR but phosphorus accumulating organisms (PAOs) and autotrophic biomass concentrations were higher in the IASBR. Converting the operation sequence from a cSBR to an IASBR promoted the denitrification and phosphorus removal processes.
The performance of the IASBR operation was examined for the treatment of real municipal wastewater in order to confirm the previous observations of the IASBR operation for the treatment of wastewater which had received primary clarification treatment, referred to as primary clarifier effluent (PCE), or by-passed primary clarification treatment, referred to as primary clarifier influent (PCI). The IASBR technology achieved better performance, in particular in nitrogen removal, when treating PCI wastewater due to the higher influent chemical oxygen demand/nitrogen (COD/N) ratios. Mathematical modelling indicated that during the treatment of the PCE wastewater heterotrophic and autotrophic biomass concentrations were reduced, in comparison to the PCI wastewater treatment. The shock load impact due to the conversion of the influent from PCE to PCI wastewater, and vice versa, was also investigated. It was observed that the effluent quality was improved when converting influent wastewater from PCE to PCI.

A mathematical model was developed to compare the performance of the cSBR and IASBR operations for the treatment of high strength slaughterhouse wastewater under various scenarios. The IASBR operation had reduced heterotrophic and autotrophic biomass concentrations due to its extended non-aeration period; however significantly improved denitrification was achieved. The impact of reducing hydraulic retention time (HRT) on treatment efficiency was studied. The reduced HRT negatively affected treatment efficiencies and resulted in a decline in effluent quality; however the treatment performance can be improved by amending the IASBR operation sequence. This study shows that increased denitrification can be achieved in an IASBR in comparison to a cSBR and altering the cSBR operation sequence to enhance denitrification can result in a decline in nitrification processes. The operation flexibility allowed the IASBR and cSBR to adapt to variations in the HRT and allowed for increased volumes of wastewater to be treated. WWTPs utilising IASBRs can reduce treatment complexity, space requirements, capital and operation costs, and increase operational flexibility.

**Keywords:** Sequencing Batch Reactor; Intermittent Aeration; Municipal Wastewater; Slaughterhouse Wastewater; Primary Clarification; Mathematical Modelling; Activated Sludge Models; GPS-X
## Terminology Explanation

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<thead>
<tr>
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<th>Meaning</th>
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<tr>
<td>A</td>
<td>Aeration</td>
</tr>
<tr>
<td>ALR</td>
<td>Ammonia Loading Rate</td>
</tr>
<tr>
<td>AOB</td>
<td>Ammonia Oxidising Bacteria</td>
</tr>
<tr>
<td>AS</td>
<td>Activated Sludge</td>
</tr>
<tr>
<td>ASBBRs</td>
<td>Anaerobic Sequencing Batch Biofilm Reactor</td>
</tr>
<tr>
<td>ASBR</td>
<td>Anaerobic Sequencing Batch Reactor</td>
</tr>
<tr>
<td>ASM</td>
<td>Activated Sludge Model</td>
</tr>
<tr>
<td>BNR</td>
<td>Biological Nitrogen Removal</td>
</tr>
<tr>
<td>BOD</td>
<td>Biochemical Oxygen Demand</td>
</tr>
<tr>
<td>BPR</td>
<td>Biological Phosphorus Removal</td>
</tr>
<tr>
<td>C</td>
<td>Carbon</td>
</tr>
<tr>
<td>CFBBR</td>
<td>Circulating Fluidised Bed Bioreactors</td>
</tr>
<tr>
<td>COD</td>
<td>Chemical Oxygen Demand</td>
</tr>
<tr>
<td>COD&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Total Chemical Oxygen Demand</td>
</tr>
<tr>
<td>cSBR</td>
<td>Conventional Sequencing Batch Reactor</td>
</tr>
<tr>
<td>DIN</td>
<td>Dissolved Inorganic Nitrogen</td>
</tr>
<tr>
<td>DO</td>
<td>Dissolved Oxygen</td>
</tr>
<tr>
<td>EBCT</td>
<td>Empty Bed Contact Times</td>
</tr>
<tr>
<td>EBPR</td>
<td>Enhanced Biological Phosphorus Removal</td>
</tr>
<tr>
<td>EPA</td>
<td>Environmental Protection Agency</td>
</tr>
<tr>
<td>EU</td>
<td>European Union</td>
</tr>
<tr>
<td>HAR</td>
<td>High Aeration Rate</td>
</tr>
<tr>
<td>HRT</td>
<td>Hydraulic Retention Time</td>
</tr>
<tr>
<td>IA</td>
<td>Intermittent Aeration</td>
</tr>
<tr>
<td>IASBR</td>
<td>Intermittently Aerated Sequencing Batch Reactor</td>
</tr>
<tr>
<td>ICS</td>
<td>Intelligent Control System</td>
</tr>
<tr>
<td>LAR</td>
<td>Low Aeration Rate</td>
</tr>
<tr>
<td>LCA</td>
<td>Life Cycle Assessment</td>
</tr>
<tr>
<td>MLSS</td>
<td>Mixed Liquor Suspended Solids</td>
</tr>
<tr>
<td>N</td>
<td>Nitrogen</td>
</tr>
<tr>
<td>NA</td>
<td>Non-Aeration</td>
</tr>
<tr>
<td>NH&lt;sub&gt;4&lt;/sub&gt;+-N</td>
<td>Ammonium-Nitrogen</td>
</tr>
<tr>
<td>NO&lt;sub&gt;2&lt;/sub&gt;-N</td>
<td>Nitrite-Nitrogen</td>
</tr>
<tr>
<td>NO&lt;sub&gt;3&lt;/sub&gt;-N</td>
<td>Nitrate-Nitrogen</td>
</tr>
<tr>
<td>NOB</td>
<td>Nitrite Oxidising Bacteria</td>
</tr>
<tr>
<td>NUIG</td>
<td>National University of Ireland, Galway</td>
</tr>
<tr>
<td>OLR</td>
<td>Organic Loading Rate</td>
</tr>
<tr>
<td>ORP</td>
<td>Oxidation Reduction Potential</td>
</tr>
<tr>
<td>P</td>
<td>Phosphorus</td>
</tr>
<tr>
<td>PAOs</td>
<td>Phosphorus Accumulating Organisms</td>
</tr>
<tr>
<td>Term</td>
<td>Meaning</td>
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<tr>
<td>--------</td>
<td>----------------------------------------------</td>
</tr>
<tr>
<td>PCE</td>
<td>Primary Clarifier Effluent</td>
</tr>
<tr>
<td>PCI</td>
<td>Primary Clarifier Influent</td>
</tr>
<tr>
<td>PLC</td>
<td>Programmable Logic Controller</td>
</tr>
<tr>
<td>PO$_4$$^3-$P</td>
<td>Ortho-Phosphate</td>
</tr>
<tr>
<td>SBBR</td>
<td>Sequencing Batch Biofilm Reactor</td>
</tr>
<tr>
<td>SBR</td>
<td>Sequencing Batch Reactor</td>
</tr>
<tr>
<td>SND</td>
<td>Simultaneous Nitrification and Denitrification</td>
</tr>
<tr>
<td>SOTE</td>
<td>Standard Oxygen Transfer Efficiency</td>
</tr>
<tr>
<td>SQND</td>
<td>Sequential Nitrification and Denitrification</td>
</tr>
<tr>
<td>SRT</td>
<td>Sludge Retention Time</td>
</tr>
<tr>
<td>SS</td>
<td>Suspended Solids</td>
</tr>
<tr>
<td>TCS</td>
<td>Time Control System</td>
</tr>
<tr>
<td>TN</td>
<td>Total Nitrogen</td>
</tr>
<tr>
<td>TOC</td>
<td>Total Organic Carbon</td>
</tr>
<tr>
<td>TON</td>
<td>Total Oxidised Nitrogen</td>
</tr>
<tr>
<td>TP</td>
<td>Total Phosphorus</td>
</tr>
<tr>
<td>UV</td>
<td>Ultra Violet</td>
</tr>
<tr>
<td>UWWTD</td>
<td>Urban Waste Water Treatment Directive</td>
</tr>
<tr>
<td>WFD</td>
<td>Water Framework Directive</td>
</tr>
<tr>
<td>WHO</td>
<td>World Health Organization</td>
</tr>
<tr>
<td>WWTP</td>
<td>Waste Water Treatment Plant</td>
</tr>
<tr>
<td>α</td>
<td>Alpha Factor</td>
</tr>
</tbody>
</table>
Chapter One

Introduction

1.1 Background

Water quality is an ever increasing problem within Ireland. Discharging of nutrients into clean water results in the growth of algae and leads to eutrophication in lakes and streams. The European Union (EU) has set guidelines regarding nutrient removal from domestic, agricultural and industrial wastewaters through a number of directives such as the Nitrates Directive (91/676/EEC), the Water Framework Directive (2000/60/EC) and the Urban Wastewater Treatment Directive (91/271/EEC). The Water Framework Directive (WFD) requires all water bodies in the EU to achieve a ‘good status’ by 2015. ‘Good status’ indicates a slight deviation from the reference qualities and conditions set by the WFD.

The Environmental Protection Agency (EPA) report on water quality in Ireland from 2007 to 2009 was released in 2011 (Irish EPA, 2011). The report gathered data from multiple locations throughout Ireland, including:

- Groundwater sites: 211 locations were tested. The gathered data showed that 15.3% of the country’s groundwater was classified with a poor status, dominantly due to excessive phosphate concentrations. Nitrate concentrations were stable; however two monitoring stations showed a statistically significant upward trend. An increased incidence of positive counts of faecal coliform bacteria was also detected.

- Rivers and streams: 2,500 monitoring sites were tested, representing 13,000 km of channel length. Although an improvement in river quality was observed just 52% of rivers were classified with a good status, with 48% classified as moderate to bad. 27 sites were classified as seriously polluted. These sites were negatively affected by municipal wastewater treatment plants, agricultural discharges, engineering works, landfills, historical mining, forestry activity and construction works.

- Lakes: 222 lakes were monitored, and 39 of these (19.1%) were moderately to strongly eutrophic. This represents a decline in satisfactory quality of more
than 4%. When assessed ecologically according to the requirements of the WFD, 52.7% of lakes were classified with a moderate to bad status.

- Estuarine and coastal water: 89 sites were monitored, with 31 sites breaching the winter dissolved inorganic nitrogen (DIN) assessment criterion. 50.5% of the bodies were intermediate to poor.

Wastewater discharges were also monitored in urban areas by the EPA, with results published in the 2012 report ‘Focus on Urban Waste Water Discharges in Ireland’ (Irish EPA, 2012). The report focused on samples gathered in 2009 and compared the data to previous years and to the requirements specified by the Urban Waste Water Treatment Regulations 2001-2010 and the 1991 Urban Waste Water Treatment Directive (UWWTD). The operation of wastewater treatment plants was reviewed in 529 locations. Its major findings were:

- 71 urban areas (≈ 500 PE) may or may not have appropriate treatment depending on specific site locations.
- 26 urban areas (≥ 500 PE) identified as having no treatment or just screening in place.
- 11 large urban areas (≥ 2,000 PE) did not meet the UWWTD requirement to have secondary treatment in place.
- 8 urban areas (≥ 10,000 PE) did not meet the UWWTD requirement to provide nutrient reduction in addition to secondary treatment for discharges to sensitive areas.
- 42 coastal towns with water based tourist activities were without secondary treatment.

The data indicated that 57% of larger urban areas met all of the effluent quality standards and monitoring requirements. When all areas with secondary treatment were included the number of plants meeting the standards was reduced to 54%. Nationally, only 42% of plants reached the standards of the Urban Waste Water Treatment Regulations 2001-2010 and the UWWTD. These reports indicate that more research and investment to improve water treatment is required if Ireland is to achieved a good status by 2015, as set by the WFD.
Recently sequencing batch reactor (SBR) technology has become popular for the treatment of various types of wastewater. SBRs consist of a number of advantages in comparison to continuous flow processes (Keller, 2005; Stricker and Beland, 2006; Mace and Mata-Alvarez, 2002), which include:

- Operational conditions are easy to control and reliable;
- Cyclic operations make data gathering easy and results are representative;
- SBRs have a lower operational cost than conventional biological treatment methods;
- SBRs can adapt to large variations in hydraulic and organic loadings;
- Filamentous growth and settling problems can be easily identified, controlled and corrected due to the settlement period in the reactor; and,
- Space requirements are reduced by ‘all-in-one’ treatment unit.

The conventional SBR (cSBR) process has been modified to accommodate increasingly stringent discharge standards. The modifications include anaerobic SBRs (ASBRs), step feed SBRs and intermittently aerated SBRs (IASBRs). The IASBR operation is considered to have increased flexibility and efficiency in comparison to the cSBR operation. It is believed that the intermittent aeration process enhances nitrogen (N) and phosphorus (P) removal when treating high strength wastewater (Zhan et al., 2009). The Environmental Engineering Research Group at Civil Engineering, NUI Galway has developed IASBR technology to treat slaughterhouse wastewater and pig manure digestate.

Mathematical modelling is a representation of real world objects, and can be used for a number of purposes, such as description or prediction of the objects (de Vries, 2001). Mathematical modelling has been widely utilised in wastewater treatment as it can be used to fully exploit the available wastewater treatment plant (WWTP) facilities in a cost effective way (Dürrenmatt and Gujer, 2012). WWTP models have been utilised to simulate various forms of treatment processes and identify the associated costs to determine their feasibility (Hernandez-Sancho et al., 2011).

**1.2 Study Objectives**

The overall aim of this PhD research was to compare IASBR and cSBR operations...
through laboratory studies and mathematical modelling. The specific objectives of this study were:

- To compare the treatment effectiveness of IASBR and cSBR operation sequences for the treatment of wastewater.
- To develop a predictive GPS-X simulation model that can be used for further comparison of the cSBR and IASBR operation sequences.
- Synthetic domestic wastewater was used to compare the IASBR and cSBR operations for the treatment of low strength wastewater with consistent influent concentrations.
- In order to test and confirm the efficiency of the IASBR operation for real municipal wastewater treatment, the effectiveness of an IASBR for the treatment of two real municipal wastewaters was investigated; one was primary clarification effluent and the other was without primary clarification treatment.
- A predictive GPS-X simulation model was developed to assess and compare the IASBR and cSBR operations in high strength slaughterhouse wastewater treatment.

### 1.3 Research Procedure

In order to address the specific objectives of this research, the research procedures are briefed below:

Two-bench scale SBR units, (i) a cSBR and (ii) an IASBR, were constructed in the laboratory to investigate and compare the operation sequences. The laboratory units were constructed from stainless steel reactors with an internal diameter of 180 mm. The two units were operated at ambient temperature and each had a working volume of 5 litres. The unit operations were controlled by a Siemens Programmable Logic Controller (PLC).

For the comparison of the different SBR operation sequences, the laboratory units were operated as a cSBR and an IASBR. Synthetic domestic wastewater was treated at two aeration rates, 0.8 L air/min and 1 L air/min. The performance of the two units was studied and compared by measuring contaminant removal rates. The removal
kinetics were studied by performing cycle phase studies. A mathematical model was constructed using the laboratory data and GPS-X software for further study of the SBR operations.

For examination of the IASBR technology, a laboratory unit was utilised for the treatment of real municipal wastewater. Municipal wastewater which had received or by-passed primary clarification treatment was treated by the IASBR. The performance of the IASBR for the treatment of both municipal wastewaters was studied and compared. A mathematical model was constructed using the laboratory data and GPS-X software for further study of the IASBRs.

GPS-X is a mathematical modelling software which can be utilised for static and dynamic simulations of numerous process units using its model library (Xu et al., 2006). For the examination of the IASBR technology a WWTP layout consisting of an influent line, an equalisation tank, an SBR and an effluent line was constructed to simulate the laboratory units. The laboratory data for the treatment of synthetic and municipal wastewaters were used to calibrate the models. The performance of the cSBR and IASBR operation sequences under various scenarios was studied and compared in terms of the contaminant removal rates and reaction kinetics.

A mathematical model for studying slaughterhouse wastewater treatment using a cSBR and an IASBR was developed using data gathered from the master’s thesis of Daniel Norton (Norton, 2009). This allowed for the comparison of various operation sequences under different scenarios.

1.4 Structure of the Dissertation
The PhD thesis structure is given below:

Chapter 2 reviews the wastewater discharge standards, details and compares the continuous flow and batch reactor treatment processes, and describes mathematical modelling of wastewater treatment.

Chapter 3 details the design and operation of the laboratory cSBR and IASBR units
and the testing procedures utilised.

Chapter 4 presents and compares the performance of the cSBR and IASBR units for the treatment of synthetic domestic wastewater using laboratory data and mathematical modelling.

In Chapter 5 the performance of an IASBR for the treatment of two municipal feedstocks is presented using laboratory data and mathematical modelling.

Chapter 6 presents a mathematical model developed to simulate slaughterhouse wastewater treatment using cSBR and IASBR operation sequences.

Finally, the conclusions obtained in the study with recommendations for further research are presented in Chapter 7.
Chapter Two

Literary Review

2.1 Introduction
This chapter firstly reviews wastewater treatment standards required by the EU and other countries and describes the wastewater treatment processes.

The continuous flow process is described and its advantages and disadvantages are discussed. The SBR technology is described in detail. SBR operations are explained and the variations are presented, with focus made on IASBRs and the factors which can affect its operation.

Modelling of WWTPs is becoming increasingly important as they can be applied in various roles, such as learning, design and control. Activated sludge models (ASM) 1, 2, 2d and 3 are reviewed and the process kinetics are discussed. The GPS-X simulation software is described, in addition with its applications.

2.2 Wastewater Treatment Standards
The purpose of wastewater treatment is to mitigate the adverse impacts of wastewater contaminants on the environment and public health. Initially wastewater treatment focused on the removal of suspended solids (SS), the treatment of biodegradable organic contaminants (in terms of chemical oxygen demand, COD; and biochemical oxygen demand, BOD) and the elimination of pathogenic organisms. In time the importance of the nutrient (N and P) removal was recognised due to their eutrophication effects.

Discharge standards have been introduced to ensure safe and reliable wastewater treatment practices. Studies on wastewater treatment primarily aim to develop or improve wastewater treatment processes to achieve specified discharge standards (Shen et al., 2009; Shengquan et al., 2008). Treatment standard requirements vary depending on a number of factors. These include:
• Receiving waters: These are the water bodies into which treated wastewater will be discharged. Their characteristics can determine the standard of effluent treatment required and therefore the treatment processes which should be utilised. In general the larger the flow volumes of the receiving water, the greater its capacity to assimilate treated effluent without adverse effects, thus easing treatment requirements. How the wastewater is to be used also determines treatment standards. For example drinking water sources will require the strictest standards.

• Wastewater classification: Depending on the strength, use and characteristics of wastewater various classifications can apply, for example municipal wastewater, slaughterhouse wastewater and landfill leachate. The type of wastewater treatment processes selected depends on these classifications.

The World Health Organisation (WHO) has recommended standards for wastewater treatment; however these are not mandatory. In Ireland standards are set by the EPA according to EU directives. In the USA individual states set their standards, with California enforcing some of the strictest standards (Blumenthal et al., 2000). Table 2.1 lists some of the discharge standards set globally for wastewater treatment.

Table 2.1. Wastewater treatment standards (units: mg/L, unless specified)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Irish EPA</th>
<th>USA EPA</th>
<th>Chinese EPA</th>
<th>Saudi Arabia</th>
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<td>Class 3</td>
<td>Class 2</td>
<td>Class 1B</td>
<td>Class 1A</td>
</tr>
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<td>BOD₅</td>
<td>25</td>
<td>5</td>
<td>60</td>
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<td>75% removal</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>TP</td>
<td>1-2</td>
<td>-</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>COD</td>
<td>-</td>
<td>-</td>
<td>120</td>
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</tr>
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</table>

a: Value in brackets is standard at temperatures ≤ 12°C.
b: Monthly average value must be ≤ 10 mg/L and weekly average value must be ≤ 15 mg/L.
c: Monthly average value must be ≤ 40 mg/L.
Variations in wastewater treatment standards may require changes in the processes utilized in WWTPs. Increasingly stringent standards can impact the operation of existing WWTPs. Numerous studies have reviewed these impacts.

Al-Jasser (2011) reviewed the conformity of WWTPs to the Saudi Arabia irrigation treatment standards. The 2003 standards had been updated in 2006, resulting in local plants having difficulty complying with the more stringent standards. The details of each set of standards for unrestricted and restricted irrigation are shown in Table 2.2. The study suggests that these standards should be adapted to the various locations in order to achieve efficient treatment.

Table 2.2. Saudi Arabia 2003-MMRA and 2006-MWE maximum allowable contaminant levels in restricted and unrestricted irrigation waters (Al-Jasser, 2011)

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<tr>
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<td>40(^{b})</td>
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<tr>
<td>BOD(_{5})</td>
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<td>10(^{a})</td>
<td>40</td>
<td>40(^{b})</td>
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<tr>
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<tr>
<td>TOC</td>
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<td>Oil &amp; grease</td>
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<td>Inorganic chemical parameters</td>
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<tr>
<td>Manganese (Mn)</td>
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## Chapter 2 Literature Review

### Unrestricted irrigation
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<td>0.5⁵</td>
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<tr>
<td>Fluoride (F)</td>
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### Restricted irrigation

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<td>2500³</td>
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<td>Nitrate (NO₃-N)</td>
<td>mg/L</td>
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<td>Free residual chlorine</td>
<td>mg/L</td>
<td>0.2</td>
<td>0.5⁵</td>
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<td>Fluoride (F)</td>
<td>mg/L</td>
<td>1</td>
<td>1</td>
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</tbody>
</table>

### Chemical compounds

- **TDS** (Total Dissolved Solids) - mg/L
- **Chloride (Cl⁻)** - mg/L
- **Sulfate (SO₄²⁻)** - mg/L
- **Ammonia (NH₃-N)** - mg/L
- **Nitrate (NO₃-N)** - mg/L
- **Free residual chlorine** - mg/L
- **Fluoride (F)** - mg/L

### Biological parameters

- **Fecal coliforms per 100 ml**
- **Intestinal nematodes per litre**
- **Intestinal nematodes per litre**

### Notes

- **a:** Monthly average BOD₅ and TSS ≤ 10 mg/L each. Weekly average BOD₅ and TSS ≤ 15 mg/L each.
- **b:** Monthly average BOD₅ and TSS ≤ 40 mg/L each, and Faecal coliforms 1000 calories/100 mL.
- **c:** Tertiary treated effluents with TDS > stated concentration can be utilised if dilution with a water of less TDS is possible, or if it will be used for irrigating crops insensitive to high TDS.
- **d:** Secondary treated effluents with TDS > stated concentrations can be utilised if dilution with water of less TDS is possible, or if it will be used for irrigating crops insensitive to high TDS.
- **e:** Free residual chlorine should be ≥ 0.2 mg/L if chlorine is used as a disinfectant.
- **f:** Faecal coliform organisms in the effluent should not be > 2.2/100 mL (MPN methods or equivalent).
- **g:** If average faecal coliform organisms in effluent are ≤ MPN 2.2/100 mL (or equivalent) for the last 7 days and the faecal coliform organisms do not exceed MPN 23/100 mL (or equivalent) in any sample the wastewater is considered adequately disinfected for unrestricted irrigation.
- **h:** Colonies.
- **i:** Only live intestinal nematodes per litre.
- **j:** Secondary treated effluents with intestinal nematodes > stated number can be used if necessary precautions for workers and consumers can be taken.

Similarly, Hardisty et al. (2013) reviewed a number of strategies for wastewater treatment and discharge and examined the effect of each in relation to Western Australian conditions. The study identifies that adapting wastewater treatment to higher standards would benefit the environment and people; however if these standards become too strict requirements would exceed what is economic and sustainable for WWTPs.

Life cycle assessment (LCA) is a procedure utilised to quantify the various environmental impacts, inputs and outputs of a specified process or action throughout its life cycle. Rodríguez-Garcia et al. (2011) monitored the performance of 24 WWTPs using LCA (reviewed by Larsen et al., 2007). The study divided the WWTPs based on six topologies which were classified according to treatment.
requirements, discharge points and water use. These six topologies were:

1. WWTPs aimed at removing organic matter from the wastewater where treated wastewater was discharged to non-sensitive area.
2. WWTPs aimed at removing organic matter and nutrients from the wastewater where treated wastewater was discharged to non-sensitive area.
3. WWTPs aimed at removing organic matter and nutrients from the wastewater where treated wastewater was discharged to sensitive area.
4. WWTP reused treated effluent for irrigation of agricultural land.
5. WWTP reused treated effluent for industrial purposes.
6. WWTP reused treated effluent for aquifer recharge.

Two functional units were used in the LCA study. These were volume (m$^3$) and eutrophication reduction (kg PO$_4^{3-}$ removed). The study found a correlation between the standards required and the treatment methods utilised to achieve these standards. The study concluded that:

- WWTPs designed for the removal of organic matter would have reduced environmental impacts and economic costs when using the volume unit.
- When utilising the volume unit to review the more demanding topologies, the treatment costs would be increased in order to obtain reduced eutrophication potential. The danger of global warming potential would also be increased.
- If the eutrophication reduction unit was utilised the drawbacks of the demanding topologies would be overcome.

Wang et al. (2012) utilised the LCA procedure in a model of six anaerobic/anoxic/oxic WWTPs to assess the different wastewater treatment standards in China. The Chinese EPA has specified four classes of treatment standards, which are Classes 3, 2, 1B and 1A, where Class 1A is the strictest (Table 2.1). The six scenarios under investigation are shown in Fig. 2.1.
This study found that the treatment required to achieve the stricter standards resulted in increased energy usage and global warming potential. These studies recommend that treatment processes must be designed to cope with variations of standards in the future.

2.3 Wastewater Treatment Processes

Efficient treatment of wastewater contaminants requires multiple processes. The number of stages used and the type of treatment utilised is dependent on the wastewater characteristics and discharge standards. Most treatment plants consist of 3 – 4 stages of treatment.

2.3.1 Preliminary treatment

This stage includes grit removal, screening and flow equalisation. The preliminary treatment focuses on removing large solids from the wastewater in order to prevent damage or interference of subsequent treatment operations (US EPA, 1999). Flow equalisation stores wastewater so that a constant flow through the treatment plant is obtained.
2.3.2 Primary treatment

Primary treatment processes are designed to remove a portion of the SS and organic matter from the wastewater. There are physical and chemical primary treatment processes. The physical processes include:

- Sedimentation: this is the separation of solids that are heavier than water from the wastewater by gravitational settling (Matko et al., 1996).
- Flotation: microscopic air bubbles are released into the reactor from the bottom and are attached to the suspended solid particles. These solids then rise to the water surface and join with other particles to form a blanket that is removed mechanically (Rubio et al., 2002).

Chemical primary treatment processes include:

- Neutralisation: this process aims to ensure that the pH of the effluent is at an acceptably neutral level (Goel et al., 2005).
- Coagulation: this process is used for the removal of fine solids. The solids settle out and are removed from the unit mechanically or manually (Bratby, 2006).

2.3.3 Secondary Treatment

Secondary treatment is designed to remove biodegradable organic matter which is dissolved or suspended. It also includes biological nutrient removal which is achieved through a combination of aerobic and anaerobic conditions. Aerobic processes are performed in the presence of oxygen by aerobic micro-organisms. There are a number of variations in the aerobic biological treatment processes. One of the most widely utilised forms of secondary treatment is the activated sludge (AS) process. Activated sludge processes consist of a tank or basin containing a suspension of biomass. The mixed liquor is mixed vigorously through the use of mixing and/or aeration devices, which also serve to supply the oxygen to the process. Staged reactor designs have been developed for AS processes to generate separate anoxic, anaerobic and aerobic stages (Metcalf and Eddy, 2003). On completion of the AS process, the SS in the treated wastewater are removed through sedimentation in a secondary clarifier.
Various forms of secondary treatment processes can be used separately or in combination to increase treatment efficiency. Biological processes utilizing one or more unaerated and aerated zones have been successfully used to remove carbon (C), N and P from wastewater. The microorganisms growing in the wastewater seek the greatest energy yield in order to have maximum synthesis (Hammer and Hammer, Jr., 2011). For organisms to reproduce and function correctly energy, C, inorganic elements and nutrients are required. Organisms can be classified by the methods they utilise C for cell growth. Heterotrophic organisms obtain C from organic matter, whereas autotrophic organisms obtain C from carbon dioxide (Davies, 2005). AS plants develop microbial communities to promote the metabolic activity of different organisms.

2.3.4 Tertiary treatment

Tertiary treatment is designed for the removal of the residual SS, for disinfection and for nutrient removal. Common processes used in tertiary treatment include:

- Depth filtration: the removal of particulate matter by passing the wastewater through a filter bed comprised of a granular or compressible filter medium (Rodgers et al., 2011). It can be used as a conditioning step for more effective disinfection, and/or as a pre-treatment for membrane filtration.
- Chemical P removal: the removal of P is performed through the addition of coagulants (Fernandes et al., 2012). Coagulants include calcium, aluminium and iron salts.
- Disinfection: A number of disinfection options exist such as chlorine, ozone or UV rays for the removal of bacteria and viruses from wastewater (Metcalf and Eddy, 2003).

A typical layout of a WWTP consisting of preliminary treatment, primary treatment, secondary treatment and disinfection is shown in Fig. 2.2.
Following tertiary treatments the treated wastewater can be discharged into the receiving water body, or receive further treatment if deemed necessary.

### 2.3.5 Continuous Flow treatment configuration

Continuous flow is a popular configuration for WWTPs in which the influent and effluent flow occurs continuously. It is widely used due to the ease with which wastewater treatment processes can be designed and large volumes of wastewater can be continuously fed into the treatment processes. WWTPs utilising continuous flow can consist of 2 – 3 secondary treatment units. These include aeration tanks, anaerobic tanks and secondary clarifiers. The hydraulic retention time (HRT) within these reactors usually ranges from 3 – 8 hrs but can be lengthened if considered necessary.

During the early stages of wastewater treatment the continuous system was considered a better option in comparison to batch treatment (Wilderer et al., 2000). This resulted in significant research being carried out on the various continuous treatment technologies. It is therefore a proven configuration which can be adapted for multiple influents due to the range of treatment options available. There are, however, a number of disadvantages, which include (Tomaszek, 2005; Pace and Harlow, 2000):

- High energy consumption,
- High operational costs,
- Problems with the disposal of large amounts of sludge,
• Future variations in influent flow or concentrations, which can reduce its effectiveness, and
• Space requirements for sufficient treatment may exceed what is available or affordable.

These drawbacks can lead to limitations in the available treatment processes, or increased waste transportation costs to alternate sites. In cases of high or variable influent, or if the influent wastewater contains high concentrations of metals, chemicals etc., the required treatment processes can lead to complex and costly designs.

Macleod and Haygarth (2010) found that satisfying the increasing demand for water resources while avoiding degradation of ecosystems constituted a complex challenge that should require viable answers following economic, social and environmental criteria. Designing a WWTP requires accounting for effluent requirements, local conditions, operational costs and environmental issues, which results in increasing complexity and difficulty (Flores-Alisma et al., 2010; Poch et al., 2004). An alternative to continuous flow plants is batch treatment configurations, such as SBRs (Singh and Srivastava, 2011). Venkata Mohan et al. (2005) found that SBRs performed relatively better than conventional continuous flow systems in terms of C removal.

2.4 Sequencing Batch Reactors

2.4.1 SBRs and their development

In SBRs metabolic reactions and the separation of the influent and effluent streams take place in a specified and repeated operation sequence (Morgenroth and Wilderer, 2007). They are increasingly popular due to their reliability, cost effectiveness and high efficiency. Batch treatment processes also include preliminary treatment and tertiary treatment stages similar to those utilised in continuous flow treatment, such as screening, grit removal and disinfection. Due to the cycle system used by SBRs flow equalisation is often necessary. Compared to continuous flow treatment SBR systems have a number of advantages including (Pace and Harlow, 2000):
• As all biological reactions occur within a single reactor, the space requirements are reduced in comparison with continuous flow treatment.
• SBRs are less complex and more flexible than continuous flow configuration, as the control system can be amended when necessary.

Activated sludge processes, in the form of SBRs, were first demonstrated by Ardern and Locketts (1914). The solids which settled within the reactor were not removed, which allowed for sufficient solids concentrations to develop. The results led to a number of SBR systems being installed at full scale in England. These systems first appeared within the US in 1915, in Milwaukee, Wisconsin. The study was undertaken by Hatton (1916a, b, c), with a cycle length of 6 hours, consisting of 60 min fill, 210 min react (aeration), 30 min settle and 60 min draw. Although O’Shaughnessy’s (1923) observations indicated that continuous flow systems would require an extended treatment period, SBR systems were converted to continuous flow systems in the 1920s for three reasons (Arden, 1927):
1. The high energy dissipation during the decant process,
2. Clogging of the coarse bubble diffusers due to the repeated sludge settling, and
3. Maintenance of the treatment devices resulting in increased operating costs.

Research on batch processes in the USA began again in Pennsylvania by Hoover et al. (1951), Hoover and Porges (1952) and Hoover (1953) who used this process to treat dairy industry wastewater. Researchers in the EU introduced the technology to municipal treatment systems. Studies by Pasveer (1960, 1961 and 1969) resulted in the re-engineering of fill and draw operations. Advancements of this process were performed in studies such as those by Irvine. Irvine and Davis (1971) utilised SBRs for a single basin reactor, which was used for the full scale treatment of industrial wastewater. The static fill process was developed by Dennis and Irvine (1979) which utilised an unmixed unaerated period. Since the 1970s, further developments have occurred, with more variations of the SBR technology appearing.

2.4.2 SBR Operation
The primary treatment phases of SBRs consist of fill, react, settle, draw and idle. Fill
involves the pumping of the influent into the reactor for treatment. This can occur all at once at the beginning of the cycle, or in some systems the wastewater can be input in portions over time, referred to as a step feed mode. The fill modes in use include:

- Static Fill: no energy input to the system. It also allows accumulation of substrates.
- Mixed Fill: mixing without forced aeration occurs, thereby allowing minimal aerobic activity. It typically allows for either anoxic or anaerobic reactions.
- Aerated Fill: mixing with forced aeration. The aeration often allows simultaneous anoxic and aerobic reactions.

The fill phase is followed by the react phase. The react phase involves the treatment of the waste in aeration and non-aeration periods. The change from aeration to non-aeration periods impacts the growth rates and relationships among the various species inside the reactor, and allows the nitrification and denitrification processes to occur. A number of sub-phases are present within the react phase, including:

- Mixed React: This involves mixing the reactor contents without forced aeration, thereby resulting in minimal aerobic activity. This sub-phase allows for anoxic and anaerobic conditions.
- Aerated React: This is mixing with forced aeration, which allows for aerobic reactions to occur.

The order in which these phases are set is also important. In order to maximise P removal the anaerobic stage must occur first in order to allow the PAOs to utilise the available organic substrates. In this stage the denitrification process also occurs.

On completion of the react phase the settle phase follows in order to allow SS to settle out of suspension, thus leaving behind the treated water. Effluent is decanted during the draw phase. The idle phase, which can be eliminated, follows the decant phase and is prior to the next cycle. Its duration is dependent on the actual design flows of the plant. A diagram indicating these phases in an SBR operation cycle is shown in Fig. 2.3.
2.4.3 Modified Sequencing Batch Reactors

Methods of enhancing cSBR treatment processes have been developed and researched. These alternate batch technologies are briefed herein.

2.4.3.1 Anaerobic Sequencing Batch Reactor (ASBR)

An ASBR is a high rate anaerobic AS process where the react phase remains unaerated. The anaerobic metabolic organisms of this process produce methane (CH$_4$) and carbon dioxide. The advantages of ASBRs include:

- Operational simplicity,
- Reduced operation costs,
- Efficient quality control of the treated effluent,
- Flexibility of use, such as seasonal control, and,
- High biogas yield.

ASBRs have been researched and tested for their versatility on various wastewaters such as dairy, piggery, landfill leachate, brewery, soymilk based substrate and wastewaters containing high concentrations of organic matter (Dugba and Zhang, 1999; Shao et al., 2008; Pinho et al., 2005; Siman et al., 2004). Hulshoff-Pol et al. (1998) studied the various mixing options available after observing that agitation was necessary to improve the transfer of the substrate to the micro-organisms in the granulated biomass. The agitation was required due to the lack of consistency in the reaction medium. Ratusznei et al. (2000) and Pinho et al. (2005) observed the effect of mechanical stirrers on the performance of ASBRs and anaerobic sequencing batch biofilm reactors (ASBBRs). Studies on recirculation and recycling of the liquid and/or gas phases as an alternate agitation option have also been performed (Brito et
al., 1997; Venkata Mohan et al., 2007). The performance of the ASBR process is directly related to biomass settlement and the self-immobilisation process utilised by most SBRs cannot always achieve good settleability.

2.4.3.2 Sequencing Batch Biofilm Reactor (SBBR)

SBBRs are an SBR modification which utilises biofilms instead of AS. SBBRs utilise the bacterial cell grown on biofilm medium surfaces within the reactors for the treatment of wastewater. These biofilms can be utilised for the biological removal of N and P (Fu et al., 2010; Wilderer and McSwain, 2004; Zhang et al., 2009). SBBR units present a number of advantages, including:

- They achieve a higher biomass concentration per unit reactor (Kim and Yang, 2005; Di Iaconi et al., 2003) which aids the process when treating high strength wastewater.
- Biofilm reactors allow for land and energy savings, operation flexibility and greater volumetric loads (Rodgers and Zhan, 2003).
- They achieve increased resistance to shock loadings (Singh and Srivastava, 2011).
- They utilise simultaneous nitrification and denitrification (SND) for N removal due to anoxic zones existing in biofilms (Choi et al., 2008).

SBBRs have been used in a number of studies. Gonzalez et al. (2009) successfully utilised SBBRs for the treatment of wastewater contaminated with biorecalcitrant pharmaceuticals. Ding et al. (2011) studied the effectiveness of an SBBR utilising an intelligent controlling system (ICS) for the treatment of raw domestic sewage. Unlike timer or PLC controlled SBBRs the ICS-SBBR operation was dependent on the temperature and dissolved oxygen (DO) concentrations within the reactor. The SBBR had a working volume of 20 L and utilised an operation of instantaneous fill, 7 hour react and 10 minute decant. The effects of various C/N ratios were also tested using synthetic wastewater. In comparison to a conventional SBBR, the ICS-SBBR reduced the HRT and the total aeration time by 56% and 50%, respectively. Removal rates for ammonium (NH$_4^+$-N), COD and TP of 99%, 96% and 100% were achieved, respectively. The C/N ratio of 12.5 was considered optimum for the SND process. Jin et al. (2012) reviewed various control systems for SBBRs. An ICS-SBBR was
compared with a conventional timer control system SBBR (TCS-SBBR). The control systems were tested at various C/N ratios of 10, 5, and 3.3. The SBBRs had working volumes of 20 L and sludge retention time (SRT) of 26 days, and utilised fibre threads as biofilm carriers. It was found that the ICS-SBBR reduced both the reaction time and the aeration time by 180 and 157 minutes, respectively. At the C/N ratio of 5, the ICS-SBBR had a TN removal efficiency of 81%, as opposed to TN removal of 65.4% in the TCS-SBBRs.

### 2.4.4 Other SBR Variations

Table 2.3 summarizes some studies on the various forms of SBRs.

<table>
<thead>
<tr>
<th>Reference</th>
<th>SBR Type</th>
<th>Study Details</th>
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<td>Ra et al. (1999)</td>
<td>Two-Stage SBR</td>
<td>BNR with an internal organic C source from piggery wastewater</td>
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<td>Timur &amp; Ozturk (1999)</td>
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<td>Mac et al. (2003)</td>
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<tr>
<td></td>
<td></td>
<td>shock loads on ASBR treating slaughterhouse wastewater at 20°C</td>
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<tr>
<td>Hu et al. (2005)</td>
<td>SBR</td>
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<td>Kapdan &amp; Ozturk (2005)</td>
<td>SBR</td>
<td>Effects of sludge age and initial dyestuff concentration on colour and COD</td>
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<td>Obaja et al. (2005)</td>
<td>SBR using internal C</td>
<td>BNR using an internal organic C source from digested piggery wastewater</td>
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<tr>
<td></td>
<td>source</td>
<td></td>
</tr>
<tr>
<td>Holman &amp; Wareham (2005)</td>
<td>SBR</td>
<td>COD, ammonia and DO time profiles in the SND process</td>
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<tr>
<td>Sarti et al. (2007)</td>
<td>ASBR</td>
<td>Pilot-scale ASBR for domestic sewage treatment</td>
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<tr>
<td>Chan &amp; Lim (2007)</td>
<td>SBR</td>
<td>Evaluation of SBR performance with aerated and unaerated fill periods in</td>
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<td>treating phenol-containing wastewater</td>
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<td>Sirianunthapiboon &amp;</td>
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<td>Treatment of wastewater containing Pb$^{2+}$ and Ni$^{2+}$</td>
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<td>Boonchuppleeng (2009)</td>
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<td>Aslan et al. (2009)</td>
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<tr>
<td>Ni et al. (2009)</td>
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<td>low-strength municipal wastewater</td>
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<td>Youwei et al. (2009)</td>
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<td>On-line monitoring for P removal process and bacterial community</td>
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<tr>
<td>Daniel et al. (2009)</td>
<td>SBR</td>
<td>Removal of ammonium via SND nitrite-shortcut in a single packed bed batch</td>
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<td>Johnson et al. (2010)</td>
<td>SBR</td>
<td>Influence of ammonium on the accumulation of polyhydroxybutyrate in aerobic</td>
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<td>open mixed cultures</td>
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<tr>
<td>Lim et al. (2011)</td>
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<td>N removal using polyurethane foam cubes of various sizes as carrier materials</td>
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<td>Kim et al. (2011)</td>
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<td>waste using temperature-phased ASBR</td>
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<td>Reference</td>
<td>SBR Type</td>
<td>Study Details</td>
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<td>Malakahmad et al. (2011)</td>
<td>SBR</td>
<td>Removal of Hg$^{2+}$ and Cd$^{2+}$ from synthetic petrochemical factory wastewater</td>
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<td>Stone et al. (2011)</td>
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<td>Shariati et al. (2011)</td>
<td>Membrane SBR</td>
<td>The effect of HRT on the performance and fouling characteristics of membrane SBRs used for the treatment of synthetic petroleum refinery wastewater</td>
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<td>Giorgetti et al. (2011)</td>
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<td>Saraphirom &amp; Reungsang (2011)</td>
<td>Granular &amp; floc biomass SBR</td>
<td>Biological hydrogen production from sweet sorghum syrup by mixed cultures</td>
</tr>
<tr>
<td>Sanchez et al. (2011)</td>
<td>Granular SBR</td>
<td>Tertiary membrane filtration of an industrial wastewater using granular or flocculent biomass SBRs</td>
</tr>
<tr>
<td>Ding et al. (2011)</td>
<td>SBBR</td>
<td>Domestic sewage treatment with an intelligent controlling system</td>
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<tr>
<td>Hu et al. (2011)</td>
<td>SBR</td>
<td>Effect of anoxic/aerobic phase fraction on N$_2$O emission under low temperature</td>
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<td>Bernat et al. (2011)</td>
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<tr>
<td>Changyong et al. (2011)</td>
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<td>Nitrite accumulation in real-time control</td>
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<tr>
<td>Chen et al. (2011)</td>
<td>Granular SBR</td>
<td>Operational strategies for N removal in granular SBR</td>
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<tr>
<td>Trabelsi et al. (2011)</td>
<td>SBR</td>
<td>Coupling short-time SBR and coagulation settling process for co-treatment of landfill leachate with raw municipal wastewater</td>
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<tr>
<td>Peng et al. (2012)</td>
<td>Double react settle SBR</td>
<td>Nitrite accumulation in a continuous system treating low strength domestic wastewater</td>
</tr>
<tr>
<td>Aziz et al. (2011)</td>
<td>SBR</td>
<td>Powdered activated C augmented double react-settle SBR process for treatment of landfill leachate</td>
</tr>
<tr>
<td>Won &amp; Ra (2011)</td>
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<td>Biological N removal with a real time control strategy using moving slope changes of pH (mV) and ORP time profiles</td>
</tr>
<tr>
<td>Bassin et al. (2011)</td>
<td>SBR</td>
<td>Nitrification of industrial and domestic saline wastewaters</td>
</tr>
<tr>
<td>Jin et al. (2012)</td>
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<tr>
<td>Lim et al. (2012)</td>
<td>Moving bed IASBR</td>
<td>Enhancement of N removal with intermittent aeration</td>
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<tr>
<td>Daverey et al. (2012)</td>
<td>SBR</td>
<td>N removal from opto-electronic wastewater using the simultaneous partial nitrification, anaerobic ammonium oxidation and denitrification (SNAD) process</td>
</tr>
<tr>
<td>Li et al. (2012)</td>
<td>IASBR</td>
<td>Partial nitrification of landfill leachate with varying influent composition under intermittent aeration conditions</td>
</tr>
</tbody>
</table>

### 2.5 Intermittently Aerated Sequencing Batch Reactors

#### 2.5.1 IASBR Technology

IASBRs are another variation of cSBR treatment processes, which utilise intermittent aeration during the react phase. Otawa et al. (2006) stated that IASBRs were cost effective for the reduction of BOD, N and P contaminants. Due to the process’s flexibility and efficiency a number of studies have utilised IASBRs for the
treatment of a variety of wastewaters, including:

2.5.1.1 Domestic Wastewater
The intermittent aeration technology was used for the removal of N from domestic wastewater by Zhao et al. (1999). This study investigated control factors for SND in a 2-stage intermittently aerated process designed for N and P removal. The 2-stage process utilised an anaerobic zone followed by an oxidation reduction potential (ORP) controlled intermittently aerated completely mixed tank (IACM). The influent to be treated was of residential origin and the study found that under low DO conditions utilising the intermittent aeration process, TN removal was 50% as opposed to fully aerobic conditions which resulted in only 15% TN removal. Increased aeration periods also favoured sequential nitrification and denitrification (SQND), rather than SND.

2.5.1.2 Digested Swine Manure
Cheng et al. (2001) and Mota et al. (2005a) utilised intermittent aeration reactors for the treatment of digested swine manure, achieving complete nitrification during the aeration periods, followed by denitrification via nitrate during the non-aerated periods. A further study by Mota et al. (2005b) investigated the effects of the length of the aeration and non-aeration periods on N removal and the nitrifying bacteria community in 5 intermittently aerated reactors run in parallel and treating digested swine wastewater. The 5 reactors had different aeration and non-aeration period ratios and it was observed that the effluent ammonia concentrations and biomass concentrations were not significantly different among the reactors. Increased non-aeration periods affected the nitrite oxidising bacteria (NOB) more than the ammonia oxidising bacteria (AOB). The reactor with the longest non-aeration time performed partial nitrification followed by denitrification via nitrite. Partial nitrification and denitrification via nitrite reduced the reactor oxygen demand for nitrification and reduced the required organic substrate for denitrification. The other four reactors performed conventional nitrification and denitrification via nitrate.

2.5.1.3 Slaughterhouse Wastewater
Experiments utilizing IASBRs for the treatment of slaughterhouse wastewater have been carried out by the Civil Engineering Department at NUI Galway. The results
showed that at an influent organic loading rate of 1.2 g COD/L·d⁻¹, average COD, TN and TP removals of 96%, 96% and 99%, respectively, were achieved (Li et al., 2008a).

2.5.1.4 Saline Wastewater

Uygur (2006) utilised an IASBR for the treatment of saline wastewater at various salt concentrations. The IASBR was operated with an SRT of 10 days and had an operation layout of anaerobic, oxic, anoxic and oxic processes of 1 hr, 3 hr, 1 hr and 1 hr, respectively, and a settling period of 45 minutes. It was observed that initially as salt concentrations increased, the nutrient removal efficiency declined. This was due to the adverse effect of salts on microorganisms (Lawton et al., 1957; Stewart et al., 1962; Kincannon et al., 1966). Halobacter halobium, a salt tolerant organism, can be utilised to improve the treatment of the saline wastewater (Panswad and Anan, 1997; 1999). COD, NH₄⁺-N and ortho-phosphate (PO₄³⁻-P) removal rates were compared at various salt concentrations when using and not using the salt tolerant halobacter in the IASBR. The halobacter resulted in consistent higher removal rates at all salt concentrations, with peak removals of 31 mg COD/g biomass/hr, 1.45 mg NH₄⁺-N/g biomass/hr and 0.18 mg PO₄³⁻-P/g biomass/hr at salt concentrations of 6%.

2.5.1.5 Meat Product Processing Wastewater

Rodriguez et al. (2011) performed a study which monitored the N removal efficiency in an IASBR for the treatment of wastewater from a meat product processing company. Two types of wastewater were investigated in this study:

1. Waste generated from washing vehicles, equipment, cans and packages containing meat products.
2. Water condensate generated from the process of transforming raw materials into animal feed using cooker processes.

The SBR cycle was 8 hrs and consisted of fill, react, settle and draw, with three cycles performed a day. The react period lasted for 6 hrs and consisted of 8-minute aeration followed by 15-minute non-aeration repeated until the end of the react period. Multiple tests were performed at various COD and NH₄⁺-N loading rates (OLR and ALR) and the best performance was achieved at an OLR and ALR of 2.49
g COD/L/d and 1.02 g NH$_4^+$-N/L/d, respectively, with 71% NH$_4^+$-N removal. Other studies have utilised IASBRs for the treatment of other types of wastewater, like piggery wastewater (Obaja et al., 2003).

### 2.5.2 Factors Affecting the Operation of IASBRs

Various factors can affect the efficiency of the IASBR processes, and the design and operation of IASBRs must consider these factors in order to achieve the desired results. These factors include the fill strategy (Guo et al., 2007), the cycle time control of the treatment process (Lemaire et al., 2008), the aeration rate and influent COD/N ratio.

The aeration rate controls the flow of oxygen into the system. Changes in the aeration rate can enhance treatment processes such as nitrification and COD removal. However if the aeration rate is not properly monitored and maintained according to the influent loading rates and cycle time, the DO available may be insufficient to achieve the desired removal rates, or alternatively can inhibit processes that require low oxygen concentrations, such as denitrification. For example, a study by Li et al. (2008b) investigated the effect of the aeration rate on the nutrient removal efficiencies of an IASBR in the treatment of slaughterhouse wastewater by examining performance at four aeration rates (0.2, 0.4, 0.8 and 1.2 L air/min), and found that 0.8 L air/min was the optimum aeration rate, achieving COD, TN and TP removal rates of 97%, 94% and 97%, respectively.

The COD/N ratio is indicative of the concentration of organic C available in comparison to N in the incoming influent. In order to achieve satisfactory N removal in the denitrification process, sufficient C concentrations are required. Therefore a low COD/N ratio will have a negative impact on denitrification. Tsuneda et al. (2006) observed that simultaneous N and P removal with a low C/N ratio was not achievable due to the lack of C substrates available. Various methods have been developed in order to enhance treatment processes affected by low COD/N ratios, such as operating at low aeration rates in order to achieve SND or through the addition of an external C source. Puig et al. (2008) observed that external C sources increased the sludge production and treatment costs. Li et al. (2011) utilised IASBRs to achieve efficient long term partial nitrification at moderately low temperature with
low COD/N ratios. This study utilised synthetic wastewater containing readily biodegradable COD concentrations of 100–300 mg/L and NH$_4^+$-N concentrations of 300 mg/L. DO concentrations during aeration periods were controlled at 0.2 mg/L and the aeration was terminated when NH$_4^+$-N concentrations had reached 20 mg/L. At the COD/N ratio of 1, effluent NH$_4^+$-N, nitrite (NO$_2^-$-N) and nitrate (NO$_3^-$-N) concentrations were about 19 mg/L, 170 mg/L and 20 mg/L, respectively. At a COD/N ratio of 1/3, effluent NH$_4^+$-N, NO$_2^-$-N and NO$_3^-$-N concentrations were about 20 mg/L, 250 mg/L and 20 mg/L, respectively.

2.6 Mathematical Modelling of Wastewater Treatment Systems

2.6.1 Mathematical Modelling

Modelling has developed from a pure research instrument into a commercially valuable and widely used tool. A model is considered to be a representation of an idea or a system that can be utilised to describe and explain processes which cannot be experienced directly for a number of reasons. Daigger (2011) described a model as an abstract representation of the real world which is used to support decision making. Various forms of models can be developed, including conceptual, graphical and mathematical models.

- A conceptual model is utilised to help the modeller know and understand the subject they are investigating. These models represent human intentions or semantics. Conceptual models are chosen independently of design or implementation concerns.

- Graphical models are variations of statistical models. Statistical models describe how random variables are related to each other. Graphical models utilise graphs to represent these relations. They are predominantly utilised in probability theory and statistics.

- Mathematical models utilise mathematical concepts and language in order to describe the necessary system. These models are widely utilised in numerous fields such as physics, biology and engineering disciplines.

Models can be used to develop supervisory control systems, increase understanding of treatment systems and improve wastewater treatment plant control. Olsson (2012)
stated that a model’s usefulness does not depend on its completeness but instead on whether it is helpful in decision making. Various definitions of the roles and uses of models have been specified in studies by Russel et al. (2002), Hulsbeek et al. (2002), and Peterson et al. (2002) who have stated that models can be applied in the following roles/purposes:

- **Learning models** are able to provide increased understanding of WWTP processes. Depending on the learning objective this can even be done with an un-calibrated model. Brdjanovic et al. (2000) increased their understanding of a full scale biological-P process using a TUDP model. Copp (2002) found that models can be used to predict the behaviour of WWTPs under various weather conditions. However when utilising a model in a service role the model must be calibrated to be a representation of the ‘true plant’.

- **Design models** can be utilised to simulate various design alternatives. They can also be used to identify consequences which may occur in the treatment processes. Eldyasti et al. (2011) used the AQUIFAS and BioWin modelling software to compare a circulating fluidised bed bioreactors (CFBBR) operation at various empty bed contact times (EBCT) and volumetric loading rates. The model data was compared with pilot-scale CFBBR experimental data.

- **Process optimisation and analysis models** allow the user to identify the most efficient treatment processes, investigate various forms of treatment controls and review the various impacts and problems of each method. Again a ‘true plant’ representation must be developed in order to test various control and operation strategies. Magri and Flotats (2008) found that modelling could be used as an efficient tool for developing treatment strategies. Hong et al. (2007) utilised a neural network model to review real time estimation of nutrient concentrations and to overcome delay problems. Separate networks were used for the anaerobic and aerobic conditions and the resultant model was applied to a bench scale SBR. The study successfully estimated $\text{NH}_4^+\text{-N}$, $\text{PO}_4^{3-}\text{-P}$ and $\text{NO}_3^\text{-N}$ concentrations using the online DO, pH and ORP information.

Applying WWTP models in these roles provides a number of advantages, such as:
Utilised as a learning tool, models provide a visual representation of plant dynamics.

Utilised for a design and process optimisation role, models can provide a cheap and effective means of testing various alternatives which can minimise future costs and allow for a shortened design period.

Hellinga et al. (1999) stated that the use of models can help towards linking full scale and laboratory scale applications.

### 2.6.2 WWTP Modelling

Modelling is being continuously developed with the understanding of the mechanisms and dynamics of BNR, anaerobic digestion, clarification and settling (Olsson et al., 2012). As research into modelling continues an increasing number of processes can be incorporated into models. To accurately achieve predictions of these processes and their interactions with each other, a mathematical understanding of each process is required. These processes must be identified along with the kinetics and stoichiometry of each process in order to achieve sufficient accuracy. The kinetics and stoichiometry of the processes refer to the rate-concentration dependence and the relationships between components in various reactions, respectively. Various models for activated sludge processes have been developed.

#### 2.6.2.1 Activated Sludge Model No. 1 (ASM1)

General acceptance of WWTP modelling was a result of ASM1 (Henze et al., 1987). ASM1 allows for further insight into biological N and C removal from municipal wastewater, and has been implemented into most commercial simulation and modelling software for N removal in WWTPs. It was developed by a task group set up by the International Association on Water Pollution Research and Control (IAWPRC) with the aim of a common model for N removal in AS systems. During the development of the ASM1 model the processes which occurred during wastewater treatment in an AS system were noted, and the fundamental kinetics of each were expressed as mathematical equations. The ASM1 model is displayed in the form of a matrix (Table 2.4).
Table 2.4. Process kinetics and stoichiometry for C oxidation, nitrification, and denitrification in ASM 1 (Scientific and Technical Report No. 9, 2000)
When using the matrix form of ASM1, the rate equations and stoichiometric coefficients, discussed in further detail in Appendix F, can be combined to simulate the differential processes within the model. These values are combined using the conversion rate equation:

\[ r_i = \sum_j v_{ij} \rho_j \]  (2.1)

where \( r_i \) is the component to be simulated, \( v_{ij} \) is the stoichiometric coefficient of the component for the relevant processes and \( \rho_j \) is the kinetic equations of the processes relevant to the component. For instance by combining the heterotrophic growth process in aerobic and anoxic conditions with the decay of heterotrophs, the heterotrophic biomass concentrations and variations can be simulated by the ASM1 model. The complete equation for heterotrophic biomass kinetics is:

\[
\frac{dX_{B,H}}{dt} = \dot{u}_H \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{S_O}{K_O,H + S_O} \right) X_{B,H} + \dot{u}_H \eta_e X_{B,H} \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{S_NO}{K_NO,H + S_NO} \right) - b_H X_{B,H} \]  (2.2)

By utilising these equations with calibrated kinetic and stoichiometric parameters, the various dynamics of the AS process can be simulated. Roeleveld and van Loosdrecht (2002) state that the ASM1 model can still be considered state of the art for modelling AS systems and is the core of other models. However, the ASM1 model has limitations (Scientific and Technical Report No. 9, 2000), which are:

- The model operates at a constant temperature, but temperature variations affect many biological process kinetics.
- The rate equation coefficients are assumed to be constant, despite the fact that they vary with time.
- The denitrification correction factors are constant and are therefore not affected by variations in system configuration.
- The hydrolysis processes are coupled and are assumed to occur simultaneously at equal rates.

In order to overcome these limitations constant research has been carried out and further ASM models have been developed in order to achieve more accurate simulations. When ASM1 was first developed, enhanced biological phosphorus
removal (EBPR) was not widely used or sufficiently understood and so was not included within the model. Therefore, as EBPR grew in popularity the ASM2 model (Henze et al., 1995) was developed to include the biological P removal process and chemical P removal via precipitation.

2.6.2.2 Activated Sludge Model 2 (ASM2)

This model combines the biological processes for COD, N and P removals and was developed by Henze et al. (1995). To achieve these simulations the ASM1 model was extended and the number of processes considered was increased. The equations of these processes are presented and described in Appendix F. This model also increases the complexity of the stoichiometric coefficients for certain components by utilising continuity equations and specific conversion factors. These continuity equations are calculated with the following layout:

$$\sum v_{ji} \cdot i_{ci} = 0 \quad (2.3)$$

where $v_{ji}$ represents the stoichiometric coefficient of Component i in Process j, and $i_{ci}$ represents the conversion factor. The increase in the model complexity is to allow the model to characterise the wastewater, and to deal with EBPR. The processes, components, stoichiometric coefficients, stoichiometric and kinetic parameters and rate equations are all displayed in the form of a matrix in Table 2.5.
<table>
<thead>
<tr>
<th>Component</th>
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</tr>
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<tbody>
<tr>
<td>Aerobic growth of X</td>
<td>S</td>
</tr>
<tr>
<td>Aerobic Hydrolysis</td>
<td>X</td>
</tr>
<tr>
<td>Storage of X</td>
<td>TSS</td>
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<tr>
<td>Fermentation</td>
<td>S</td>
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<tr>
<td>Redissolution</td>
<td>S</td>
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<tr>
<td>Denitrification</td>
<td>S</td>
</tr>
<tr>
<td>Lysis of X</td>
<td>S</td>
</tr>
</tbody>
</table>

### Observed Conversion Rates

- \( \text{Process Rate, } \rho \)
- \( \text{Kinetic Parameters: } \gamma \)
- \( \text{PAOs: } q \)
- \( \text{Metal-Hydroxides: } K \)
- \( \text{PAOs: } K \)
- \( \text{PAOs: } b \)
- \( \text{PAOs: } \eta \)
- \( \text{PAOs: } O_2 \)
- \( \text{PAOs: } NH_4 \)
- \( \text{PAOs: } ALK \)
- \( \text{PAOs: } PHA \)
- \( \text{PAOs: } PP \)

### Process Rate

- \( \rho \)
- \( \gamma \)
- \( q \)
- \( K \)
- \( b \)
- \( \eta \)
- \( O_2 \)
- \( NH_4 \)
- \( ALK \)
- \( PHA \)
- \( PP \)

### Kinetic Parameters

- \( \gamma \)
- \( q \)
- \( K \)
- \( b \)
- \( \eta \)
- \( O_2 \)
- \( NH_4 \)
- \( ALK \)
- \( PHA \)
- \( PP \)

### Table 2.5 Process and stoichiometry for COD oxidation, denitrification and P removal in ASM2 (Scientific and Technical Report No. 9, 2000)
Although ASM2 incorporated the EBPR process into the ASM1 model, further research on EBPR revealed limitations in the model as it failed to account for denitrifying PAOs. This has since been demonstrated to be an important aspect of PAOs in a modelling context (Mino et al., 1995). Therefore the simulation accuracy of this model is insufficient for use as a design tool. This indicates that further study on the ASM2 model is necessary. The ASM2d model was developed to account for the denitrifying PAOs present in AS systems (Henze et al., 1999).

### 2.6.2.3 Activated Sludge Model 2d (ASM2d)

This model improves simulation of the phosphate and nitrate dynamics. The model is similar to ASM2; however some variations have been made to the processes, rate equations and the stoichiometric parameters, which are presented in Appendix F. The variations to this model are shown in the form of a matrix as described in Table 2.6.
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The inclusion of these new processes and the changes to the stoichiometric coefficients result in changes to the dynamic equations for some components. The modifications made to the ASM2 model in ASM2d results in accurate simulations of nitrate and phosphate dynamics. Yagci et al. (2005) used the ASM2d model to simulate the performance of EBPR in SBRs. The model was calibrated utilising data gained from a laboratory-scale SBR which had a total volume of 4 L and a total cycle time of 6 hrs. The unit treated synthetic wastewater which utilised acetate as the only C source. Four variations of COD/TP ratios were tested which were 400/20, 400/35, 400/45 and 400/60, respectively. After sufficient data had been gathered from the studies, ASM2d was calibrated using an iterative calibration algorithm for EBPR, shown in Fig. 2.4. The model predicted the treatment performance well; however it was necessary to recalibrate it for each data set.

![Figure 2.4. Proposed iterative calibration algorithm for EBPR (Yagci et al., 2005)](image_url)
The ASM2 and ASM2d models are formed by adapting the ASM1 model to account for P removal along with C and N removal. Another adaptation of the ASM1 model is the ASM3 model which was developed by Gujer et al. (2000) for biological N removal from WWTPs. It was developed to rectify the limitations of the ASM1 model.

2.6.2.4 Activated Sludge Model 3 (ASM3)

The primary amendment made to the ASM1 model during the development of ASM3 is the assumption that all readily biodegradable substrates are taken up and stored into an internal cell component prior to biomass growth, thereby recognising the importance of polymer storage. Therefore ASM3 does not consider biomass growth directly on external substrates, unlike ASM1. ASM3 is also considered to be easier to calibrate; Gernaey et al. (2004) states that this is due to all state variables being directly influenced by a change in a parameter value in ASM1, whereas the direct influence is considerably lower in ASM3. The matrix form of the ASM3 model is shown in Table 2.7 and the equations are discussed in greater detail in Appendix F.
Table 2.7. Process kinetics and stoichiometry for C oxidation, nitrification and denitrification in ASM3 (Gujer et al., 1999) (Scientific and Technical Report No. 9, 2000)

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</table>

Autotrophic organisms, nitrifying activity

2 Aerobic storage of $S_o$ | $x_2$ | -1 | $y_2$ | $z_2$ |
3 Anoxic storage of $S_o$ | $-y_3$ | $x_3$ | $y_3$ | $z_3$ |
4 Aerobic growth of $X_A$ | $x_4$ | $y_4$ | $z_4$ | 1 | $-\frac{x_4}{y_4}$ | $\frac{y_4}{z_4}$ |
5 Anoxic growth (denitrification) | $-y_5$ | $x_5$ | $y_5$ | $z_5$ | 1 | $-\frac{x_5}{y_5}$ | $\frac{y_5}{z_5}$ |
6 Aerobic endogenous respiration | $x_6$ | $y_6$ | $z_6$ | $f_1$ | -1 | -1 | $-\frac{1}{f_1}$ |
7 Anoxic endogenous respiration | $x_7$ | $y_7$ | $z_7$ | $f_1$ | -1 | -1 | $-\frac{1}{f_1}$ |
8 Aerobic respiration of $X_{TOT}$ | $x_8$ | 1 | -1 |
9 Anoxic respiration of $X_{TOT}$ | $-y_9$ | $x_9$ | $y_9$ | $z_9$ | -1 | 1 |

Autotrophic organisms, nitrifying activity

10 Aerobic growth of $X_A$ | $x_{10}$ | $y_{10}$ | $f_{x_A}$ | $1/y_{x_A}$ | $-\frac{x_{10}}{y_{10}}$ | $\frac{y_{10}}{f_{x_A}}$ |
11 Aerobic endogenous respiration | $x_{11}$ | $y_{11}$ | $f_{x_A}$ | $1/y_{x_A}$ | $-\frac{x_{11}}{y_{11}}$ | $\frac{y_{11}}{f_{x_A}}$ |
12 Anoxic endogenous respiration | $y_{12}$ | $-\frac{x_{12}}{y_{12}}$ | $x_{12}$ | $f_{x_A}$ | $1/y_{x_A}$ | $-\frac{x_{12}}{y_{12}}$ | $\frac{y_{12}}{f_{x_A}}$ |

Composition Matrix $i_k$

k | Conservatives
1 | ThOD | $g_{ThOD}$ | -1 | 1 | 1 | 1 | -L71 | -L57 | 1 | 1 | 1 | 1 | 1 | 1 |
2 | Nitrogen | $g_N$ | $f_{x_N}$ | $f_{x_N}$ | 1 | 1 | 1 | $f_{x_N}$ | $f_{x_N}$ | $f_{x_N}$ | $f_{x_N}$ |
3 | Ionic Charge | Mole | 1/14 | -1/14 | -1 |

Observables

4 | SS | $g_{SS}$ | $f_{x_{SS}}$ | $f_{x_{SS}}$ | $f_{x_{SS}}$ | $f_{x_{SS}}$ | $f_{x_{SS}}$ | 0.6 | $f_{x_{SS}}$ |

Observed Conversion Rates ($ML^{-3}T^{-1}$)

$\bar{r}_i = \sum_j n_j \rho_j$
Similarly to the other ASM models, the dynamics of the various components are described by combining the stoichiometric coefficients and the rate equations. ASM3 is considered to perform better in situations where the storage of readily biodegradable substrate is significant or when the WWTP is undergoing significant anaerobic/anoxic conditions (Koch et al., 2000).

2.6.2.5 Example of application of ASM models

The ASM models have been widely utilised in numerous studies. Some of these are summarised in Table 2.8.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Model</th>
<th>Study description</th>
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<tbody>
<tr>
<td>Hvala et al. (2001)</td>
<td>ASM1</td>
<td>Design of a SBR sequence with an input load partition in a simulation based experimental environment</td>
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<td>Smets et al. (2002)</td>
<td>ASM1</td>
<td>Linearization of ASM1 for fast and reliable predictions</td>
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<tr>
<td>Lee et al. (2002)</td>
<td>ASM1 combined with membrane fouling model</td>
<td>Modelling of submerged membrane bioreactors for wastewater treatment</td>
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<tr>
<td>Stare et al. (2005)</td>
<td>ASM1 (modified)</td>
<td>Modelling, identification, and validation of models for predictive ammonia control in a wastewater treatment plant</td>
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<tr>
<td>Plattes et al. (2006)</td>
<td>ASM1</td>
<td>Modelling and dynamic simulation of a moving bed bioreactor for the treatment of municipal wastewater</td>
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<td>Holenda et al. (2007)</td>
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<td>DO control of the AS wastewater treatment process using model predictive control</td>
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<td>Hanhan et al. (2011)</td>
<td>ASM1</td>
<td>Mechanism and design of intermittent aeration activate sludge process for N removal</td>
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<td>Katsogiannis et al. (1998)</td>
<td>ASM2</td>
<td>Adaptive optimization of a nitrifying SBR</td>
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<tr>
<td>Furumai et al. (1998)</td>
<td>ASM2 (modified)</td>
<td>Modelling long term nutrient removal in a SBR</td>
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<tr>
<td>Marsili Libelli et al. (2001)</td>
<td>ASM2d</td>
<td>Implementation, study and calibration of a modified ASM2d for the simulation of SBR processes</td>
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<td>Kim et al. (2001)</td>
<td>ASM2</td>
<td>SBR system for P removal</td>
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<td>Hong &amp; Bhamidimarri (2002)</td>
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<td>Evolutionary self-organising modelling of a municipal wastewater treatment plant</td>
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<td>Serralta et al. (2004)</td>
<td>ASM2d</td>
<td>An extension of ASM2d including pH calculation</td>
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<td>ASM2d</td>
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<tr>
<td>Sin and Vanrolleghem (2006)</td>
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<td>Evolution of an ASM2d-like model structure due to operational changes of an SBR process</td>
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<td>Magri et al. (2009)</td>
<td>ASM2</td>
<td>Process simulation and microbial community analysis of liquid fraction of pig slurry by intermittent aeration</td>
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<td>Fan and Xie (2011)</td>
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<td>Mannina et al. (2012)</td>
<td>ASM2</td>
<td>Uncertainty assessment of ASM2 for biological N wastewater treatment plant</td>
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<tr>
<td>Koch et al. (2001)</td>
<td>ASM3</td>
<td>Prediction of N removal and sludge production</td>
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<td>ASM3 with two-step nitrification–denitrification</td>
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<td>Ni et al. (2010)</td>
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<td>Ferrai et al. (2010)</td>
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<td>Fang et al. (2011)</td>
<td>ASM3</td>
<td>Biological N removal in a full-scale wastewater treatment plant</td>
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</table>

### 2.6.3 Modelling of SBR process

SBR treatment processes are generally operated with constant predefined treatment stages. These stages are completely flexible and can be modified depending on the wastewater type and volume. However, variations in wastewater strength and volume can affect treatment requirements and therefore these predefined stages may result in low treatment efficiency (Aguado et al., 2009). Nasr et al. (2011) stated that the flexibility provided by SBR processes has resulted in the development of models in order to identify the ideal combination of growth conditions for various microorganisms. In order to successfully simulate SBR treatment, mass balance equations are utilised to help indicate the effects various components have on each other. The mass balance equation for the total reactor volume is:

\[ V_T = V_o + QT_F \]  \hspace{1cm} (2.4)

where \( V_T \) is total reactor volume, \( V_o \) is the initial reactor volume before the fill phase and during the idle phase, \( T_F \) is the fill time and \( Q \) is the flow rate in the fill phase.

One commonly accepted assumption in SBR models is that biological conversion only occurs during the react phase and not during the settle, decant or idle phases. This means that soluble components are unchanged outside of the react phase, but particulate components are affected by wasting the sludge. The mass balance equation for particulate compounds is:

\[ X_{i0} = \frac{V_T X_{i0}(1 - 1/m\theta_s)}{V_o} \]  \hspace{1cm} (2.5)

where \( X_{i0} \) is the concentration of the particulate component at the beginning of the next cycle, \( V_T \) is total reactor volume, \( X_{i0} \) is concentration of the particulate at the end of the react phase, \( m \) is the number of treatment cycles in one day, \( V_o \) is the initial reactor volume and \( \theta_s \) is the sludge age. This equation has two assumptions:
1. The excess sludge wasted at the end of the react phase is dependent on the SRT; and
2. The settling process is considered to be ideal with no solids loss in the effluent.

SBRs have been successfully simulated in numerous studies, some of which are summarised in Table 2.9.

### Table 2.9. SBR simulation studies

<table>
<thead>
<tr>
<th>Reference</th>
<th>Software</th>
<th>Study description</th>
</tr>
</thead>
<tbody>
<tr>
<td>de Kreuk et al. (2006)</td>
<td>Aquasim</td>
<td>Kinetic model of nutrient removal in a granular sludge SBR</td>
</tr>
<tr>
<td>Luccarini et al. (2009)</td>
<td>ProM Framework</td>
<td>Formal verification of wastewater treatment processes using events detected from continuous signals by means of artificial neural networks</td>
</tr>
<tr>
<td>Aguado et al. (2009)</td>
<td>Artificial Neural Network</td>
<td>A methodology for SBR identification with artificial neural networks</td>
</tr>
<tr>
<td>Ganigué et al. (2010)</td>
<td>Matlab</td>
<td>Partial nitrification of landfill leachate in a SBR</td>
</tr>
<tr>
<td>Kashani &amp; Shahhosseini</td>
<td>Artificial Neural Network</td>
<td>Modelling batch reactors using generalised dynamic neural networks</td>
</tr>
<tr>
<td>Ni et al. (2010)</td>
<td>AquaSIM</td>
<td>Evaluate the impacts of predators on biomass components and oxygen uptake in SBR and continuous systems</td>
</tr>
<tr>
<td>Flores-Alsina et al. (2011)</td>
<td>BSM2G</td>
<td>Including greenhouse gas emissions during benchmarking of wastewater treatment plant control strategies</td>
</tr>
<tr>
<td>Belchior et al. (2012)</td>
<td>BSM1</td>
<td>DO control of the AS wastewater treatment process using stable adaptive fuzzy control</td>
</tr>
<tr>
<td>Hernandez-del-Olmo et al. (2011)</td>
<td>BSM1</td>
<td>An emergent approach for the control of WWTP by means of reinforcement learning techniques</td>
</tr>
<tr>
<td>Fan et al. (2012)</td>
<td>Matlab</td>
<td>A kinetic modelling for C metabolism in SBR under multiple aerobic/anoxic conditions</td>
</tr>
<tr>
<td>Ganigué et al. (2012)</td>
<td>Matlab</td>
<td>Impact of influent characteristics on a partial nitrification SBR treating high N loaded wastewater</td>
</tr>
<tr>
<td>Nasr et al. (2012)</td>
<td>Matlab</td>
<td>Application of artificial neural network for the prediction of El-AGAMY wastewater treatment plant performance</td>
</tr>
</tbody>
</table>

These models have been designed using various modelling and simulation software, which are detailed in Table 2.10.
Table 2.10. Modelling Software options

<table>
<thead>
<tr>
<th>Modelling Software</th>
<th>Software Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AquaSIM (Eawag, Switzerland, <a href="http://www.eawag.ch">www.eawag.ch</a>)</td>
<td>This software was developed to allow users to develop and utilise various aquatic models. This software requires significant user input, as the users define the configuration of the treatment systems. This software requires significant understanding of the relevant treatment processes.</td>
</tr>
<tr>
<td>BioWin (EnviroSim Associates LTD., Canada, <a href="http://www.envirosim.com">www.envirosim.com</a>)</td>
<td>The software utilises linked processes and users have full flexibility to specify the details of various processes. This software is based on Matlab/Simulink. It utilises a number of different model foundations including the AS models (ASM1, ASM2d, ASM3, ASM1sbr, ASM1tm). These foundations are accessible allowing for easy modification.</td>
</tr>
<tr>
<td>SIMBA (ifak system, Magdeburg, Germany, <a href="http://www.ifak-system.com">www.ifak-system.com</a>)</td>
<td>This software is based on Matlab/Simulink. It utilises a number of different model foundations including the AS models (ASM1, ASM2d, ASM3, ASM1sbr, ASM1tm). These foundations are accessible allowing for easy modification.</td>
</tr>
<tr>
<td>STOAT (WRc, Swindon, UK, <a href="http://www.wrcplc.co.uk">www.wrcplc.co.uk</a>)</td>
<td>This software can simulate single treatment processes or various wastewater treatment plant layouts. Models for numerous treatment processes are available, which allows for various designs to be tested. This software utilises model for various processes with different model foundations including the AS models and Mantis models. Single treatment processes or entire wastewater treatment plants can be simulated with various designs. These processes and foundations can be easily modified using the GPS-X software.</td>
</tr>
<tr>
<td>GPS-X (Hydromantis, Ontario, Canada, <a href="http://www.hydromantis.com">www.hydromantis.com</a>)</td>
<td>This software utilises model for various processes with different model foundations including the AS models and Mantis models. Single treatment processes or entire wastewater treatment plants can be simulated with various designs. These processes and foundations can be easily modified using the GPS-X software.</td>
</tr>
</tbody>
</table>

This research utilised the Hydromantis GPS-X software to investigate the SBR and IASBR technology.

2.6.4 GPS-X Modelling Software

GPS-X is a multipurpose computer program which can be used for the modelling and simulation of municipal and industrial WWTPs. It is a dedicated WWTP simulator and contains a library of models, such as ASM1, ASM2, Mantis and Mantis2, which allows for multiple unit operations to be simulated accurately. This prevents GPS-X from being limited to any one specific process. GPS-X allows the user to examine the complex interactions of a WWTP’s unit processes. This can be done both interactively and dynamically (Hydromantis, 2006). Rather than just simulating a single AS process, GPS-X is capable of simulating a WWTP which consists of multiple AS tanks combined with a sedimentation tank. This allows for GPS-X simulations to account for all the plant treatment processes and each of their effects on the effluent quality (Gernaey et al., 2004).

2.6.4.1 GPS-X Layout

The GPS-X software allows the user to build and modify a WWTP by utilising its library of models and through utilising its Objects. GPS-X Objects are
representations of WWTP unit processes. The Objects include various aspects of the treatment process, such as:

- **Hydraulic configuration**: The treatment processes have incorporated the relevant flow equations (influent, effluent and sludge decant etc.).
- **Physical attributes**: The Object dimensions and equipment positions can be easily defined and amended.
- **Operational attributes**: The kinetics and actions performed by the processes are incorporated.
- **Display variables**: Variables affecting the various treatment processes are easily accessible through tables which can allow for sensitivity analysis.
- **Stream labels**: Allows easy identification of various flows going in and out of processes.
- **Sources**: Influent composition can be controlled allowing the impact of numerous loading rates to be investigated.

The Objects utilised in this study are discussed in Section 4.4.1, chapter 4. Once the model Objects have been set up and connected, the objects must be calibrated if this is required to achieve the objective.

### 2.6.4.2 GPS-X Simulation Development and Calibration

Gernaey et al. (2004) suggested a step-wise approach to develop a model into an efficient simulation. The main steps for this approach have been specified in a number of studies (Coen et al., 1996; Peterson et al., 2002; Hulsbeek et al., 2002). During model development the developer must consider:

1. **Model objective/role**: The role of the simulation details the calibration and accuracy requirements and model complexity.
2. **Model selection**: the model role and parameters to be studied affect the choice of models (e.g. ASM1, ASM2d, ASM3) required for the simulations.
3. **Hydraulic flow**: If hydraulic flow control processes and equations are necessary for the simulation software, these must be identified and incorporated into the model.
4. **Influent definition**: the influent volume and characteristics (COD, TN, TP, SS etc.) must be defined.
5. Model testing: after the flow and treatment processes of the model have been prepared simulations must be performed to confirm that the chosen parameters allow the processes to achieve steady state.

6. Model calibration: If calibration is necessary for the simulation the stoichiometric, kinetic and operation parameters must be altered and validated so that the model data can accurately simulate the observed field data.

7. Unfalsification: This step confirms that the simulation data accurately depicts the actions observed in the actual treatment processes and can achieve the objective.

8. After unfalsification the model can be utilised for its intended purpose.

By utilising the Objects available in GPS-X, Steps 1 – 5 can be achieved easily depending on the complexity of the model. At this point the model may require calibration. A specific set of data which has been collected from lab-scale, pilot-scale or full scale plants can be used for the calibration procedure. Whether a model needs to be calibrated depends on its purpose. Peterson (2000) states that if a model is utilised for educational purposes or in situations where qualitative comparisons are sufficient, default parameters can be used and no calibration is necessary. However, if the process performance is being evaluated calibration may be necessary in order to make the model an accurate representation of studied processes. GPS-X models are calibrated using a process engineering approach (Hulsbeek et al., 2002; Peterson et al., 2002). This is based on an understanding of the treatment processes and the model’s structure. Models are calibrated by altering one parameter at a time to achieve an acceptable prediction.

2.6.4.3 Applications of GPS-X in modelling SBRs

GPS-X has been successfully utilised to simulate numerous types of WWTPs in various studies. SBR modelling using GPS-X is very flexible, with various options available depending on the complexity of the SBR and the modelling purpose. Three main SBR Objects are available, which are the SBR, Advanced SBR and Manual SBR. Examples utilising GPS-X for modelling WWTPs are shown in Table 2.11.
Table 2.11. GPS-X modelling of WWTP

<table>
<thead>
<tr>
<th>Reference</th>
<th>Study description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pon et al. (1999)</td>
<td>Evaluating WWTP control strategies</td>
</tr>
<tr>
<td>Hvala et al. (2001)</td>
<td>Design of a SBR sequence with an input load partition in a simulation-based</td>
</tr>
<tr>
<td></td>
<td>experimental environment</td>
</tr>
<tr>
<td>Ky et al. (2001)</td>
<td>Modelling biological P removal from a cheese factory effluent by an SBR</td>
</tr>
<tr>
<td>Filali-Mefnassi et al. (2004)</td>
<td>Treatment of slaughterhouse wastewater in a SBR</td>
</tr>
<tr>
<td>Filali-Mefnassi et al. (2005)</td>
<td>SND process in SBR treating slaughterhouse wastewater</td>
</tr>
<tr>
<td>Nuhohlu et al. (2005)</td>
<td>Modelling AS processes</td>
</tr>
<tr>
<td>Zhi-rong et al. (2005)</td>
<td>Optimization of SBRs</td>
</tr>
<tr>
<td>Andres et al. (2006)</td>
<td>Model-based optimum design of SBRs for COD and N removal from slaughterhouse</td>
</tr>
<tr>
<td></td>
<td>wastewater</td>
</tr>
<tr>
<td>Corominas et al. (2006)</td>
<td>Model-based evaluation of an on-line control strategy for SBRs based on oxygen</td>
</tr>
<tr>
<td></td>
<td>uptake rate (OUR) and oxidation reduction potential (ORP) measurements</td>
</tr>
<tr>
<td>Stricker &amp; Béland (2006)</td>
<td>SBR versus continuous flow process for pilot plant research</td>
</tr>
<tr>
<td>Abdel-Kader (2009)</td>
<td>Comparison study between SBR and conventional AS</td>
</tr>
<tr>
<td>Abdel-Kader (2011)</td>
<td>Grey water treatment efficiency by rotating biological contactor systems</td>
</tr>
<tr>
<td>Wei et al. (2011)</td>
<td>Influencing factors and mathematical model of denitrification phosphate uptake</td>
</tr>
<tr>
<td></td>
<td>process</td>
</tr>
</tbody>
</table>

2.7 Summary

This chapter presents the background of this PhD research. Wastewater treatment processes are reviewed. SBRs and various modified SBRs are described. The kinetic equations of various AS models (ASM1, ASM2, ASM2d and ASM3) are briefed. Modelling of wastewater treatment with simulation software available is summarized, with focus given to the Hydromantis GPS-X software.
Chapter Three

Materials and Methods

3.1 Introduction
Two experimental SBR units, a cSBR unit and an IASBR unit, were designed, constructed and operated to test the treatment efficiency of different operation sequences for synthetic domestic wastewater and real municipal wastewater treatment. Operational data obtained during operation of the units was used to calibrate mathematical models.

3.2 Laboratory Unit Design
The laboratory units consisted of two stainless steel reactors with an internal diameter of 180 mm. The first reactor was constructed at a height of 235 mm and after reviewing the design later on it was decided to construct the second reactor at a height of 280 mm. The units were operated at ambient temperature and had a working volume of 5 litres each. The units were automated with functions controlled by a Siemens PLC. The PLC received input signals from float switches and controlled the peristaltic pumps, magnetic stirrers and air pumps. A schematic layout of the laboratory units is presented in Fig. 3.1.

![Figure 3.1. Laboratory Unit](image)
During each cycle 3 litres of wastewater was treated in each unit resulting in a HRT of 10 hrs, with two Masterflex L/S peristaltic pumps used to fill wastewater and withdraw treated wastewater. The reactors were constantly stirred with magnetic stirrers during the fill and react phases. During the aeration periods air was supplied using an aquarium air pump through permeable stone diffusers located at the bottom of the reactor; the air flow rate was regulated by an air flow meter. The constructed laboratory units and the PLC are shown in Fig. 3.2 a and b, respectively.

![Laboratory unit and PLC](image)

(a)

(b)

**Figure 3.2.** Laboratory unit (a) and PLC (b)

Once a day, 500 ml of mixed liquor was withdrawn from the reactor just before the settle phase, resulting in an SRT of 10 days (without consideration of solids loss in
3.3 Operation Sequences

The operation sequences of the cSBR and IASBR units are detailed in Fig. 3.3, with a SBR cycle duration of 6 hours. Both units had four cycles per day. In each IASBR cycle, there were three alternating 50-minute non-aeration and 50-minute aeration periods. In each cSBR cycle, there was a 150-minute non-aeration period followed by a 150-minute aeration period. Under both operation sequences a mixed fill was utilised which occurred at the start of the react phase and lasted for 10-minutes. The settle, decant and idle periods occurred sequentially after the react period for both operation sequences and were 40, 10 and 10 minutes, respectively.

![Figure 3.3. cSBR and IASBR operation sequences](image)

### 3.3.1 Programmable Logic Controller (PLC)

A PLC, a microprocessor based industrial controller, was used in this study to control the cSBR and IASBR operations. The Siemens PLC utilised Step 7-Micro/Win 32 software. The software was in a ladder logic format and was uploaded to the PLC via a Siemens USB/PPI multi master cable. The ladder logic format requires:

- The circuits are arranged in horizontal lines which the PLC microprocessor scans down through in order of appearance;
- The circuit lines can have multiple inputs but only one output; and
- All instructions must precede the output.

Within this study, the primary inputs were time and the reactor float switches. The outputs were the treatment actions and the fill and decant processes of each reactor.
The treatment actions of mixing, aeration and settlement were controlled using time inputs, while the fill phase, and for a period the decant phase, were controlled using a combination of time inputs and signals received from the float switches. The time inputs were controlled using the PLCs internal clock. The phases of the IASBR and cSBR units and the corresponding actions of the PLC are shown in Table 3.1.

### Table 3.1. IASBR/cSBR periods and corresponding PLC actions

<table>
<thead>
<tr>
<th><strong>cSBR &amp; IASBR Periods</strong></th>
<th><strong>PLC Action</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>The influent pump is turned on according to a specified time input, and turned off by the float switch, or by a maximum allowable operation period.</td>
</tr>
<tr>
<td>Non-Aerated React</td>
<td>The magnetic stirrer is turned on and off according to a specified time input.</td>
</tr>
<tr>
<td>Aerated React</td>
<td>The air pump is turned on and off according to a specified time input. The magnetic stirrer remains active during the aeration periods.</td>
</tr>
<tr>
<td>Settle</td>
<td>The magnetic stirrer and air pump are turned off according to a specified time input. Under the original design a solenoid valve was utilised to discharge the treated effluent from the reactors; however due to logistic problems this layout was replaced by a peristaltic pump which is turned on and off according to specific time inputs.</td>
</tr>
<tr>
<td>Draw</td>
<td>No Actions taken.</td>
</tr>
</tbody>
</table>

Further details regarding the PLC networks are presented in Appendix C.

### 3.4 Sampling and Analysis

Samples of influent wastewater, mixed liquor suspended solids (MLSS) and treated effluent were taken from the reactors to test once a day. Phase testing for typical SBR cycles was performed to examine the actions occurring inside the reactors by taking samples at specific time intervals during a complete cSBR and IASBR treatment cycle. Time intervals ranged from 10 to 75 minutes depending on the treatment phase and the operation sequence.

$\text{NH}_4^+\cdot\text{N}$, $\text{NO}_2^-\cdot\text{N}$, TON and $\text{PO}_4^{3-}\cdot\text{P}$ were analyzed using a Konelab 20 analyzer (Thermo Clinical Labsystems, Vantaa, Finland). SS and MLSS were measured in
accordance with the standard APHA methods (APHA, 1995). BOD$_5$ concentrations were measured with an OxiDirect BOD-System (Lovibond, UK). COD concentrations were measured using the closed reflux method and a Hach spectrophotometer (model DR/2010; Hach, USA). TN and TP were measured with two methods, using TN and TP kits according to the manufacturers’ protocol (Hach, USA) and using a nutrient analyzer (Biotector, BioTector, Analytical Systems Ltd. Ireland) which also measured total organic C (TOC).

DO, pH and ORP were real time monitored using electrodes (HI-9828 multi-parameter electrode, Hanna Instruments, United Kingdom).

### 3.5 Summary

This chapter presents the laboratory unit utilised in this study, including the equipment and the unit setup. The operation sequences are detailed for the cSBR and the IASBR, with focus on the control procedure of the PLC ladder logic code. The sampling and testing methods are also presented.
Chapter 4

Comparison of cSBR and IASBR operations
Chapter Four

Comparison of conventional SBR and IASBR operations for the treatment of synthetic domestic wastewater

4.1 Introduction
Laboratory-scale cSBR and IASBR units (described in Chapter 3) were operated to treat synthetic domestic wastewater. The units were operated at a low aeration rate (LAR) of 0.8 L air/min during Days 1 – 22 and a high aeration rate (HAR) of 1 L air/min during Days 23 - 55.

The laboratory performance data of the cSBR and IASBR was compared and then utilised to validate a GPS-X mathematical model. This was then used to compare the cSBR and IASBR processes.

4.2 Synthetic Domestic Wastewater
The components of the synthetic wastewater included: 403 mg/L sodium acetate, 30 mg/L yeast extract, 120 mg/L dried milk, 30 mg/L urea, 60 mg/L NH₄Cl, 100 mg/L Na₂HPO₄.12H₂O, 50 mg/L KHCO₃, 130 mg/L NaHCO₃, 50 mg/L MgSO₄.7H₂O, 2 mg/L FeSO₄.7H₂O, 2 mg/L MnSO₄.H₂O, and 3 mg/L CaCl₂.6H₂O. Synthetic wastewater was made every 1 – 2 days and constantly stirred by 2 magnetic stirrers during storage. The average concentrations of total COD (CODₜ), 5-day BOD (BOD₅), total nitrogen (TN) and total phosphorus (TP) were 346 mg/L, 230 mg/L, 33 mg/L and 10 mg/L, respectively. The use of synthetic wastewater allowed concentrations to remain consistent in the influent during the study, with little fluctuation.

4.3 Results and Discussion

4.3.1 Effects of the aeration rate on the overall performance of the cSBR and IASBR
The performance of the cSBR and IASBR units at each aeration rate is given in
Table 4.1. In the IASBR unit at the LAR the nutrient concentrations in the effluent were within the discharge standards set by the Irish EPA. The average effluent concentrations of the cSBR unit were also within discharge standards at the LAR; however peak TN concentrations of 24 mg/L exceeded the emission standards. At the HAR, both the IASBR and the cSBR units failed to reach the emission standards for P, with average effluent concentrations of 3.1 mg/L and 4.6 mg/L in the IASBR and cSBR, respectively. The HAR lowered the P removal efficiencies of both units despite improving COD, BOD$_5$ and NH$_4^+$-N removals. NH$_4^+$-N concentrations were decreased from 0.9 mg/L to 0.4 mg/L in the IASBR, and from 1.32 mg/L to 0.4 mg/L in the cSBR when the aeration rate was increased from 0.8 L air/min to 1 L air/min. The HAR deteriorated the TN removal performance of the reactors. Effluent TN concentrations rose from 8.5 mg/L at the LAR to 10.2 mg/L at the HAR in the IASBR and from 12.4 mg/L at the LAR to 13.4 mg/L at the HAR in the cSBR, respectively. The peak TN concentrations of 15.6 mg/L in the cSBR at the HAR exceeded discharge standards.

**Table 4.1. Performance of IASBR and cSBR units at 0.8 and 1 L air/min aeration rates**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>UWWT Standards (mg/L)</th>
<th>IASBR Effluent (mg/L)</th>
<th>cSBR Effluent (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LAR</td>
<td>HAR</td>
</tr>
<tr>
<td>BOD$_5$</td>
<td>25</td>
<td>4.5</td>
<td>4.1</td>
</tr>
<tr>
<td>TSS</td>
<td>35</td>
<td>13.7</td>
<td>10.6</td>
</tr>
<tr>
<td>PO$_4^{3-}$-P</td>
<td>-</td>
<td>0.07</td>
<td>3.1</td>
</tr>
<tr>
<td>NH$_4^+$-N</td>
<td>-</td>
<td>0.9</td>
<td>0.4</td>
</tr>
<tr>
<td>TN</td>
<td>10 – 15</td>
<td>8.5</td>
<td>10.2</td>
</tr>
<tr>
<td>TP</td>
<td>1 – 2</td>
<td>1.3</td>
<td>-</td>
</tr>
<tr>
<td>COD</td>
<td>-</td>
<td>53.5</td>
<td>38.2</td>
</tr>
</tbody>
</table>

*Numbers in bracket are standard deviation.

MLSS concentrations varied slightly in each unit at the two aeration rates. In the IASBR at the LAR the average MLSS concentration was 2.3 g/L. At the HAR the concentration was increased slightly to 2.7 g/L. The increased aeration rate reduced the effluent SS concentrations which may have resulted in the increased reactor MLSS concentrations at the HAR. In the cSBR the average MLSS concentrations were 2.6 g/L and 2.7 g/L at the LAR and HAR, respectively.
Yang et al. (2010) used a novel sequencing batch moving bed membrane bioreactor (SBMBMBR) for simultaneous N and P removal from wastewater, and achieved removal efficiencies of 93.5%, 82.6%, 95.6% and 84.1% for COD, TN, NH\(_4^+\)-N and TP, respectively. The initial average influent COD, NH\(_4^+\)-N, and PO\(_4^{3-}\)-P concentrations were 400 mg/L, 30 mg/L and 4 mg/L, respectively. The nutrient removal performance of the SBMBMBR was studied at different influent COD/TN/TP ratios that ranged from 34.9/6.2/1 to 120.5/9.3/1. These results are similar to those observed in this study (Table 4.1). They also observed a rapid decrease in P removal when the length of the aeration period was increased. The high DO concentrations in the aeration period (up to 6 mg/L) inhibited the N and P removal. Their research confirms that the efficiency of cSBRs and IASBRs is highly affected by the aeration rate.

### 4.3.2 Phase study at the two aeration rates for the IASBR and cSBR

The NH\(_4^+\)-N concentration reductions in the IASBR at the LAR occurred primarily in the aeration periods. Reductions of 5, 6.5 and 2.2 mg/L occurred in Aeration Periods 1, 2 and 3, respectively (Fig. 4.1a). DO concentrations peaked at 0.6, 1.6 and 5.1 mg/L at the end of Aeration Periods 1, 2 and 3, respectively (Fig. 4.1b). In the cSBR unit the NH\(_4^+\)-N concentrations were reduced by 13.42 mg/L during the aeration period (Fig. 4.1c) and the DO concentrations peaked at 4.4 mg/L at the end of the aeration period (Fig. 4.1d).

In the IASBR the oxygen concentrations quickly reduced to zero when the aeration ceased. In Aeration Period 3 DO jumped during the first 5 – 10 minutes after the aeration commenced, and then increased very slowly and levelled off, before it quickly rose. The first DO jump could be due to the completion of organic matter degradation; this phenomenon has been observed by Zhan et al. (2009) and Gieseke et al. (2002). When the nitrification process occurred, oxygen was consumed and DO concentrations levelled off. After completion of the nitrification process, DO concentrations started to increase again. This was observed in the cSBR unit as well with the DO concentrations increasing again after 280 minutes of the react phase, i.e. 30 minutes into the aerated react period. pH was between 7.3 and 8, and between 7.25 and 8 in the IASBR and cSBR, respectively. In the IASBR the peak value was observed in the middle of Non-aeration Period 2 (Minute 120; Fig. 4.1b). In the
cSBR the peak pH value was observed at the beginning of the non-aeration period (Minute 17.5; Fig. 4.1d), and the peak value during the aeration period was observed (Minute 192; Fig. 4.1d). It is well known that the nitrification process decreases pH and denitrification increases pH. During Aeration Period 3 the pH value levelled off after a reduction and then increased. This point, referred to as the pH valley, coincided with the rapid increase of DO indicating completion of the nitrification process. This is also observed in the cSBR unit.

At the HAR in the IASBR the majority of NH$_4^+$-N removal occurred in Aeration Period 1 with a reduction of 11 mg/L (Fig. 4.2a). 1.5 mg/L and 2.5 mg/L were removed in Non-aeration Period 2 and Aeration Period 2, respectively. The DO concentrations peaked at 3 mg/L, 9 mg/L and 9 mg/L at the end of Aeration Periods 1, 2 and 3 (Fig. 4.2b), respectively. The rapid increase of DO concentrations after the completion of nitrification during Aeration Period 2 prevented a clear observation of the DO control point. In the cSBR at the HAR, the NH$_4^+$-N concentrations were reduced by 10.9 mg/L during the aeration period (Fig. 4.2c). The DO concentration peaked at 4.97 mg/L at the end of the aeration period (Fig. 4.2d). The DO in the IASBR slowly reduced to zero during Non-aeration Period 3. In the IASBR the pH gradually decreased from 8.7 after the fill phase to about 7.8 at the end of Aeration Period 1 and then fluctuated between 7.7 and 8.2 in the rest of the cycle. The ‘pH valley’ was observed during Aeration Period 2; however it occurred so quickly that it would be difficult to be used as an operation control point. In the cSBR the pH decreased from 7.57 to 7.34 at the beginning of the aeration period. The pH varied between 7.2 and 7.6 for the remainder of the react period. The ‘pH valley’ was again observed during the aeration period (Minute 260, Fig. 4.2d). It occurred clearly and was easily identified.

In the IASBR at the LAR the TON (NO$_3^-$-N plus NO$_2^-$-N) production during aeration periods was 13.6 mg/L, with productions of 3.1, 8.0 and 2.6 mg/L in Aeration Periods 1, 2 and 3, respectively, indicating that NH$_4^+$-N removed during the aeration periods was converted to NO$_3^-$-N and NO$_2^-$-N. The denitrification process reduced TON during Non-aeration Periods 2 and 3. In the cSBR the TON production was 7.3 mg/L; however no TON was removed during the denitrification process. At the HAR the TON production mainly occurred in Aeration Period 1 in the IASBR, with
productions of 10.5 mg NO$_3^-$-N/L and 1.7 mg NO$_2^-$-N/L. Denitrification was limited during the HAR operation. This was due to DO remaining in the non-aeration periods following the aeration periods. TON production of 11.44 mg/L was observed in the cSBR at the HAR. No denitrification was observed as the TON production was not followed by an anoxic period.

At the LAR, PO$_4^{3-}$-P concentrations decreased from 54 mg/L at the beginning of Aeration Period 1 in the IASBR to 14, 10, 2.0 and 0.9 mg/L at the end of Aeration Period 1, Non-aeration Period 2, Aeration Period 2 and Aeration Period 3, respectively. Concentrations were reduced to 0.4 mg/L by the end of the settle phase (Fig. 4.1a). PO$_4^{3-}$-P concentrations in the cSBR were reduced from 58 mg/L to 1.6 mg/L at the end of the aeration period. Concentrations were reduced further during the settle phase to 0.4 mg/L (Fig. 4.1c). At the HAR, the high DO concentrations resulted in the collapse of the EBPR process (Brdjanovic et al., 1998). In the IASBR the PO$_4^{3-}$-P concentrations were decreased from 35 mg/L at the start of Aeration Period 1 to 10, 8.5 and 6 mg/L at the end of Aeration Period 1, Non-aeration Period 2 and Aeration Period 3, respectively (Fig. 4.2a). PO$_4^{3-}$-P concentrations had reduced to 5.7 mg/L by the end of the settle phase. This decline was also observed in the cSBR unit. PO$_4^{3-}$-P concentrations were reduced from 42 mg/L to 7.4 mg/L at the end of the settle phase. Zhu et al. (2011) compared the effectiveness of a cSBR to a humus soil SBR (HS-SBR) for the treatment of P. PO$_4^{3-}$-P removals were observed to be 97% and 80% in the HS-SBR and cSBR, respectively, less than the P removal observed at the LAR in this study. The peak and average uptake rates of PO$_4^{3-}$-P at the LAR were 1.54 and 0.63 mmol P/hr (or 47.8 and 19.6 mg P/l hr), respectively. The overall uptake rates observed by Zhu et al. (2011) were 1.98 mmol P/hr and 1.45 mmol P/hr in the HS-SBR and cSBR, respectively. Kargi et al. (2005) tested PO$_4^{3-}$-P release and uptake rates with different C sources and observed a peak PO$_4^{3-}$-P uptake rate of 8.1 mg P/L.hr. The high PO$_4^{3-}$-P uptake levels confirm the occurrence of EBPR in the IASBR and cSBR at the LAR.
Chapter 4 Comparison of cSBR and IASBR operations

(a) IASBR Nutrient Phase Data at LAR

(b) IASBR DO and pH Phase Data at LAR
Figure 4.1. NH$_4^+$-N, NO$_2^-$-N, PO$_4^{3-}$-P and NO$_3^-$-N concentrations in the IASBR (a) and cSBR (c) and DO and pH data in the IASBR (b) and cSBR (d) in a typical SBR cycle at the LAR.
Chapter 4 Comparison of cSBR and IASBR operations

(a) IASBR Nutrient Phase Data at HAR

(b) IASBR DO and pH Phase Data at HAR
4.4 Model Development and Calibration

Modelling and parameter simulations were performed using the GPS-X 6.1.1 software (Hydromantis). The ASM2d model was utilised in this study and validated using the laboratory unit cSBR data. Simulations were run for a 200 day period.

Figure 4.2. NH$_4^+$-N, NO$_2^-$-N, PO$_4^{3-}$-P and NO$_3^-$-N concentrations in the IASBR (a) and cSBR (c) and DO and pH data in the IASBR (b) and cSBR (d) in a typical SBR cycle at the HAR.
4.4.1 Simulation Objects

The GPS-X software allows the user to develop and build a wastewater treatment model by utilising its various Objects. The Objects utilised in this simulation were:

- **Influent Wastewater**: This Object controls the wastewater characteristics (contaminant concentrations) and volume in this study. The influent wastewater volume per day was changed to the total influent utilised by the cSBR reactor in one day of operation (4 cycles). The influent wastewater concentrations were the average concentrations in the synthetic wastewater in this study. Figure 4.3 shows the influent advisor window, which contains the wastewater quality utilised for the model validation, sensitivity analysis and simulation studies.

![Figure 4.3. Influent wastewater characteristics used in the mathematical modelling](image)

The influent advisor simplifies the influent defining process. Parameters are interconnected so that changing one parameter can have a knock on effect. These connections can be displayed as equations to help understand the effects the variations can have.
• Equalisation Tank: As the wastewater was treated in the cSBR and the IASBR in batches, the equalisation tank was utilised to control the influent flow and store the influent wastewater without treatment until the start of the next cycle. The flow of wastewater leaving this tank was controlled through Excel sheet commands.

• Manual SBR: Three variations of SBR are available in the GPS-X software:
  - Sequencing Batch Reactor (SBR),
  - Advanced SBR, and
  - Manual SBR.

The Manual SBR is the only variation capable of simulating the IASBR operation sequence. Manual SBRs are controlled using Excel spread sheets which detail the actions of the treatment processes at set time intervals. This study controlled the cSBR and IASBR influent flow, decant flow, sludge decant flow, equalisation tank flow and air flow at L/min. The flow rate values specified depended on the time interval utilised. Increasing the time interval increased the time between each data point, which resulted in fewer values being obtained in each phase; however decreasing the time interval reduced the maximum simulation period. In order to observe that the cSBR and IASBR systems had achieved steady state operation a 5 minute time interval was utilised. This interval allowed for a maximum simulation period of 225 days. An example of the Excel spread sheet is displayed in Table 4.2.

Table 4.2. Manual SBR and Equalisation Tank control sheet: time interval and flow rates detailed for treatment processes.

<table>
<thead>
<tr>
<th>T (min)</th>
<th>qconWastewater r2 (L/min)</th>
<th>qaircon29 (L/min)</th>
<th>mixcon29 (L/min)</th>
<th>qinwastewater2 (L/min)</th>
<th>qcondecant2 (L/min)</th>
<th>qconwaste2 (L/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time interval</td>
<td>Influent flow rate</td>
<td>Air flow rate</td>
<td>Mixer activation</td>
<td>Equalisation Tank effluent flow rate</td>
<td>Effluent flow rate</td>
<td>Sludge waste flow rate</td>
</tr>
<tr>
<td>5 minutes</td>
<td>0.3 L/min</td>
<td>1 L/min</td>
<td>0 is off, 1 is on</td>
<td>0.3 L/min</td>
<td>0.3 L/min during a regular cycle, 0.25 L/min when sludge removed</td>
<td>0.05 L/min</td>
</tr>
</tbody>
</table>
Comparison of cSBR and IASBR operations

- Discharge: This tool is the effluent discharge point for the treated wastewater.

These Objects were combined for the simulation of the laboratory scale wastewater treatment units (Fig. 4.4). The unit layout was tested to identify any problems with the control and physical parameters.

![Simulation unit layout](image)

**Figure 4.4. Simulation unit layout**

### 4.4.2 Model Validation

Simulated cycle phase data and effluent water quality data were compared to the cSBR observed data at HAR in order to validate the model. In this study validation was first performed through the comparison of the effluent water quality data. Parameters were amended step by step in order to minimise the number of parameters changed. The parameters reviewed include:

- **Dissolved Oxygen**: this parameter is primarily affected through amendments to the standard oxygen transfer efficiency (SOTE) value and the alpha factor ($\alpha$). Using the Hydromantis technical reference guide (Hydromantis.com) the SOTE value is calculated using Equation 4.1:

\[
SOTE = A_1 + A_2 \times AF + A_3 \times AF^2 + A_4 \times d + A_5 \times DD
\]

where AF is the air flow rate per diffuser (scfm per diffuser), d is the diffuser submergence (.3m), DD is the diffuser density (diffusers/9.3ft$^2$), and $A_1$, $A_2$, $A_3$, $A_4$, and $A_5$ are regression parameters. $\alpha$ is the ratio of the mass transfer coefficient in wastewater to that in clean water (Rosso and Stenstrom, 2006).

- **Free and Ionised Ammonia**: The stoichiometric and kinetic parameters controlling the nitrification process were considered for this parameter. The stoichiometric parameter reviewed was the autotrophic yield ($Y_A$). The kinetic parameters reviewed were the autotrophic maximum specific growth rate ($\mu_A$) and the autotrophic decay rate ($b_A$).
• TON: The stoichiometric and kinetic parameters controlling the denitrification process were considered for this validation step. The stoichiometric parameter reviewed was the heterotrophic yield ($Y_H$). The kinetic parameter reviewed was the heterotrophic decay rate ($b_H$).

• $PO_4^{3-}$-P: The kinetic parameter controlling the P uptake and release processes were considered for this validation step. The kinetic parameter reviewed was the maximum specific growth rate of poly-P accumulating biomass ($\mu_{PAO}$).

The values of major parameters used to calibrate the model are shown in the Appendices.

The simulation data was compared with the observed effluent water quality data to identify the accuracy of the model. Table 4.4 compares the average effluent data observed in laboratory studies with the simulation data.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Observed Data (mg/L)</th>
<th>Model Data (mg/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$NH_4^+$-N</td>
<td>0.4</td>
<td>0.57</td>
</tr>
<tr>
<td>TON</td>
<td>13.4</td>
<td>12.7</td>
</tr>
<tr>
<td>$PO_4^{3-}$-P</td>
<td>4.6</td>
<td>7</td>
</tr>
</tbody>
</table>

The average TON effluent concentrations had a difference of only 5% which was considered acceptable for this study. The average $NH_4^+$-N concentrations for the simulation and laboratory data were both < 0.6 mg/L, so the simulation data was acceptable. The comparison of the average effluent data for $PO_4^{3-}$-P concentrations showed a significant difference; however the effluent concentrations in the model matched peak effluent concentrations observed in the cSBR of 7.4 mg/L, and both observed and simulated $PO_4^{3-}$-P concentrations exceeded discharge standards.

R-values were calculated to indicate the correlation between laboratory cycle phase data and the corresponding simulation data using Excel spreadsheets. Ratner (2007) stated that the R-value can indicate the strength of the linear relationship between data sets. Strong linear relationships are indicated by an R-value of 0.7 to 1,
moderately linear relationships are indicated by an R-value of 0.3 to 0.7 and weak linear relationships are indicated by R-values of 0 to 0.3. The observed and simulated NH$_4^+$-N concentrations had strong linear relationships with an r-value of 0.9. The TON concentrations had a strong linear relationship with an R-value of 0.88. The PO$_4^{3-}$-P concentrations had a moderately linear relationship with an R-value of 0.55. The cycle phase data for the model and observed data is compared in Fig. 4.5.

The difference in the NH$_4^+$-N concentrations is likely due to further adjustments being required in the influent advisor, with increased organic nitrogen concentrations present in the model in comparison to the synthetic wastewater. A significantly reduced release of PO$_4^{3-}$-P was also observed, due to the decline of EBPR in the model, whereas slight uptake in the laboratory-scale study allowed for increased release of PO$_4^{3-}$-P concentrations.

![NH$_4^+$-N Phase Data Comparison](image_url)

(a) NH$_4^+$-N model and observed phase data comparison
The significant difference between the simulation data and the observed data were then compared using a T-test. T-Tests were performed using Excel spreadsheets. The null hypothesis for this comparison was that there was no significant difference between the simulation and observed data if P values were $\geq 0.05$. The P values for comparison of the $\text{NH}_4^+$-N, TON and $\text{PO}_4^{3-}$-P observed and simulation data were 0.17, 0.7 and 0.06, respectively, indicating that there was no significant difference between the observed and simulation data for $\text{NH}_4^+$-N, TON and $\text{PO}_4^{3-}$-P
concentrations.

### 4.4.3 Sensitivity Analysis

A sensitivity analysis is performed to observe the effects of the large uncertainties in system parameters. The sensitivity of effluent water quality to changes in the parameter values listed in Table 4.5 was evaluated. The GPS-X software contains a sensitivity analysis tool which can be used to study the sensitivity to any relevant parameters (see Appendix C).

The sensitivity analysis was conducted on parameters presented in Table 4.5. The effluent water quality parameters reviewed were COD, TKN, TP, \( \text{NH}_4^+ \)-N and \( \text{PO}_4^{3-} \)-P.

#### Table 4.5. Sensitivity analysis parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Utilised Value</th>
<th>Sensitivity Analysis Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heterotrophic Yield</td>
<td>g COD/gCOD</td>
<td>0.63</td>
<td>0.189 – 0.754</td>
</tr>
<tr>
<td>PAOs Yield</td>
<td>gCOD/gCOD</td>
<td>0.625</td>
<td>0.0625 – 0.625</td>
</tr>
<tr>
<td>Autotrophic Yield</td>
<td>gCOD/gN</td>
<td>0.22</td>
<td>0.072 – 0.288</td>
</tr>
<tr>
<td>Heterotrophic Rate</td>
<td>1/d</td>
<td>6</td>
<td>3 – 8.4</td>
</tr>
<tr>
<td>Heterotrophic Decay</td>
<td>1/d</td>
<td>0.2</td>
<td>0.04 – 0.4</td>
</tr>
<tr>
<td>Denitrification</td>
<td>-</td>
<td>0.8</td>
<td>0.1 – 1</td>
</tr>
<tr>
<td>Reduction Factor</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PAOs Growth Rate</td>
<td>1/d</td>
<td>0.98</td>
<td>0.4 – 1.4</td>
</tr>
<tr>
<td>PAOs Lysis Rate</td>
<td>1/d</td>
<td>0.2</td>
<td>0.03 – 0.3</td>
</tr>
<tr>
<td>Autotrophic Growth</td>
<td>1/d</td>
<td>1.2</td>
<td>0.14 – 1.4</td>
</tr>
<tr>
<td>Autotrophic Decay</td>
<td>1/d</td>
<td>0.08</td>
<td>0.05 – 0.15</td>
</tr>
</tbody>
</table>

The parameters which had the most significant impact on the water quality were the heterotrophic yield (Fig. 4.7a), heterotrophic maximum specific growth rate (Fig. 4.7b), denitrification reduction factor (Fig. 4.7c), PAO maximum specific growth rate (Fig. 4.7d), PAO decay rate (Fig. 4.7e), autotrophic yield (Fig. 4.7f), autotrophic
maximum specific growth rate (Fig. 4.7g) and autotrophic decay rate (Fig. 4.7h). The effluent concentrations taken from the final cycle of the simulation period (Day 200 value) were used to examine the sensitivity.

(a) Sensitivity Analysis: Heterotrophic Yield

(b) Sensitivity Analysis: Heterotrophic Growth Rate
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(c) Sensitivity Analysis: Nitrification Reduction Factor

(d) Sensitivity Analysis: PAO Max Specific Growth Rate
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(e) Sensitivity Analysis: PAO Lysis Rate

(f) Sensitivity Analysis: Autotrophic Yield
(g) Sensitivity Analysis: Autotrophic Max Specific Growth Rate

Figure 4.7: Impacts of the variation of heterotrophic yield (a), heterotrophic growth rate (b), nitrification reduction factor (c), PAOs growth rate (d), PAOs lysis rate (e), autotrophic yield (f), autotrophic growth rate (g), and autotrophic decay rate (h) on effluent COD, TKN, NH$_4^+$-N, TON, TP and PO$_4^{3-}$-P concentrations

### 4.4.3.1 Heterotrophic Yield ($Y_H$)

The $Y_H$ sensitivity analysis utilised minimum and maximum values of 0.189 gCOD/gCOD and 0.754 gCOD/gCOD. Increments of 0.063 gCOD/gCOD were used to obtain 10 data sets.
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The COD effluent concentrations were not significantly affected by changes to the $Y_H$ parameter. A 1.7% increase was observed with minimum and maximum COD concentrations of 56.4 mg/L and 57.4 mg/L, respectively.

The effluent TKN and $NH_4^+$-N concentrations were significantly affected by changes to the $Y_H$ parameter. The TKN concentrations were reduced from 31.8 mg/L to 0.8 mg/L and the $NH_4^+$-N concentrations were reduced from 30.7 mg/L to 0.04 mg/L, when $Y_H$ was increased and autotrophic biomass concentrations were increased. The most significant drop in TKN and $NH_4^+$-N effluent concentrations was observed when $Y_H$ was increased to 0.63 gCOD/ gCOD. The TKN concentrations were reduced from 25.7 mg/L to 1.4 mg/L and the $NH_4^+$-N concentrations were reduced from 24.9 mg/L to 0.6 mg/L. The increased reductions of TKN and $NH_4^+$-N were due to the increased nitrifier biomass. The heterotrophic biomass concentrations were increased by 27% from 1,085.6 mg COD/L to 1,383.5 mg COD/L, and the autotrophic biomass concentrations were increased from 5 mg COD/L to 55.1 mg COD/L.

A significant increase in TON production was observed with effluent concentrations increasing from 1 mg/L to 12.7 mg/L when $Y_H$ was increased to 0.63 gCOD/gCOD. If $Y_H$ exceeded 0.63 gCOD/gCOD the heterotrophic biomass concentrations were reduced from 1,383.5 mg COD/L to 713.9 mg COD/L and effluent TON concentrations were increased to 13 mg/L. The inhibition of the denitrification process by the reduced heterotroph concentrations and the increased nitrification rates resulted in the increased effluent TON concentrations.

Significant variations were observed in the effluent TP and $PO_4^{3-}$-P concentrations when increasing the $Y_H$ parameter. Effluent TP and $PO_4^{3-}$-P concentrations were reduced gradually by 22% and 21.8%, respectively, as $Y_H$ was increased from 0.189 gCOD/gCOD to 0.63 gCOD/gCOD. The effluent TP concentrations were reduced from 10.1 mg/L to 7.9 mg/L and the effluent $PO_4^{3-}$-P concentrations were reduced from 10 mg/L to 7.8 mg/L. Rapid reductions were observed in the effluent TP and $PO_4^{3-}$-P concentrations, respectively, after $Y_H$ was increased to 0.693 gCOD/gCOD. The TP concentrations were reduced from 7.9 mg/L to 0.2 mg/L and the $PO_4^{3-}$-P concentrations were reduced from 7.8 mg/L to 0.01 mg/L. The reduction occurred
due to a significant increase in the reactor PAO concentrations. As $Y_H$ was increased, the PAO biomass concentrations were increased from 0 to 94.6 mg COD/L as $Y_H$ was increased to 0.63 gCOD/gCOD. PAO biomass concentrations had increased to 747 mg COD/L when $Y_H$ was increased to 0.693 gCOD/gCOD. Following the rapid reduction effluent $\text{PO}_4^{3-}$ concentrations were steady at 0.012 mg/L.

### 4.4.3.2 Heterotrophic maximum specific growth rate ($\mu_H$)

The $\mu_H$ sensitivity analysis was tested at minimum and maximum values of 3 d$^{-1}$ and 8.4 d$^{-1}$. Increments of 0.6 d$^{-1}$ were utilised to obtain 10 data sets.

No significant variation was observed in the effluent COD concentrations. A decrease of 4.9% was observed between the maximum and minimum values of 59.3 mg/L and 56.4 mg/L, respectively.

A gradual increase was observed in the effluent TKN and $\text{NH}_4^+$-N concentrations as $\mu_H$ was increased. The TKN concentrations were increased from 0.9 mg/L to 3.1 mg/L and the $\text{NH}_4^+$-N concentrations were increased from 0.2 mg/L to 2.4 mg/L. The increased effluent concentrations were due to the reduced nitrifier concentrations within the reactor. Nitrifier biomass concentrations were reduced by 10.2% from 57.1 mg/L, at 3 d$^{-1}$, to 51.3 mg/L, at 8.4 d$^{-1}$.

Effluent TON concentrations did not vary very much. A decrease of 5.4% was observed between the maximum and minimum values of 12.9 mg/L and 12.2 mg/L, respectively.

After $\mu_H$ was increased to 6 d$^{-1}$, effluent TP concentrations were increased from 0.2 mg/L to 7.9 mg/L and effluent $\text{PO}_4^{3-}$-P concentrations were increased from 0.01 mg/L to 7.8 mg/L. The observed rapid increase in effluent P concentrations was due to a significant reduction in the PAO concentrations from 762 mg COD/L to 94.8 mg COD/L at the min and max $\mu_H$.

### 4.4.3.3 Denitrification Reduction Factor ($\eta_{\text{NO}_3}$)

The $\eta_{\text{NO}_3}$ sensitivity analysis was performed at minimum and maximum value of 0.1 and 1, respectively. Increments of 0.1 were utilised to obtain 10 data points.
No significant variations were observed in the effluent COD effluent concentrations. Minimum and maximum values varied by 4% from 56.5 mg/L to 58.9 mg/L, respectively.

The effluent TKN and \( \text{NH}_4^+ \)-N concentrations were observed to reduce as \( \eta_{\text{NO3}} \) was increased to 0.7. The TKN concentrations were reduced by 79%, from 3.8 mg/L to 0.78 mg/L, and the \( \text{NH}_4^+ \)-N concentrations were reduced by 99%, from 3.1 mg/L to 0.02 mg/L. The low \( \eta_{\text{NO3}} \) inhibited the growth of heterotrophs during the anoxic periods, increasing the COD concentrations in the aerobic periods which inhibited the nitrification process. As \( \eta_{\text{NO3}} \) increased the heterotrophic biomass concentrations increased from its minimum to maximum values of 538.5 mg COD/L and 1,452.4 mg COD/L, respectively. When \( \eta_{\text{NO3}} \) was increased to 0.7 the autotroph biomass concentrations increased from 51.2 mg COD/L to 59.6 mg COD/L. After \( \eta_{\text{NO3}} \) exceeded 0.7 the effluent TKN and \( \text{NH}_4^+ \)-N concentrations both increased from 1.7 mg/L to 0.927 mg/L, respectively. This increase occurred due to a reduction in the nitrifier biomass concentrations to 54.5 mg COD/L due to increased endogenous respiration at the increased heterotroph concentrations.

TON concentrations varied by 12% between the minimum and maximum concentrations of 12 mg/L and 13.7 mg/L due to the variations in the heterotrophic biomass concentrations and the nitrifier biomass. The heterotroph biomass concentrations increased as \( \eta_{\text{NO3}} \) increased and the nitrifier biomass concentrations increased until increased competition from the heterotrophs resulted in endogenous respiration.

The lower \( \eta_{\text{NO3}} \) values resulted in increased PAOs biomass growth and reduced effluent TP and \( \text{PO}_4^{3-} \)-P concentrations of 0.156 mg/L and 0.01 mg/L, respectively. The effluent TP and \( \text{PO}_4^{3-} \)-P concentrations were increased to 7.87 mg/L and 7.82 mg/L when \( \eta_{\text{NO3}} \) was increased to 0.8 due to increased heterotrophic growth. The PAO biomass concentrations reduced from 847.6 mg COD/L to 52.8 mg COD/L as \( \eta_{\text{NO3}} \) was increased from 0.1 to 1.

4.4.3.4 Maximum growth rate of poly-P accumulating biomass (\( \mu_{\text{PAO}} \))

The \( \mu_{\text{PAO}} \) sensitivity analysis was performed with minimum and maximum values of
0.4 d\(^{-1}\) and 1.4 d\(^{-1}\). Increments of 0.1 d\(^{-1}\) were utilised to obtain 10 data sets.

No significant variation was observed in the effluent COD concentrations. The maximum and minimum values of COD were 58.5 mg/L and 58.5 mg/L, respectively.

The most significant variations in the effluent TKN and NH\(_4\)\(^+\)-N concentrations were observed when \(\mu_{PAO}\) was increased from 0.9 d\(^{-1}\) to 1 d\(^{-1}\), 1.3 d\(^{-1}\) and 1.4 d\(^{-1}\). When \(\mu_{PAO}\) was increased to 1 d\(^{-1}\), the TKN concentrations were reduced from 2.7 mg/L to 1.1 mg/L, and NH\(_4\)\(^+\)-N concentrations were reduced from 1.9 mg/L to 0.3 mg/L. The increased NH\(_4\)\(^+\)-N reductions were due to an increase in the nitrifier biomass concentrations from 51.7 mg COD/L to 57.5 mg COD/L accompanying a reduction in heterotrophic biomass concentrations from 1,528.8 mg COD/L to 586.7 mg COD/L. Following this reduction the TKN and NH\(_4\)\(^+\)-N concentrations were increased significantly as \(\mu_{PAO}\) was raised from 1 d\(^{-1}\) to 1.3 d\(^{-1}\). The effluent TKN concentrations were increased from 1.1 mg/L to 11.5 mg/L, and effluent NH\(_4\)\(^+\)-N concentrations were increased from 0.3 mg/L to 10.7 mg/L. The decline in the nitrification process was due to competition from the PAO biomass with the autotrophic biomass. The increased PAO growth resulted in a decline in the nitrifier biomass concentrations to 32.3 mg COD/L. A slight reduction in the TKN and NH\(_4\)\(^+\)-N concentrations was observed after \(\mu_{PAO}\) was raised to 1.4 d\(^{-1}\). The TKN concentrations were reduced from 11.5 mg/L to 7.6 mg/L, and NH\(_4\)\(^+\)-N concentrations were reduced from 10.7 mg/L to 6.8 mg/L. The TKN and NH\(_4\)\(^+\)-N concentrations were reduced as the growth of PAO biomass declined at the increased growth rate. The decline in PAO growth allowed for increased autotrophic growth to 40.8 mg COD/L which caused the reduction in the TKN and NH\(_4\)\(^+\)-N concentrations.

When \(\mu_{PAO}\) was raised to 1 d\(^{-1}\) an increase in the TON concentrations of 6.7% from 11.9 mg/L to 12.7 mg/L was observed due to the enhanced NH\(_4\)\(^+\)-N nitrification rate. The decline in the nitrification process as \(\mu_{PAO}\) was raised to 1.3 d\(^{-1}\) resulted in a decrease in the effluent TON concentrations of 50.4% from 12.7 mg/L to 6.3 mg/L. The slight increase in nitrification when \(\mu_{PAO}\) was raised to 1.4 d\(^{-1}\) resulted in an increase in the TON concentrations from 6.3 mg/L to 8 mg/L.
The most significant variation in the effluent TP and PO$_4^{3-}$-P concentrations was observed after $\mu_{PAO}$ was increased to 1 d$^{-1}$. Reductions of 98.1% and 99.9% were observed in the TP and PO$_4^{3-}$-P concentrations, respectively. The effluent TP concentrations were reduced from 8.3 mg/L to 0.2 mg/L and the effluent PO$_4^{3-}$-P concentrations were reduced from 8.3 mg/L to 0.01 mg/L. The reduced concentrations were due to the increased PAO biomass concentrations from 0 to 778 mg COD/L. As $\mu_{PAO}$ was increased to 1.3 d$^{-1}$ a slight reduction was observed in the effluent PO$_4^{3-}$-P concentrations from 0.01 mg/L to 0.005 mg/L; however the TP concentrations increased slightly by 14.1%, possibly due to the increased effluent solid concentrations. After $\mu_{PAO}$ was increased to 1.4 d$^{-1}$, the effluent TP concentrations reduced due to a reduction in the effluent solid concentrations.

4.4.3.5 Poly-P Accumulating Biomass Lysis Rate ($b_{PAO}$)

The $b_{PAO}$ sensitivity analysis was performed at a minimum and maximum of 0.03 d$^{-1}$ and 0.3 d$^{-1}$, respectively. Increments of 0.03 d$^{-1}$ were utilised to obtain 10 data sets.

A slight decrease of effluent COD was observed as $b_{PAO}$ was increased with minimum and maximum values of 56.4 mg/L and 58.2 mg/L, respectively.

Significant reductions were observed in the effluent TKN and NH$_4^+$-N concentrations when $b_{PAO}$ was increased to 0.06 d$^{-1}$. The TKN concentrations were reduced by 93% from 24.2 mg/L to 1.77 mg/L and the NH$_4^+$-N concentrations were reduced by 96% from 23.4 mg/L to 1.03 mg/L. The concentrations were reduced by 21% when $b_{PAO}$ was increased to 0.09 d$^{-1}$. The nitrification rate increased as $b_{PAO}$ was increased, due to reduced PAOs biomass concentrations within the reactor which reduced the competition between the PAOs and autotrophs for oxygen. The effluent TKN and NH$_4^+$-N concentrations were increased by 32% and 42%, respectively, after $b_{PAO}$ was increased to 0.21 d$^{-1}$. NH$_4^+$-N concentrations were increased at low $b_{PAO}$ values due to the increased PAO concentrations which utilised the DO concentrations. The limited DO inhibited the growth of nitrifiers and reduced the nitrification rate. The heterotrophic biomass concentrations were increased from 390.5 mg COD/L to 1,528.5 mg COD/L and the autotrophic biomass concentrations were increased from 0.01 mg COD/L to 51.6 mg COD/L as the lysis rate was increased and PAO biomass concentrations were reduced from 1,608.8 mg COD/L to
Effluent TON concentrations increased from 0 to 12.2 mg/L due to the increased nitrification rate observed at the $b_{PAO}$ of 0.18 d$^{-1}$. After $b_{PAO}$ was increased to 0.21 d$^{-1}$ effluent TON concentrations were decreased by only 2.5%.

At $b_{PAO}$ values of between 0.03 d$^{-1}$ to 0.18 d$^{-1}$, effluent TP concentrations were increased from 0.15 mg/L to 0.2 mg/L and effluent $PO_4^{3-}$-P concentrations were increased from 0.004 mg/L to 0.01 mg/L. The slight increase in concentrations was due to reductions in the PAO concentrations at the increased decay rate from 1,608.8 mg COD/L to 842.9 mg COD/L. When $b_{PAO}$ was increased to 0.21 d$^{-1}$ the effluent TP and $PO_4^{3-}$-P concentrations were increased from 0.2 mg/L to 8.3 mg/L and from 0.01 mg/L to 8.3 mg/L, respectively, due to the significant reduction of PAO concentrations within the reactor.

4.4.3.6 Autotrophic yield ($Y_A$)

The $Y_A$ sensitivity analysis was performed at minimum and maximum values of 0.072 gCOD/gN and 0.288 gCOD/gN. Increments of 0.024 gCOD/gN were utilised to obtain 10 data sets. An increase of 3.6% was observed between the maximum and minimum values of 56.4 mg/L and 58.5 mg/L, respectively.

A gradual reduction was observed in the TKN and $NH_4^+$-N concentrations as $Y_A$ was increased. The TKN concentrations were reduced from 2.9 mg/L to 0.8 mg/L and the $NH_4^+$-N concentrations were reduced from 2.1 mg/L to 0.01 mg/L. These reductions occurred due to an increase in the nitrifier biomass concentrations from 16.9 mg COD/L to 76.3 mg COD/L.

A slight increase was observed in the effluent TON concentrations as $Y_A$ was increased. As the nitrification rate was increased the TON concentrations were increased by 13.8% from 11.9 mg/L to 13.8 mg/L. The increased nitrification rate and reduced denitrification rate due to reduced heterotroph concentrations caused the increased TON concentrations.

Gradual reductions of 10.1% and 10.2% were observed at first in the effluent TP and $PO_4^{3-}$-P concentrations, respectively, as $Y_A$ was increased from 0.072 gCOD/gN to
0.24 gCOD/gN. The effluent TP concentrations were reduced from 8.4 mg/L to 7.5 mg/L and the PO$_4$$^{3-}$-P concentrations were reduced from 8.3 mg/L to 7.45 mg/L. When $Y_H$ was increased to 0.264 gCOD/gN there was a rapid reduction in the effluent TP and PO$_4$$^{3-}$-P concentrations. The TP concentrations were reduced from 7.5 mg/L to 0.2 mg/L and the PO$_4$$^{3-}$-P concentrations were reduced from 7.45 mg/L to 0.02 mg/L. The P reduction occurred due to a significant increase of the PAO biomass concentrations from 152.3 mg COD/L to 531.9 mg COD/L. The increased PAO biomass concentrations were due to increased availability of DO for aerobic growth at higher $Y_A$ values.

**4.4.3.7 Autotrophic maximum specific growth rate ($\mu_A$)**

The $\mu_A$ sensitivity analysis was performed at minimum and maximum values of 0.14 d$^{-1}$ and 1.4 d$^{-1}$. Increments of 0.14 d$^{-1}$ were utilised to obtain 10 data sets.

A decrease by 1.2% in effluent COD concentrations was observed between the maximum and minimum values of 57.2 mg/L and 56.5 mg/L, respectively.

When $\mu_A$ was increased to 0.84 d$^{-1}$, the effluent TKN and NH$_4$$^{+}$-N concentrations were reduced by 36.3% and 37.7%, respectively. Further reductions were observed as $\mu_A$ was increased due to the increase in autotrophic biomass in the system, leading to a higher nitrification rate. The minimum effluent TKN and NH$_4$$^{+}$-N concentrations were 0.93 mg/L and 0.15 mg/L, respectively. Autotrophic biomass concentrations were increased from 0 to 55.9 mg COD/L.

Effluent TON concentrations were increased from 0 to 12.5 mg/L. The most significant TON concentration increase was observed to coincide with the first decrease in the TKN and NH$_4$$^{+}$-N concentrations. Effluent TON concentrations were increased from 0.1 mg/L to 4.9 mg/L. The TON concentrations peaked at 12.7 mg/L when $\mu_A$ was increased to 1.12 d$^{-1}$. Further $\mu_A$ increments were observed to have a limited impact on the TON concentrations.

When $\mu_A$ was increased to 0.7 d$^{-1}$ the effluent TP and PO$_4$$^{3-}$-P concentrations were reduced from 8.4 mg/L to 0.2 mg/L and the effluent PO$_4$$^{3-}$-P concentrations were reduced from 8.4 mg/L to 0.01 mg/L. Once $\mu_A$ had been increased to 1.26 d$^{-1}$, the TP
and $\text{PO}_4^{3-}$-P concentrations rose rapidly from 0.2 mg/L to 8.2 mg/L and 0.01 mg/L to 8.1 mg/L, respectively. The high P concentrations when $\mu_A$ was less than 0.7 d$^{-1}$ resulted from low PAO concentrations of 0. After $\mu_A$ was increased to 0.7 d$^{-1}$ the low autotroph and heterotroph concentrations of 0.7 mg COD/L and 476.4 mg COD/L reduced the competition with the PAOs which were increased from 0.04 mg COD/L to 806.2 mg COD/L. This caused the rapid reduction in effluent P concentrations. The rapid increase in effluent P concentrations that followed was due to increased competition between the nitrifiers and the PAOs. The increased autotrophic biomass concentrations increased the consumption of DO, inhibiting the aerobic poly-P storage and aerobic PAO growth processes.

### 4.4.3.8 Autotrophic decay rate ($b_A$)

The $b_A$ sensitivity analysis was performed with a minimum and maximum value of 0.05 d$^{-1}$ and 0.15 d$^{-1}$, respectively. Increments of 0.01 d$^{-1}$ were utilised to obtain 11 data sets. A variation of 1% was observed in the effluent COD concentrations between the minimum and maximum values of 56.5 mg/L and 57.1 mg/L, respectively.

A linear increase was observed in the effluent TKN and $\text{NH}_4^+$-N effluent concentrations after $b_A$ was increased to 0.1 d$^{-1}$. The effluent TKN concentrations were increased from 0.97 mg/L to 9.4 mg/L, and the effluent $\text{NH}_4^+$-N concentrations were increased from 0.2 mg/L to 8.7 mg/L. The increased decay rate reduced the concentration of nitrifiers by 45.6% from 55.1 mg COD/L to 30 mg COD/L which reduced the nitrification rate.

A reduction in the effluent TON concentrations was observed as the nitrification rate declined. TON concentrations were reduced gradually after $b_A$ was increased to 0.11 d$^{-1}$. A total reduction of 31% was observed between the maximum and minimum values of 12.9 mg/L and 8.9 mg/L, respectively.

After $b_A$ had been increased to 0.09 d$^{-1}$ the effluent TP and $\text{PO}_4^{3-}$-P concentrations were decreased by 98% and 99.9%, respectively. The effluent TP concentrations were reduced from 7.9 mg/L to 0.2 mg/L and the effluent $\text{PO}_4^{3-}$-P concentrations were reduced from 7.8 mg/L to 0.01 mg/L. The reductions occurred due to reduced
competition between the PAOs and autotrophs. The autotrophic biomass concentration was reduced by 6.5% from 55.1 mg COD/L to 51.5 mg COD/L which coincided with an increase in the PAO biomass concentrations from 94.8 mg COD/L to 761 mg COD/L.

4.4.4 Simulation Data

After validation through the cSBR data the model was utilised to compare the cSBR and IASBR systems. The average COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P concentrations observed in the laboratory study were utilised for these simulations. This was performed through the use of IASBR and cSBR operation layouts in two simulation runs, described in Table 4.6.

<table>
<thead>
<tr>
<th>Simulations</th>
<th>Description</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 1</td>
<td>The calibrated models for cSBR and IASBR were run for 200 days each with review of cycle phase data and effluent water quality data, and autotrophic, heterotrophic and PAOs biomass.</td>
<td>Allowed for the operation sequences to be compared for various biomass and nutrient data.</td>
</tr>
<tr>
<td>Run 2</td>
<td>The calibrated model was run for 200 day period. In the first 100 days, the model was run as a cSBR. And then, at Day 100, the simulation was converted to an IASBR.</td>
<td>Investigated the effects of changes in the operation sequence on treatment efficiency.</td>
</tr>
</tbody>
</table>

Simulated effluent water quality data for COD, TKN, NH$_4^+$-N, TON, TP and PO$_4^{3-}$-P, and cycle phase data for the DO, NH$_4^+$-N, TON and PO$_4^{3-}$-P parameters were obtained after running the model, in addition with biomass concentrations of heterotrophs, autotrophs and PAOs.

4.4.4.1 Run 1 results

The SBR operation sequences utilised were described in Fig. 3.1. Steady state operation was achieved after 35 days and 30 days of operation of the IASBR and cSBR systems, respectively, when the variation of the average biomass concentrations was ignorable. The average effluent water quality concentrations obtained in both simulations are shown in Table 4.7.
Comparison of the average effluent water quality data indicates that the COD concentrations were relatively similar for each operation, with a difference of only 0.4%. The effluent TKN, NH$_4^+$-N, TON, TP and PO$_4^{3-}$-P concentrations were all lower in the IASBR operation than in the cSBR operation. Effluent NH$_4^+$-N concentrations for both simulated operations were within the discharge standards, but the concentrations were lower in the IASBR operation. The effluent TON concentrations in the IASBR operation were 42% lower than in the cSBR operation. When utilising the cSBR operation very little PO$_4^{3-}$-P release and uptake occurred. The IASBR operation achieved much lower PO$_4^{3-}$-P effluent concentrations.

**Table 4.7. Simulation results for effluent water quality in Run 1 (mg/L)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>cSBR</th>
<th>IASBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>55.7</td>
<td>55.5</td>
</tr>
<tr>
<td>TKN</td>
<td>1.26</td>
<td>0.75</td>
</tr>
<tr>
<td>NH$_4^+$-N</td>
<td>0.57</td>
<td>0.1</td>
</tr>
<tr>
<td>TON</td>
<td>12.6</td>
<td>7.3</td>
</tr>
<tr>
<td>TP</td>
<td>7.73</td>
<td>0.96</td>
</tr>
<tr>
<td>PO$_4^{3-}$-P</td>
<td>7.7</td>
<td>0.9</td>
</tr>
</tbody>
</table>

The simulation results indicate that the IASBR operation sequence was more efficient for the denitrification and EBPR processes. Comparison of the TON and PO$_4^{3-}$-P phase data further demonstrates the benefits of the IASBR operation in promoting the denitrification and EBPR processes. The TON and PO$_4^{3-}$-P phase data is compared in Fig. 4.8.
The simulated phase study data presented similar findings to that of the laboratory units. To further investigate and compare the IASBR and the cSBR operations, the model was utilised to gather biomass concentrations of heterotrophs, autotrophs and PAOs. The biomass concentrations followed a similar trend under both operation sequences. Biomass concentrations on the graphs were at their highest during the idle phase when the liquid volume in the reactor was at its lowest. In the discussion below, only biomass concentrations in the react phase were presented. The biomass concentrations remained stable and did not vary significantly in the react phase.
Phase data for all biomass concentrations in Run 1 are shown in Fig. 4.9.

(a) cSBR and IASBR Heterotrophic Biomass simulation phase data comparison

(b) cSBR and IASBR Autotrophic Biomass simulation phase data comparison
Lower concentrations of heterotrophic biomass were observed in the IASBR operation sequence than in the cSBR operation sequence. The average stable concentrations during a single cycle under the cSBR and the IASBR operation sequences were 1,346 and 1,141 mg COD/L, respectively. A difference of 14% was observed in the heterotrophic concentrations. The increased growth observed in the cSBR could be due to more COD available for heterotrophic growth, which resulted from the lower denitrification efficiency.

No significant variation was observed in the autotrophic biomass concentrations under both operation sequences; average concentrations under the cSBR and the IASBR operation sequences were 54 and 55 mg COD/L, respectively.

The most significant variation was observed in the PAOs biomass concentrations. PAO concentrations were significantly higher under the IASBR operation sequence. Average stable concentrations under the cSBR and the IASBR operation sequences were 92 mg COD/L and 354 mg COD/L, respectively.
4.4.4.2 Run 2 results

During Run 2 the effluent water quality data and the biomass concentration data were compared to study how the change in operation from cSBR to IASBR would affect the operation performance. The effluent concentrations are shown in Fig. 4.10.

(a) Run 2 cSBR and IASBR COD effluent concentrations

(b) Run 2 cSBR and IASBR TKN effluent concentrations
(c) Run 2 cSBR and IASBR NH$_4^+$-N effluent concentrations

(d) Run 2 cSBR and IASBR TON effluent concentrations
There was no significant variation in the effluent COD concentrations after the conversion of the operation sequence. An immediate reduction in all N concentrations was observed after operations were converted from a cSBR to an IASBR. The effluent TKN concentrations were reduced from 1.3 mg/L to 0.7 mg/L and the effluent NH₄⁺-N concentrations were reduced from 0.5 mg/L to 0.1 mg/L. The reduction of the NH₄⁺-N concentrations was attributed to the slight increase in autotrophic biomass concentrations present in the IASBR operation sequence (Fig. 4.10).
4.11). Effluent TON concentrations were reduced from 12.7 mg/L to 8.7 mg/L and the intermittent aeration pattern allowed denitrification to occur following the nitrification process. Reductions in the TON concentrations occurred immediately after the conversion to the IASBR operation.

TP and PO$_4^{3-}$-P effluent concentrations were reduced from 7.8 mg P/L to 0.2 mg P/L. The immediate reduction was due to the promotion of PAOs growth by the IASBR operation.

(a) Run 2 cSBR and IASBR Heterotrophic Biomass concentrations

(b) Run 2 cSBR and IASBR Autotrophic Biomass concentrations
Comparison of cSBR and IASBR operations

Figure 4.11. Run 2 phase data for heterotrophs (a), autotrophs (b) and PAOs (c) biomass.

Autotrophic biomass concentrations varied little despite the operation variation. The heterotrophic biomass concentrations were reduced gradually over a 40 day period by 14% from 1,342 mg COD/L to 1,156 mg COD/L. The PAO concentrations experienced the most significant change. Concentrations were immediately increased over a period of 45 days after the operation sequence was changed, from 105 mg COD/L to 357 mg COD/L. This clearly indicates that the IASBR operation promoted the production of PAOs.

4.5 Summary

The treatment efficiency of a cSBR and an IASBR was examined and compared in laboratory and modelling studies. The laboratory study found that the IASBR had superior N removal efficiency, in comparison with the cSBR. The cSBR laboratory data was utilised to construct a model to simulate and further compare the IASBR and cSBR operations. It was found that utilising the IASBR operation sequence resulted in improved N removal via denitrification and P removal. This operation also increased the autotrophic biomass and PAO concentrations in comparison with the cSBR operation.
Chapter 5

Impact of by-passing Primary Clarification on IASBR Treatment

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Chapter Five

Efficiency of IASBRs treating municipal wastewater receiving and by-passing primary clarification

5.1 Introduction
The efficiency of the IASBR technology in treating municipal wastewater receiving and by-passing primary clarification was investigated to assess if it is possible to improve the nutrient removal efficiency of low C municipal wastewater by by-passing primary clarification in an Irish context. The IASBR data was then utilised to validate a mathematical model which further investigated influent variations on IASBR treatment efficiency through a sensitivity analysis and assess the IASBR technology for the treatment of real municipal wastewater.

5.2 Municipal Wastewater
The wastewater used in this study was obtained from the Mutton Island municipal wastewater treatment plant, Co. Galway, Ireland. Wastewater was collected once a week and stored in a room where the temperature was 10 °C until use. During Experiment 1, the wastewater was collected from the primary clarifier effluent (PCE) line, and contained average CODt, TN and TP concentrations of 127 mg/L, 19.6 mg/L and 1.3 mg/L, respectively. This resulted in an average COD/N ratio of 6.5. During Experiment 2, the wastewater was collected from the post grit removal line, which was referred to as the primary clarifier influent (PCI). The PCI contained average CODt, TN and TP concentrations of 306 mg/L, 20.5 mg/L and 4.7 mg/L, respectively. The COD/N ratio was 14.9, on average. The feed utilised and the purpose of each Experiment is detailed in Table 5.1.

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Wastewater Type</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Primary Clarifier Effluent (PCE)</td>
<td>To analyse the effectiveness of the IASBR for the treatment of wastewater after primary clarification</td>
</tr>
<tr>
<td>2</td>
<td>Primary Clarifier Influent (PCI)</td>
<td>To analyse the effectiveness of the IASBR for the treatment of wastewater without primary clarification</td>
</tr>
</tbody>
</table>
5.3 Laboratory Results

5.3.1 Overall performance of the IASBR for the treatment of PCI and PCE wastewater

The reactor was operated during Experiment 1 at an aeration rate of 0.6 L air/min during days 1 – 65. In Experiment 2, the reactor was operated at the same aeration rate during days 66 – 121. The SRT was maintained at 10 days and the HRT was 0.42 days. The performance of the IASBR during the two experiments is given in Table 5.2. During both experiments average effluent concentrations were within the discharge standards set by the Irish EPA (Table 5.2). However, N removal during Experiment 1 was inconsistent, with peak effluent concentrations of 25 mg N/L. The low COD/N ratio deteriorated the overall performance of the IASBR. In Experiment 2 the performance of the system was improved due to the increased COD/N ratio and the average TN effluent concentrations were reduced to 9.9 mg N/L. In their study, Puig et al. (2010) by-passed 27% of the influent flow directly to the biological reactor and found that the by-passing process failed to significantly improve the effluent quality and nutrient removal efficiency, with effluent removal efficiencies for CODt, TKN and TP of 89%, 97% and 71%, respectively, before by-passing and 90%, 97% and 76%, respectively, after by-passing. Their study concluded that despite an increase in the C/N ratio the removal efficiencies and the microbial populations did not improve. Earlier studies using modelling programs under full scale conditions also found no clear conclusion on the effect of by-passing on the N removal efficiency (Exposito et al., 2001; van Veldhuizen et al., 1999).

Table 5.2. Performance of IASBR during the two experiments

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Discharge Standard</th>
<th>Primary Clarifier Effluent</th>
<th>Primary Clarifier Influent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Effluent Concentration</td>
<td>Removal Efficiency (%)</td>
</tr>
<tr>
<td>SS</td>
<td>60</td>
<td>17.6 (4.6)</td>
<td>-</td>
</tr>
<tr>
<td>COD</td>
<td>125-250 or &gt; 75% removal</td>
<td>55 (25.9)</td>
<td>56</td>
</tr>
<tr>
<td>TN</td>
<td>15 – 40 or &gt; 80% removal</td>
<td>10.1 (6.9)</td>
<td>49</td>
</tr>
<tr>
<td>TP</td>
<td>1</td>
<td>1.27 (0.35)</td>
<td>2.3</td>
</tr>
<tr>
<td>NH₄⁺ -N</td>
<td>10</td>
<td>0.47 (0.65)</td>
<td>97.6</td>
</tr>
<tr>
<td>PO₄³⁻ -P</td>
<td>-</td>
<td>1.24 (0.5)</td>
<td>7.7</td>
</tr>
</tbody>
</table>
The MLSS concentrations varied greatly between the two experiments. The low COD concentrations in the feed in Experiment 1 resulted in average MLSS concentrations in the IASBR of 0.95 g MLSS/L. SS concentrations in the effluent were 17.6 mg SS/L. Experiment 1 resulted in a lack of C being available, thus reducing the solids content and preventing proper solid settling during the settlement period (Wei et al., 2003). In Experiment 2 the increased COD levels in the PCI wastewater resulted in a significant increase in the reactor MLSS concentrations with average concentrations of 2.4 g MLSS/L. The increased C concentration also improved the settling capability of AS in the reactor, reducing the effluent SS concentrations to 15.8 mg SS/L, despite an increase in influent SS concentrations. The sludge volume index (SVI) reduced from an average of 105 mL/g during Experiment 1 to 65.6 mL/g during Experiment 2. This reduction of the SVI improved the effluent water quality. Low SVI is thought to be of major importance to SBRs (Ahmad et al., 2007; Rodriguez et al., 2011).

Guo et al. (2011) studied the effects of the C/N ratio on wastewater treatment efficiency by varying the ratio from 1.5 to 6.9. At the low C/N ratio of 1.5 removal efficiencies for NH$_4^+$-N and PO$_4^{3-}$-P of 50% and 46%, respectively, were achieved. At the increased C/N ratio of 6.9, peak NH$_4^+$-N and PO$_4^{3-}$-P removals of 78% and 96%, respectively, were achieved. In the current study, NH$_4^+$-N removal efficiencies in Experiments 1 and 2 were 97% and 98%, respectively. PO$_4^{3-}$-P removal efficiencies were 7.7% in Experiment 1 and 36% in Experiment 2. The relatively poor PO$_4^{3-}$-P removal may be due to the low influent PO$_4^{3-}$-P concentrations.

### 5.3.2 Phase study on the performance of the IASBR unit for the two types of wastewater

During Experiment 1, the NH$_4^+$-N concentration reductions occurred during the aeration periods. Reductions of 3.3 mg/L, 4.3 mg/L and 2.7 mg/L were observed in Aeration Periods 1, 2 and 3, respectively (Fig. 5.1a), corresponding to 32%, 42% and 25% of NH$_4^+$-N reduction, respectively. NH$_4^+$-N concentration reductions also occurred during the non-aeration periods. DO concentrations increased rapidly during the aeration periods and peaked at 7.5 mg/L, 7.3 mg/L and 7.2 mg/L at the end of Aeration Periods 1, 2 and 3, respectively (Fig. 5.1b). The rapid increase in DO concentrations was due to the low concentrations of biomass within the reactor. DO
concentrations slowly reduced to 0 during Non-aeration Periods 2 and 3. This reduced the duration of the non-aeration periods and inhibited the denitrification and P removal processes.

During Experiment 2 the NH$_4^+$-N concentration reductions occurred primarily during Aeration Periods 1 and 2. Reductions of 3.9 mg/L, 2.03 mg/L and 0.02 mg/L were observed during Aeration Periods 1, 2 and 3, respectively (Fig. 5.2a). DO concentrations peaked at 1.7 mg/L, 6.2 mg/L and 7.4 mg/L at the end of Aeration Periods 1, 2 and 3, respectively (Fig. 5.2b). During Experiment 2 an increase was observed in the nitrification rate in comparison to Experiment 1. During Non-aeration Period 3, the DO concentrations slowly reduce to 0 over a 30 minute period. This slow reduction indicates that the aeration rate was too high to promote denitrification during all non-aeration periods. As the nitrification process had not completed prior to Non-aeration Periods 1 and 2 the DO was consumed and reduced rapidly to 0 in these periods.

During Experiment 1 the pH value was between 6.86 and 7.56, with an average value of 7.03. The variation of pH was significantly influenced in the aeration periods during Experiment 1 (Fig. 5.1b).

During Experiment 2, pH was between 6.94 and 7.69, with an average value of 7.1. During Aeration Period 1, pH decreased, indicating that the nitrification process occurred. During Aeration Period 2 in Experiment 2, the ‘pH valley’ was observed (Minute 175, Fig. 5.2b), when pH decreased and levelled off before increasing again. The ‘pH valley’ coincided with the rapid increase in DO, indicating that the nitrification process had been completed. During Experiment 1, the pH values reduced during the non-aeration periods. There was no indication that denitrification occurred. In Experiment 2 an increase in pH was observed during Non-aeration Period 2, indicating that denitrification had occurred.

During Experiment 1 the total TON production in the aeration and non-aeration periods was 13.3 mg/L. The production during Aeration Periods 1, 2 and 3 was 1.5 mg/L, 5.2 mg/L and 4.2 mg/L, respectively (Fig. 5.1a). No denitrification was observed during the non-aeration periods, with further TON production during Non-
aeration Periods 2 and 3 of 1.1 mg/L and 0.7 mg/L, respectively. During Experiment 2 the rapid nitrification process resulted in TON production occurring primarily during Aeration Period 1. Total TON production of 3.62 mg/L was observed after denitrification. Production of TON in Aeration Periods 1, 2 and 3 was 4.2 mg/L, 0 and 0.33 mg/L, respectively. Denitrification was observed during Experiment 2 in Non-aeration Period 2.

No significant EBPR was observed during Experiments 1 and 2. The low P concentrations in the PCE and PCI wastewater inhibited the development of an EBPR process and the low influent PO\textsubscript{4}\textsuperscript{3-}-P concentrations allowed the effluent PO\textsubscript{4}\textsuperscript{3-}-P concentrations to be within the required standards. During Experiment 1 the low influent P concentrations, the high DO concentrations and the low COD concentrations inhibited the EBPR process. PO\textsubscript{4}\textsuperscript{3-}-P release in anaerobic conditions requires high C substrates to occur; however in the PCE which had low COD concentrations the PO\textsubscript{4}\textsuperscript{3-}-P concentrations remained relatively unchanged during the aerated and non-aerated react periods. The increased COD concentrations during Experiment 2 resulted in low uptake of PO\textsubscript{4}\textsuperscript{3-}-P during the aeration periods.
Figure 5.1. NH$_4^+$-N, NO$_2^-$-N, TON and PO$_4^{3-}$-P concentrations (a) and DO and pH data (b) in a typical IASBR cycle for PCE wastewater treatment.
Figure 5.2. \( \text{NH}_4^+ \)-N, NO\(_2\)-N, TON and PO\(_4^{3-}\)-P concentrations (a) and DO and pH data (b) in a typical IASBR cycle for the PCI wastewater treatment.

5.3.3 Alternative cycle layout

During Experiment 1 a second cycle layout, referred to as Layout B, was also utilised (Fig. 5.3). Layout B utilised a shortened aeration period in order to limit the impact of the DO concentrations in the non-aeration periods. The SBR cycle duration was 6 hours with four cycles per day. In each cycle there were four alternating non-aeration and aeration periods. The non-aeration periods were 50-
The results obtained with this operation sequence are presented in Table 5.3. Despite the reduced duration of the aeration periods, no improvement was observed in the denitrification process. Average and peak effluent TON concentrations of 12 mg/L and 22 mg/L were observed, respectively. Average MLSS concentrations in the reactor were 0.74 g/L. Effluent SS concentrations were averaged at 15.7 mg SS/L. Despite the reduced aeration periods continuous nitrification was observed in the aeration and non-aeration periods. No control points, such as the pH valley and DO elbow, were identified.

The NH$_4^+$-N concentration reductions under Layout B occurred during the aeration and non-aeration periods. Reductions of 0.73 mg/L, 1.1 mg/L, 0.4 mg/L and 2.2 mg/L were observed in Aeration Periods 1, 2, 3 and 4 (Fig. 5.4a). The majority of
NH$_4^+$-N concentration reductions occurred in Aeration Period 4. NH$_4^+$-N concentrations also reduced during the non-aeration periods due to the presence of DO. Reductions of 0.8 mg/L, 1.7 mg/L and 1 mg/L were observed in Non-aeration Periods 2, 3 and 4. 9.2%, 13.9%, 5% and 27.7% of NH$_4^+$-N removal occurred in Aeration Periods 1, 2, 3 and 4, respectively, and 10.1%, 1.7% and 12.6% NH$_4^+$-N reduction occurred during Non-aeration Periods 2, 3 and 4. The high DO concentrations were due to the low organic matter content in the reactor. DO concentrations peaked at 7.2 mg/L, 6.96 mg/L, 6.73 mg/L and 6.72 mg/L at the end of Aeration Periods 1, 2, 3 and 4, respectively (Fig. 5.4b). When the aeration was terminated the DO concentrations were reduced to 0 over a prolonged period in every non-aeration period, reducing the duration of the anaerobic periods and thereby inhibiting denitrification. The pH value during Layout B followed a similar trend as Layout A for the treatment of PCE wastewater (Fig. 5.4b).

During Layout B the total TON production in the aeration and non-aeration periods was 4.6 mg/L (Fig. 5.4a). TON production of 0.8 mg/L, 0.9 mg/L, 0.8 mg/L and 0.94 mg/L was observed in Aeration Periods 1, 2, 3 and 4, respectively. TON production of 0.4 mg/L, 0.1 mg/L and 0.5 mg/L was observed in Non-aeration Periods 2, 3 and 4, respectively. Initial TON concentrations of 4.4 mg/L were observed at the beginning of Aeration Period 1.

The low influent P and COD concentrations inhibited the EBPR process during Layout B (Fig. 5.4a).

These results indicate that despite the reduced aeration periods the COD/N ratio continued to inhibit the IASBR treatment efficiency.
Figure 5.4. \( \text{NH}_4^+ - \text{N}, \text{NO}_2^- - \text{N}, \text{TON} \) and \( \text{PO}_4^{3-} - \text{P} \) concentrations (a) and DO and pH data (b) for a typical operation cycle in Layout B.

5.4 Model Development and Calibration

Modelling and parameter simulation were performed using GPS-X and the ASM2d model as described in Section 4.4, chapter 4. The model was validated using the laboratory unit data from the IASBR treating PCI wastewater. Simulations were run for a 200 day period.
5.4.1 Simulation Objects
The Object layout utilised in Section 4.4.1, chapter 4, was utilised for this study. The Excel spread sheet control for this model utilised the same layout and values as the model developed in Section 4.4.1, chapter 4; the aeration rate was 0.6 L air/min in compliance with the municipal wastewater laboratory data. After the plant layout had been designed the model was validated using the PCI treatment data.

5.4.2 Model Calibration
The validation procedure is shown in Section 4.4.2, chapter 4. The average COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P concentrations in the laboratory study were utilised in these simulations. The parameter values utilised in this study are detailed in Appendix D.

The primary variations between this model and that validated in Chapter 4 are the values of $\eta_{\text{NO}_3}$ and $b_\text{A}$ parameters. These variations occurred due to the differences between real municipal wastewater and synthetic wastewater. The real municipal wastewater had higher solid concentrations and lower P concentrations. In addition, the loading rates to the IASBR varied greatly when using real wastewater. Fig. 5.5 compares the simulation phase study data with the laboratory phase study data.

(a) NH$_4^+$-N model and observed phase data comparison
Figure 5.5. Comparison of simulation and observed phase study data for NH$_4^+$-N (a), TON (b) and DO (c)

Strong linear relationships between the simulation data and the observed data were found for the NH$_4^+$-N and TON phase data, with R-values of 0.97 and 0.96, respectively. The R-value for the DO phase data was 0.86, also suggesting a strong linear relationship.
When using T-tests to check if there was significant difference between the simulation data and the observed data, the calculated P values for the NH$_4^+$-N, TON and DO phase data were 0.06, 0.65 and 0.32, respectively. The P values all exceeded 0.05, indicating that there was no significant difference between the simulation and observed data.

### 5.4.3 Sensitivity Analysis

The procedure for the sensitivity analysis has been described in Appendix C. A sensitivity analysis was performed on the influent characteristics which would experience variations through primary clarification treatment. Table 5.5 details the influent wastewater characteristics which were tested during the sensitivity analysis. The average COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P concentrations observed in the PCI treatment laboratory study were utilised as the default values in the sensitivity analysis.

Table 5.5. Sensitivity analysis parameters (Average PCI influent characteristics were used as default; NH$_4^+$-N=19.5 mg/L; PO$_4^{3-}$-P= 0.7 mg/L)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Utilised Value</th>
<th>Sensitivity Analysis Range</th>
<th>Approach Utilised for Parameter Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD/N Ratio</td>
<td>-</td>
<td>14.93</td>
<td>4.88 – 58.54</td>
<td>Influent COD increased (100 – 1,200 mg/L)</td>
</tr>
<tr>
<td>COD/P Ratio</td>
<td>-</td>
<td>65.11</td>
<td>306 – 38.25</td>
<td>Influent TP increased (1 – 8 mg/L)</td>
</tr>
<tr>
<td>COD/VSS Ratio</td>
<td>gCOD/gVSS</td>
<td>1.8</td>
<td>1 – 9</td>
<td>-</td>
</tr>
<tr>
<td>Fermentable/biodegradable fraction of total COD</td>
<td>-</td>
<td>0.2</td>
<td>0.1 – 0.5</td>
<td>-</td>
</tr>
</tbody>
</table>

The effluent characteristics reviewed were COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P. The most significant impact on the effluent characteristics was due to variations in the COD/N ratio (Fig. 5.6). No significant effects were observed in the effluent characteristics during the sensitivity analysis of the other parameters.
Figure 5.6. Sensitivity Analysis: COD/N ratio

The COD/N sensitivity analysis was performed at minimum and maximum ratios of 4.88 and 58.5, respectively. This was performed by maintaining the influent TN concentrations at 20.5 mg/L but varying the influent COD concentrations. This sensitivity analysis was performed due to the observed reduction of the COD/N ratio by primary clarification. The performance of this sensitivity analysis would help to identify at what COD/N ratio primary clarification would be required to maintain efficient treatment in the IASBR.

Significant variations were observed in the COD effluent concentrations which were increased from 7.37 mg/L to 415 mg/L as the COD/N ratio was increased from 4.88 to 58.5, respectively. COD concentrations were increased after the COD/N ratio was increased to 39. Effluent COD concentrations immediately rose by 60% and continued to increase in a linear trend.

The effluent TN concentrations experienced a significant variation as the COD/N ratio was increased. When the ratio was increased from 4.88 to 19.5 effluent TN concentrations were reduced from 14.2 mg/L to 6.39 mg/L. The concentrations were reduced due to the increased COD available for heterotrophic denitrification. After the COD/N ratio had been increased to ≥ 24.4, the effluent TN concentrations were increased from 6.39 mg/L to 41.6 mg/L. This increase was due to the inhibition of
the nitrification process by the increased COD concentrations within the reactor and reduced DO concentrations (Fig. 5.7). The nitrifier concentrations were reduced significantly from a peak concentration of 70.9 mg COD/L at a COD/N ratio of 29.27 to a minimum concentration 0 at the COD/N ratio of 39.02 as the DO concentrations were reduced and the heterotrophic biomass concentrations were increased from 391.2 mg COD/L at a COD/N ratio of 4.88 to 2,690.9 mg COD/L at a COD/N ratio of 39.02.

![COD/N ratio: DO concentrations](image)

**Figure 5.7.** Average DO cycle data at increasing COD/N ratios

The effluent NH$_4^+$-N concentrations were observed to increase significantly as the influent COD/N ratio was increased. Effluent concentrations were increased from 0.02 mg/L to 40.9 mg/L when the COD/N ratio was increased from 4.88 to 58.5. The most significant increase took place when the COD/N ratio was increased to 29.3, from 0.4 mg/L to 25 mg/L. The decline of the nitrification process was due to the increased COD concentrations within the reactor at the increased COD/N ratios, reduced DO concentrations and reduced nitrifier biomass concentrations.

The TON concentrations were reduced from 14 mg/L at a COD/N ratio of 4.88 to 0 at a COD/N ratio of 39. This reduction occurred due to the reduced nitrification activity and increased COD concentrations available for denitrification.
A gradual increase in the effluent TP and $\text{PO}_4^{3-}$-P concentrations was observed as the influent COD/N ratio was increased, from 0.92 mg/L to 6.01 mg/L and from 0.89 mg/L to 5.95 mg/L, respectively. The increased effluent P concentrations were due to increased solids concentrations within the effluent. The PAO biomass concentrations did not vary at increased COD/N ratios, which were equal to 0.

The sensitivity analysis on the COD/N ratio indicates that the IASBR could treat municipal wastewater by-passing primary clarification which had COD/N ratios less than 29.3 to within discharge standards for the COD, TN and $\text{NH}_4^+$-N concentrations. COD/N ratios of 29.3 or greater were observed to inhibit the nitrification process of the IASBR operation sequence and resulted in increased effluent TN and $\text{NH}_4^+$-N concentrations.

### 5.4.4 Simulation Tasks

After the validation had been completed the model was utilised for further understanding of the IASBR technology. Two simulation runs were performed, which are described in Table 5.6. The average COD, TN, TP, $\text{NH}_4^+$-N and $\text{PO}_4^{3-}$-P concentrations observed in the laboratory study for the PCI and PCE wastewaters were utilised in these simulations.

<table>
<thead>
<tr>
<th>Simulation Run</th>
<th>Description</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 1</td>
<td>The calibrated IASBR model was run for two separate 200-day periods for the treatment of PCE and PCI wastewater.</td>
<td>To further compare the treatment efficiency of an IASBR for PCE and PCI wastewater through simulation data of biomass and nutrients.</td>
</tr>
<tr>
<td>Run 2</td>
<td>The calibrated IASBR model was run for four separate 200-day periods for the treatment of PCE and PCI wastewater. The impact of the conversion of the influent wastewater from PCE to PCI, and vice-versa, was simulated.</td>
<td>To investigate the impact of shock loading on IASBR operation sequences treating PCI and PCE wastewater.</td>
</tr>
</tbody>
</table>

Simulated COD, TKN, $\text{NH}_4^+$-N, TON, TP and $\text{PO}_4^{3-}$-P effluent data, and phase data
in typical SBR operation cycles for NH$_4^+$-N, TON and PO$_4^{3-}$-P concentrations were obtained in addition with biomass concentrations of heterotrophs, autotrophs and PAOs.

5.4.4.1 Run 1 Simulation
The average COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P concentrations in PCI and PCE wastewaters measured in the laboratory were utilised in Run 1 simulations. Steady state operations for the treatment of PCE and PCI wastewater were achieved after a period of 40 days and 30 days, respectively. T-tests were performed on all effluent data sets and results indicated a statistically significant difference between the IASBR treatment of the PCE and PCI wastewaters, with P values equal to 0. The average effluent water quality of the IASBR technology for the treatment of the PCE and PCI wastewaters is displayed in Table 5.7.

A difference of 3% in the average TP and PO$_4^{3-}$-P concentrations for the treatment of the PCE and PCI wastewaters was observed. The COD and NH$_4^+$-N concentrations were observed to increase when the IASBR was treating PCI wastewater; however the average effluent concentrations for both PCI and PCE influent wastewater treatment were within the discharge standards. The TN and TON effluent concentrations were increased by 40% when the IASBR was treating the PCE influent wastewater.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>PCI</th>
<th>PCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>16.8</td>
<td>8</td>
</tr>
<tr>
<td>TN</td>
<td>7.4</td>
<td>11.9</td>
</tr>
<tr>
<td>NH$_4^+$-N</td>
<td>0.04</td>
<td>0.02</td>
</tr>
<tr>
<td>TON</td>
<td>7.2</td>
<td>11.8</td>
</tr>
<tr>
<td>TP</td>
<td>1.45</td>
<td>1.5</td>
</tr>
<tr>
<td>PO$_4^{3-}$-P</td>
<td>1.44</td>
<td>1.48</td>
</tr>
</tbody>
</table>

Further comparison of the IASBR operations was performed using the cycle phase data gathered from the simulations.
A similar nitrification trend was observed in the NH$_4^+$-N phase data for the treatment of the PCE and PCI wastewater (Fig. 5.8a). NH$_4^+$-N reduction occurred primarily during Aeration Periods 1 and 2, like the observed data. The TON phase data demonstrated the inhibitory effects of the PCE wastewater’s low COD/N ratio on the denitrification process (Fig. 5.8b). TON concentrations did not reduce to 0 during Non-aeration Period 1 when treating PCE. When treating the PCI wastewater, TON reduction occurred during the non-aeration periods, with concentrations reduced to 0 in Non-aeration Period 1.

Small variations were observed in the PO$_4^{3-}$-P phase data; the low P concentrations in the influent wastewaters limited the development of the EBPR process (Fig. 5.8c). A slight release of P was observed during the treatment of the PCI wastewater followed by an uptake during Aeration Period 1. P concentrations were gradually increased during the remainder of the treatment cycle.

(a) PCE and PCI NH$_4^+$-N simulation phase data
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(b) PCE and PCI TON simulation phase data

![IASBR PCE vs. PCI: TON Phase Data](image)

(c) PCE and PCI PO$_4^{3-}$-P simulation phase data

![IASBR PCE vs. PCI: PO$_4^{3-}$-P Phase Data](image)

**Figure 5.8.** NH$_4^+$-N (a), TON (b) and PO$_4^{3-}$-P (c) concentrations for one simulated cycle

The heterotrophic and autotrophic biomass concentrations in typical SBR cycles are presented in Fig. 5.9. The PAO biomass concentrations reduced to 0 during the treatment of PCE and PCI wastewaters over an 18 day period.
Lower concentrations of heterotrophic biomass were found in the IASBR during the treatment of the PCE wastewater. The average concentrations of the heterotrophic biomass in the react phase during a PCE and PCI treatment cycle were 489 mg COD/L and 1,165 mg COD/L, respectively. The elevated heterotrophic growth during the PCI treatment was due to the increased COD concentrations available for biomass growth.
The average concentrations of the autotrophic biomass in the react phase during the PCE and PCI treatment cycle were 57 mg COD/L and 65 mg COD/L, respectively. The reduced influent TKN concentrations inhibited the autotrophic biomass growth despite the reduced competition from heterotrophs. During the treatment of the PCI wastewater the increased influent TKN enhanced the autotrophic biomass growth.

5.4.4.2 Run 2 Simulations

The average COD, TN, TP, NH$_4^+$-N and PO$_4^{3-}$-P concentrations in the PCI and PCE wastewaters measured in the laboratory study were utilised in Run 2 simulations. In Run 2 the variations of the effluent quality and biomass concentrations were examined to study the shock loading impacts of changes in the influent wastewater from PCE to PCI wastewater, and vice-versa, on IASBR operations. The simulations performed are detailed in Table 5.8.

**Table 5.8 Run 2 simulation descriptions**

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Description</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>The IASBR treated PCE wastewater until 10 days after steady state was achieved (50 days). Influent wastewater was then converted to PCI.</td>
<td>To investigate the immediate impact on treatment efficiency of influent conversion from a low to a high COD/N.</td>
</tr>
<tr>
<td>B</td>
<td>Simulation B utilised the same procedure as Simulation A; however the influent order was reversed. PCI wastewater was treated until 10 days after steady state was achieved (40 days) and the influent wastewater was then converted to PCE.</td>
<td>To investigate the immediate impact on treatment efficiency of influent conversion from a high to a low COD/N.</td>
</tr>
<tr>
<td>C</td>
<td>During Simulation C the IASBR treated PCE wastewater. Scenarios were investigated while the feed was converted to PCI wastewater before steady state had been achieved.</td>
<td>To investigate the impact on treatment efficiency of influent conversion during unsteady state operations from a low to a high COD/N.</td>
</tr>
<tr>
<td>D</td>
<td>Simulation D utilised the same procedure as Simulation C; however the influent order was reversed. The conversion to PCE wastewater before steady state had been achieved.</td>
<td>To investigate the impact on treatment efficiency of influent conversion during unsteady state operations from a high to a low COD/N.</td>
</tr>
</tbody>
</table>
5.4.4.2.1 Simulation A Results

The IASBR operation was utilised for the treatment of PCE wastewater over a period of 50 days in order to achieve steady state operations. Then the influent wastewater was converted to PCI wastewater in order to observe the shock loading effects. A significant variation was observed in the COD, TN, TON, TP and \( \text{PO}_4^{3-} \)-P concentrations immediately after the influent conversion (Fig. 5.10). A slight variation was observed in the \( \text{NH}_4^+ \)-N concentrations following the influent conversion (Fig. 5.10).

(a) Simulation A PCE and PCI COD effluent concentrations

(b) Simulation A PCE and PCI TN effluent concentrations
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(c) Simulation A PCE and PCI $\text{NH}_4^+$-N effluent concentrations

(d) Simulation A PCE and PCI TON effluent concentrations
There was a significant increase in the effluent COD concentrations after the influent conversion. Average concentrations were increased from 8.02 mg/L to 16.8 mg/L, after steady state operations had been achieved. After one day of treating the PCI wastewater effluent concentrations had been increased from 8.47 mg/L to 15.3 mg/L. Steady COD removal was achieved 2 days after the influent conversion.
Rapid reductions in both the effluent TN and TON concentrations were found within one day after the influent was switched from PCE to PCI. During the steady state for PCE wastewater treatment the TN and TON effluent concentrations were 12 mg/L and 11.9 mg/L, respectively. The TN and TON effluent concentrations were then reduced to 9.4 mg/L and 9.17 mg/L, respectively. Steady TN and TON concentrations were achieved 6 days after the influent had been converted to the PCI wastewater and were reduced to 7.57 mg/L and 7.31 mg/L, respectively.

A slight reduction in the \( \text{NH}_4^+ - \text{N} \) concentrations was noticed immediately after the influent conversion had occurred which may be due to increased biomass synthesis. This slight reduction was followed by a gradual increase in \( \text{NH}_4^+ - \text{N} \) concentrations over a 9 day period as steady nitrification was achieved from 0.021 mg/L to 0.05 mg/L. The slight increase in effluent \( \text{NH}_4^+ - \text{N} \) concentrations was due to increased influent \( \text{NH}_4^+ - \text{N} \) concentrations and the inhibition of the nitrification process by the increased influent COD concentrations.

The TP concentrations were reduced from 1.47 mg/L to 0.992 mg/L and the \( \text{PO}_4^{3-} - \text{P} \) concentrations were reduced from 1.46 mg/L to 0.969 mg/L within one day of the influent conversion to PCI wastewater. These reductions were due to the increased P consumption for biomass synthesis and the slight reduction observed in the effluent solid concentrations. However, there was a gradual increase in the effluent concentrations because of the increased influent TP concentrations. Steady effluent P concentrations were achieved 16 days after the influent conversion and the average effluent TP and \( \text{PO}_4^{3-} - \text{P} \) concentrations were 1.45 mg/L and 1.44 mg/L, respectively.

Slightly increased heterotrophic biomass concentrations were observed immediately after the influent was shifted from PCE to PCI from 500.3 mg COD/L to 513.8 mg COD/L. The heterotrophic biomass concentration after steady state had been achieved was 529.3 mg COD/L (Fig. 5.11). This increase was due to the increased influent COD concentrations available for heterotrophic growth.
A slight increase in autotrophic biomass concentrations was observed immediately after the influent conversion occurred, from 59.6 mg COD/L to 61.3 mg COD/L. The steady state autotrophic biomass concentrations were 66.1 mg COD/L (Fig. 5.12).

No significant negative impacts were observed when converting the influent
wastewater from PCE to PCI. COD and NH$_4^+$-N concentrations were increased slightly. The denitrification process was enhanced, with increased heterotrophic and autotrophic growth.

5.4.4.2.2 Simulation B Results

The IASBR operation was utilised for the treatment of PCI wastewater over a period of 40 days in order to achieve steady state operation. Then, the influent was converted to PCE wastewater. A significant variation was observed in the effluent COD, TN, TP, TON and PO$_4^{3-}$-P concentrations immediately after the influent conversion (Fig. 5.13). A slight variation was observed in the NH$_4^+$-N concentrations following the influent conversion (Fig. 5.10).

![Run 2 PCI to PCE: COD](image)

(a) Simulation B PCE and PCI COD effluent concentrations
(b) Simulation B PCE and PCI TN effluent concentrations

(c) Simulation B PCE and PCI NH₄⁺-N effluent concentrations
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(d) Simulation B PCE and PCI TON effluent concentrations

(e) Simulation B PCE and PCI TP effluent concentrations
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(f) Simulation B PCE and PCI $\text{PO}_4^{3-} - \text{P}$ effluent concentrations

**Figure 5.13.** COD (a), TN (b), $\text{NH}_4^+ - \text{N}$ (c), TON (d), TP (e) and $\text{PO}_4^{3-} - \text{P}$ (f) concentrations

An immediate reduction in the effluent COD concentrations was observed after the influent conversion. Effluent COD concentrations continued to reduce until stable COD concentrations were achieved after 3 days. Average COD concentrations before and after the influent conversion were 16.8 mg/L and 8.02 mg/L, respectively. The low influent COD concentrations in PCE were the cause of the effluent COD reduction.

The TN and TON effluent concentrations immediately increased after the influent conversion. The TN concentrations were increased from 7.4 mg/L to 10.4 mg/L, and the TON concentrations were increased from 7.2 mg/L to 10.2 mg/L, after one day of treating PCE wastewater. The TN and TON concentrations continued to increase until 10 days and 8 days, respectively, had passed.

After the influent conversion occurred, average steady state $\text{NH}_4^+ - \text{N}$ concentration before and after the influent conversion were 0.04 mg/L and 0.02 mg/L, respectively. Steady state concentrations were achieved after a period of 9 days.

A significant increase was observed in the effluent TP and $\text{PO}_4^{3-} - \text{P}$ concentrations
immediately after the influent conversion to PCE wastewater. TP concentrations were increased from 1.43 mg/L to 1.96 mg/L and PO\(_4^{3-}\)-P concentrations were increased from 1.41 mg/L to 1.95 mg/L after one day of influent wastewater conversion. Effluent TP concentrations peaked at 2.07 mg/L two days after the conversion, and then began to reduce and became stable after 12 days. Effluent PO\(_4^{3-}\)-P concentrations peaked at 2.03 mg/L two days after the conversion, and then began to reduce and became stable after 10 days. The increased influent PO\(_4^{3-}\)-P concentrations and reduced influent COD concentrations available for biomass growth resulted in the increased effluent P concentrations in the beginning.

A decline in heterotrophic biomass concentrations was found during the treatment of the PCE wastewater. The decline occurred immediately after the influent conversion (Fig. 5.14). One day after the influent conversion the heterotrophic biomass concentrations were reduced from 1,198.4 mg COD/L to 1,118.8 mg COD/L. The steady state heterotrophic biomass concentrations reduced from 1,174.4 mg COD/L to 482.1 mg COD/L after the conversion. The reduced growth was due to the reduced COD concentrations available for heterotrophic biomass growth.

![Figure 5.14. Heterotrophic biomass concentrations before and after influent conversion over a one day period](image)

The decline in the autotrophic biomass concentrations occurred over an extended
period of time and no significant variation was observed immediately after the influent conversion (Fig. 5.15). Average stable autotroph biomass concentrations were reduced from 66.3 mg COD/L to 56.5 mg COD/L after the conversion of the influent from PCI to PCE wastewater.

![Run 2 PCI to PCE: Autotrophic Phase Data](image)

**Figure 5.15.** Autotrophic biomass concentrations before and after influent conversion over a one day period

### 5.4.4.2.3 Simulation C Results

This study was divided into three sections in order to observe the impacts of converting the influent from PCE to PCI wastewater prior to the achievement of steady state. The ‘unsteady’ states of the heterotrophs, autotrophs and PAOs were investigated and influent conversion was performed 7 days prior to the achievement of biomass steady state.

Three 200 day simulations were performed with the influent conversion occurring in the unsteady states of the heterotrophs (Simulation C1), autotrophs (Simulation C2) and PAOs (Simulation C3), whose biomass concentrations became stable 15 days, 30 days and 11 days after the PCE wastewater treatment commenced, respectively. Variations were observed in the COD, TN, TP, NH$_4^+$-N, TON and PO$_4^{3-}$-P effluent concentrations immediately after the influent conversion to PCI wastewater (Fig. 5.16)
Chapter 5 Impact of by-passing Primary Clarification on IASBR Treatment

(a) Simulation C PCE and PCI COD effluent concentrations

(b) Simulation C PCE and PCI TN effluent concentrations
Chapter 5 Impact of by-passing Primary Clarification on IASBR Treatment

(c) Simulation C PCE and PCI $\text{NH}_4^+$ effluent concentrations

(d) Simulation C PCE and PCI TON effluent concentrations
Identical trends were observed in the COD concentrations after the influent conversion had occurred in Simulations C1, C2 and C3. During Simulations C1, C2 and C3 concentrations were increased from 7.62 mg/L to 14.4 mg/L, 7.66 mg/L to 14.6 mg/L and 7.72 mg/L to 14.6 mg/L, respectively, after one day of treatment of the PCI wastewater. This indicated that the length of the period prior to the influent conversion had a slight effect on effluent COD concentrations.
The effluent TN and TON concentrations had similar patterns during Simulations C1, C2 and C3; slight variations between the simulations occurred. An immediate reduction in TN concentrations was observed in Simulation C1, C2 and C3. The TN concentrations during Simulations C1, C2 and C3 were reduced from 11.9 mg/L to 9.4 mg/L, 12 mg/L to 9.42 mg/L and 12 mg/L to 9.39 mg/L, respectively, after one day of treating the PCI wastewater. The TON concentrations were reduced from 11.7 mg/L to 9.19 mg/L, from 11.8 mg/L to 9.21 mg/L, and from 11.9 mg/L to 9.17 mg/L, respectively, after one day of PCI treatment in Simulations C1, C2 and C3, respectively. TN and TON concentrations were stable after a 9 days and 8 days, respectively.

A rapid increase was observed in the NH$_4^+$-N concentrations during Simulations C1, C2 and C3; slight variations between the simulations occurred until steady state was achieved. After one day of treating the PCI wastewater the effluent concentrations were increased from 0.018 mg/L to 0.019 mg/L, from 0.019 mg/L to 0.02 mg/L, and from 0.021 mg/L to 0.022 mg/L in Simulations C1, C2 and C3, respectively. After 14 days in each simulation effluent NH$_4^+$-N concentrations became stable and were identical at 0.044 mg/L.

Rapid reductions in the TP and PO$_4^{3-}$-P concentrations were observed in Simulations C1, C2 and C3 following the influent conversion. The TP concentrations were reduced from 1.5 mg/L to 1.01 mg/L, from 1.48 mg/L to 0.997 mg/L, and from 1.47 mg/L to 0.992 mg/L during Simulations C1, C2 and C3, respectively, within one day of the influent conversion. The PO$_4^{3-}$-P concentrations were reduced from 1.48 mg/L to 0.98 mg/L, from 1.47 mg/L to 0.97 mg/L, and from 1.46 mg/L to 0.97 mg/L after one day of treating the PCE wastewater in Simulations C1, C2 and C3, respectively. TP and PO$_4^{3-}$-P concentrations were stable after 18 days and 16 days after the influent conversion.

A similar heterotrophic growth trend was observed in Simulations C1, C2 and C3 (Fig. 5.17). Increased heterotrophic concentrations were observed in simulations C1 and C3 as the influent conversion occurred at an earlier period. The difference in heterotrophic biomass concentrations among the three simulations was very slight. The heterotrophic growth was increased rapidly after the influent conversion.
occurred as was observed in Simulation A.

**Figure 5.17.** Heterotrophic biomass concentrations before and after influent conversion over a one day period

A similar trend was observed in the autotrophic biomass concentrations with an average difference of 7.9% observed in the Simulation C2 and C3 autotrophic biomass concentrations (Fig. 5.18).

**Figure 5.18.** Autotrophic biomass concentrations before and after influent conversion over a one day period
5.4.4.2.4 Simulation D Results
This study was divided into three sections in order to assess the impact of converting the influent wastewater from PCI to PCE wastewater prior to the biomasses achieving steady state. The ‘unsteady’ states of the heterotrophs, autotrophs and PAOs were investigated. Influent conversion occurred 7 days prior to the achievement of biomass steady state.

Three 200-day simulations were performed with the influent conversion occurring in the unsteady states of the heterotrophs (Simulation D1), autotrophs (Simulation D2) and PAOs (Simulation D3) in 6 days, 18 days and 4 days after the PCI wastewater treatment commenced, respectively. Variations were observed in the COD, TN, TP, NH$_4^+$-N, TON and PO$_4^{3-}$-P effluent concentrations immediately after the influent conversion to PCE wastewater (Fig. 5.19)

(a) Simulation D PCE and PCI COD effluent concentrations
(b) Simulation D PCE and PCI TN effluent concentrations

(c) Simulation D PCE and PCI NH$_4^+$-N effluent concentrations
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(d) Simulation D PCE and PCI TON effluent concentrations

(e) Simulation D PCE and PCI TP effluent concentrations
Similar trends were observed in the COD concentrations after the influent conversion had occurred in Simulations D1, D2 and D3. An immediate COD reduction occurred in each simulation. During Simulations D1, D2 and D3 concentrations were decreased from 17 mg/L to 10.4 mg/L, from 16.6 mg/L to 10.3 mg/L, and from 16.4 mg/L to 10.2 mg/L, respectively, after one day of PCE wastewater treatment. Effluent COD concentrations were stable after 9 days.

An immediate increase in the effluent TN and TON concentrations was observed in Simulations D1, D2 and D3. During Simulations D1, D2 and D3 concentrations were increased from 7.76 mg/L to 10.4 mg/L, from 7.47 mg/L to 10.4 mg/L, and from 7.65 mg/L to 10.4 mg/L, respectively, after one day of treating the PCE wastewater. The TON concentrations were increased from 7.53 mg/L to 10.3 mg/L, from 7.23 mg/L to 10.2 mg/L, and from 7.42 mg/L to 10.2 mg/L, respectively, after one day of PCE treatment in Simulations D1, D2 and D3, respectively. Effluent TN and TON concentrations were stable after 13 days and 11 days, respectively.

Immediately after the influent conversion the NH$_4^-$.N concentrations were increased; however the increments were reduced as the time before the conversion was earlier.

(f) Simulation D PCE and PCI PO$_4^{3-}$-P effluent concentrations

**Figure 5.19.** COD (a), TN (b), NH$_4^+$-N (c), TON (d), TP (e) and PO$_4^{3-}$-P (f) concentrations
The slight increase was immediately followed by a gradual reduction in NH$_3^+$-N concentrations. After five days of treating the PCE wastewater the effluent concentrations were reduced from 0.0275 mg/L to 0.025 mg/L, from 0.0432 mg/L to 0.0268 mg/L, and from 0.0241 mg/L to 0.0217 mg/L in Simulations D1, D2 and D3, respectively. Effluent NH$_4^+$-N concentrations were stable after 26 days in each simulation.

The TP concentrations were increased from 1.24 mg/L to 1.32 mg/L, from 1.44 mg/L to 1.95 mg/L, and from 1.16 mg/L to 1.77 mg/L during Simulations D1, D2 and D3, respectively, within one day of the influent conversion. The PO$_4^{3-}$-P concentrations were increased from 1.22 mg/L to 1.31 mg/L, from 1.41 mg/L to 1.94 mg/L, and from 1.13 mg/L to 1.76 mg/L after one day of treating the PCE wastewater in Simulations D1, D2 and D3, respectively. Effluent TP and PO$_4^{3-}$-P operations were stable 18 days and 14 days after the influent conversion, respectively.

The heterotrophic biomass concentrations were reduced immediately after the influent conversion (Fig. 5.20). An average difference of 15% was observed in the Simulation D2 and D3 heterotrophic biomass concentrations.

![Run 2 PCI to PCE Unsteady State: Heterotrophic Phase Data](image)

**Figure 5.20.** Autotrophic biomass concentrations before and after influent conversion over a one day period.
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There was an average difference of 29% in the simulation D2 and D3 autotrophic concentrations (Fig. 5.21).

![Run 3 PCI to PCE Unsteady State: Autotrophic Phase Data](image)

**Figure 5.21.** Autotrophic biomass concentrations before and after influent conversion over a one day period

### 5.5 Summary

The effects of by-passing primary clarification on the efficiency of an IASBR treating real municipal wastewater were examined by using laboratory and simulation studies. The laboratory studies found that treating wastewater which had by-passed the primary clarification improved the performance of the IASBR, in particular in N removal. This study shows that by-passing the primary clarifiers can result in enhanced nutrient treatment by the IASBR.

However loading variations may exceed the plant’s treatment capacity without primary clarifier treatment. Therefore a bypassing system should be utilised. The simulation data indicated that influent converting from the low to high COD/N ratios resulted in a rapid increase in IASBR treatment efficiency.
Chapter 5
Impact of by-passing Primary Clarification on IASBR Treatment
Chapter Six

Mathematical Modelling of Slaughterhouse Wastewater Treatment using the IASBR and cSBR systems

6.1 Introduction

The performance data of a pilot-scale SBR treating slaughterhouse wastewater with a continuous aeration pattern and an intermittent aeration pattern (Norton, 2009) was utilised to validate a GPS-X mathematical model. The original pilot-scale study consisted of 8 testing phases and Phases 5 and 6 were utilised for validation to compare cSBR and IASBR processes in treating slaughterhouse wastewater.

6.2 Materials

6.2.1 Pilot-scale Unit and Operation

The pilot-scale unit is described in the Appendix E. It was set up in Western Proteins, Ballyhaunis, Co. Mayo, Ireland. The operation sequences of the continuous aeration cSBR (i.e. cSBR) and the intermittent aeration pattern (i.e. IASBR) are detailed in Fig. 6.1, with a SBR cycle duration of 8 hrs. Both operations had three cycles per day. In each IASBR cycle, there were four alternating non-aeration and aeration periods; the four non-aeration periods and the first three aeration periods were 50 minutes long each and the final aeration period was 30-minutes long. In each cSBR cycle, there was a 10 minute non-aeration period followed by a 370-minute aeration period. Under both operation sequences a mixed fill occurred at the start of the react phase and lasted for 5-minutes. The settle, decant and idle periods occurred sequentially after the react period for both operation sequences and were 90, 5 and 5 minutes long, respectively.
6.2.2 Slaughterhouse Wastewater and Seed Sludge

The wastewater used in this study was taken from a forward feed pipe from the balance tank to the first aeration tank at the Western Proteins’ WWTP in Ballyhaunis, Co. Mayo. During Phase 5 the average influent organic loading rate was 510 mg COD/L/d, and the wastewater contained average TSS, COD, TN, NH$_4^+$-N and PO$_4^{3-}$-P concentrations of 1887 mg SS/L, 4,674 mg/L, 414 mg/L, 408 mg/L and 33 mg/L, respectively. During Phase 6 the influent organic loading rate was 490 mg COD/L/d, and the wastewater contained average TSS, COD, TN, NH$_4^+$-N and PO$_4^{3-}$-P concentrations of 1407 mg SS/L, 4,380 mg /L, 381 mg/L, 465 mg/L and 29 mg/L, respectively.

6.3 Pilot-Scale Study Results

The results obtained during the operation of the pilot-scale study are presented in Appendix E.

6.4 Model Development and Calibration

Modelling and parameter simulations were performed using the GPS-X 6.1.1 software. The ASM1 model was utilised in this study due to the lack of P influent and effluent data available from the pilot study. The model was validated using the pilot unit cSBR and IASBR data. Simulations were run for a 200 day period.

6.4.1 Simulation Objects

The Object layout utilised in Section 4.4.1, chapter 4, was utilised in this study. The physical parameters of the equalisation tank and the Manual SBR unit were modified.
to simulate the pilot-scale system and the influent characteristics were the average slaughterhouse wastewater concentrations observed during Phases 5 and 6. The Excel spread sheet control for this model utilised the same layout as the model developed in Section 4.4.1, chapter 4. The pilot study utilised an aeration rate of 80 L air/min which was activated for one third of every minute during the aeration periods; therefore an average aeration rate of 28.34 L air/min was utilised during the modelling.

### 6.4.2 Model Validation

Simulated effluent data were compared to the observed effluent concentrations of the cSBR and IASBR operations in order to validate the model. The validation procedure utilised for this study is shown in Section 4.4.2, chapter 4. The values of the major parameters used to calibrate the model are shown in Appendix E. The simulated data was compared with the observed effluent quality data to identify the accuracy of the model (Table 6.1).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Phase 5</th>
<th></th>
<th></th>
<th>Phase 6</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pilot-scale</td>
<td>Simulation</td>
<td>% Difference</td>
<td>Pilot-scale</td>
<td>Simulation</td>
<td>% Difference</td>
</tr>
<tr>
<td>COD</td>
<td>109</td>
<td>120</td>
<td>9.2</td>
<td>117</td>
<td>114</td>
<td>2.6</td>
</tr>
<tr>
<td>TN</td>
<td>96</td>
<td>98</td>
<td>2</td>
<td>8</td>
<td>8.1</td>
<td>1.2</td>
</tr>
<tr>
<td>NH₄⁺-N</td>
<td>1.4</td>
<td>0.2</td>
<td>86</td>
<td>0.5</td>
<td>0.2</td>
<td>60</td>
</tr>
</tbody>
</table>

The validated model was accurate for the COD and TN effluent concentrations; however large variations were observed in the NH₄⁺-N effluent concentrations. The variations in the NH₄⁺-N concentrations occurred due to the variations in the influent wastewater quality and site temperature. The observed NH₄⁺-N concentrations in the pilot study were within the discharge standards of 10 mg/L.

### 6.4.3 Simulation Data

After the validation of the mathematical model further study and comparisons of the cSBR and IASBR systems were performed. The average COD, TN and NH₄⁺-N concentrations in the influent wastewater observed in the pilot study were utilised for
these simulations. The cSBR and IASBR operation layouts were utilised in three simulation runs which are presented in Table 6.3.

**Table 6.3. Simulation run descriptions**

<table>
<thead>
<tr>
<th>Simulation Run</th>
<th>Description</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 1</td>
<td>The calibrated model for the cSBR and IASBR was run for a 200 day period for the treatment of Phase 5 slaughterhouse wastewaters with a review of cycle phase data and effluent water quality data, and autotrophic and heterotrophic biomass.</td>
<td>This run allowed for comparison of the cSBR and IASBR operation sequences.</td>
</tr>
<tr>
<td>Run 2</td>
<td>The calibrated model was run for a 200 day period for the treatment of Phase 5 slaughterhouse wastewaters. The cSBR operation was altered so that those in the total aeration and non-aeration periods were equal to the IASBR operation sequence.</td>
<td>This run allowed for further comparison between the cSBR and IASBR operation sequences.</td>
</tr>
<tr>
<td>Run 3</td>
<td>The calibrated model was run for 200 days for treatment of Phase 5 wastewater with various HRT.</td>
<td>This run examined the flexibility of the IASBR operation sequences.</td>
</tr>
</tbody>
</table>

Simulated COD, TKN, NH$_4^+$-N and TON effluent data, and phase data in typical SBR operation cycles for the DO, NH$_4^+$-N, and TON parameters were obtained, in addition to biomass concentrations of heterotrophs and autotrophs.

**6.4.3.1 Run 1 Simulation**

The average COD, TN and NH$_4^+$-N concentrations in the influent wastewater measured in the pilot study treating the Phase 5 wastewater were utilised in Run 1 simulations. The SBR operation sequences are described in Fig. 6.1. The average effluent water characteristics obtained in the simulation for both operation sequences are shown in Table 6.4.
### Table 6.4. Simulated cSBR and IASBR effluent characteristics when treating Phase 5 wastewater (mg/L)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>IASBR</th>
<th>cSBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>121</td>
<td>120</td>
</tr>
<tr>
<td>TN</td>
<td>9.2</td>
<td>98.2</td>
</tr>
<tr>
<td>TKN</td>
<td>1.1</td>
<td>1.3</td>
</tr>
<tr>
<td>NH₄⁺-N</td>
<td>0.18</td>
<td>0.18</td>
</tr>
<tr>
<td>TON</td>
<td>8.1</td>
<td>96.8</td>
</tr>
</tbody>
</table>

Steady state operation was achieved after 70 days of the cSBR operation and 45 days after the conversion to the IASBR operation. No significant variation was observed in the effluent COD concentrations (Fig. 6.2a); however large variations were observed in the effluent TN and TON concentrations.

The effluent TKN concentrations were reduced after the conversion from a cSBR to an IASBR operation (Fig. 6.2b). However, no significant variations were observed in the NH₄⁺-N effluent concentrations (Fig. 6.2c). The most significant variations were observed in the effluent TN and TON concentrations. The average TN concentrations were reduced from 98.2 mg/L to 9.2 mg/L after the operation conversion (Fig. 6.2d) and the TON concentrations were reduced from 96.8 mg/L to 8.1 mg/L (Fig. 6.2e). The short non-aeration period in the cSBR operation, affecting the denitrification process, caused high effluent TN and TON concentrations.
Chapter 6 Mathematical Modelling of Slaughterhouse Wastewater Treatment

(a) Run 1 cSBR and IASBR COD Effluent Concentrations

(b) Run 1 cSBR and IASBR TKN Effluent Concentrations

(c) Run 1 cSBR and IASBR NH$_4^+$-N Effluent Concentrations
Further comparisons of the operation sequences were performed using the NH$_4^+$-N and TON cycle phase data gathered from the simulations. A significant difference in the nitrification trend was observed in the NH$_4^+$-N phase data for the cSBR and the IASBR operations (Fig. 6.3). In the cSBR NH$_4^+$-N reductions occurred primarily in the first three hours of the react phase and in the IASBR the NH$_4^+$-N reduction was observed primarily during Aeration Periods 1, 2 and 3.
TON production and reduction were different for the cSBR and IASBR operations (Fig. 6.4). In the cSBR, TON production was observed during the Aeration Period. In the IASBR, TON production occurred primarily in Aeration Periods 1, 2 and 3. Denitrification took place during Non-aeration Periods 2, 3 and 4 which reduced the TON concentrations in the IASBR.

The average stable concentration of the cSBR heterotrophic biomass was 1,467.9 mg
COD/L (Fig. 6.5). Little heterotrophic growth was observed in the Non-aeration Period in the cSBR operation. The average stable concentration of the IASBR heterotrophic biomass was 963 mg COD/L.

![Run 1 Phase 5 Heterotrophic Phase Data](image)

**Figure 6.5.** cSBR and IASBR heterotrophic biomass concentration phase data

Difference in the growth and decay of autotrophic biomass was observed in the autotrophic biomass concentrations in the cSBR and IASBR (Fig. 6.6). Autotrophic biomass grew primarily during the first three hours of the Aeration Period in the cSBR, which was followed by a slight decline due to endogenous respiration. Peak autotrophic biomass concentrations in the cSBR were 96.7 mg COD/L. At the end of the operation cycle, the autotrophic biomass concentrations were 96.4 mg COD/L. The average autotrophic biomass concentration in the cSBR was 96.11 mg COD/L.

Autotrophic biomass growth was observed primarily during Aeration Periods 1, 2 and 3 in the IASBR with a slight decline in Aeration Period 4. The average autotrophic biomass concentration was 83 mg COD/L.
The data obtained during Run 1 matched the pilot study findings and indicated that the extended non-aeration period of the IASBR significantly enhanced denitrification.

6.4.3.2 Run 2 Simulation

The average COD, TN and NH$_4^+$-N concentrations in the influent measured in the pilot study for Phases 5 and 6 were utilised in Run 2 simulation. During Run 2 simulation, an amended cSBR operation sequence was hypothesized. The amended operation had total aeration and non-aeration durations equalling those utilised in the IASBR operation sequence. The amended cSBR operation sequence is presented in Fig. 6.7.
The average effluent water quality obtained in the simulations for the IASBR and amended cSBR operation sequences are shown in Table 6.5.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>IASBR</th>
<th>Amended cSBR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Phase 6</td>
<td>Phase 5</td>
</tr>
<tr>
<td>COD</td>
<td>114</td>
<td>121</td>
</tr>
<tr>
<td>TN</td>
<td>8.1</td>
<td>13</td>
</tr>
<tr>
<td>TKN</td>
<td>1.2</td>
<td>2.2</td>
</tr>
<tr>
<td>(\text{NH}_4^+)-N</td>
<td>0.15</td>
<td>1.4</td>
</tr>
<tr>
<td>TON</td>
<td>6.9</td>
<td>10.8</td>
</tr>
</tbody>
</table>

Steady state operation was achieved after 80 days of the amended cSBR operation. Large differences were observed in the effluent TN, TON, TKN and \(\text{NH}_4^+\)-N concentrations under the amended cSBR operation in comparison to Run 1 cSBR operation.

No difference was observed in the effluent COD concentrations after conversion from the amended cSBR to the IASBR (Fig. 6.8a). The amended cSBR operations significantly affected the effluent TN and TON concentrations (Fig. 6.8 b and c, respectively). The effluent TN concentrations were reduced by 86.7% and 88% during the treatment of the Phase 5 and 6 wastewaters, respectively, in comparison to the Run 1 cSBR. A similar trend was observed in the effluent TON concentrations which were reduced by 88.8% and 90.3% during the treatment of the Phase 5 and 6 wastewaters in comparison to the Run 1 cSBR.

The effluent TKN concentrations were increased in the amended cSBR operation during the treatment of Phases 5 and 6 wastewaters, respectively, in comparison to the Run 1 cSBR. The increased TKN concentrations in the amended cSBR effluent were due to the decline in the nitrification process (Fig. 6.8d).

The \(\text{NH}_4^+\)-N effluent concentrations were increased during the treatment of Phases 5 and 6 wastewaters in comparison to the Run 1 cSBR. The decline in the nitrification process of the amended cSBR was due to the reduced aeration period (Fig. 6.8e).
(a) Run 2 cSBR and IASBR COD effluent concentrations

(b) Run 2 cSBR and IASBR TN effluent concentrations
(c) Run 2 cSBR and IASBR TKN effluent concentrations

(d) Run 2 cSBR and IASBR TON effluent concentrations
Further comparison of the operation sequences of the amended cSBR and the IASBR was performed using the NH$_4^+$-N and TON cycle phase data gathered from the simulations.

There was a significant difference in the NH$_4^+$-N phase data in the amended cSBR during the treatment of Phases 5 and Phase 6 wastewaters in comparison to the Run 1 cSBR operations. In the amended cSBR the nitrification process continued throughout the entire Aeration Period. The reduced aeration duration inhibited nitrification and reduced the NH$_4^+$-N removal efficiency in comparison to the Run 1 cSBR data. In the IASBR the NH$_4^+$-N reduction occurred primarily during Aeration Periods 1, 2 and 3 (Fig. 6.9).

Figure 6.8. Effluent COD (a), TN (b), TKN (c), TON (d) and NH$_4^+$-N (e) concentrations
The TON production and reduction were significantly different for the amended cSBR during the treatment of the Phase 5 and Phase 6 wastewaters in comparison to the Run 1 cSBR operation data. In the amended cSBR, TON was produced during the aeration period with similar concentrations in comparison to the Run 1 cSBR; however due to the extended non-aeration period enhanced denitrification was observed. The extended non-aeration period reduced the TON concentrations at the
beginning of the react phase. In the IASBR, TON production occurred primarily in Aeration Periods 1, 2 and 3. Denitrification was observed during Non-aeration Periods 2, 3 and 4 which reduced the effluent TON concentrations. In addition, denitrification in the IASBR occurred at the beginning of the react phase (Fig. 6.10).

**Figure 6.10.** Amended cSBR and IASBR TON phase data during the treatment of Phase 5 (a) and Phase 6 (b) wastewaters

The growth and decay trend of heterotrophic biomass was similar for the treatment
of Phase 5 and Phase 6 wastewaters (Fig. 6.11). Anoxic heterotrophic growth was observed in the amended cSBR with peak heterotrophic biomass concentrations of 652.3 mg COD/L and 570.7 mg COD/L during the treatment of Phase 5 and Phase 6 wastewaters, respectively. The heterotrophic biomass concentrations declined after these peak concentrations during the remainder of the non-aeration period. Aerobic heterotrophic biomass grew during the aeration period with peak heterotrophic biomass concentrations of 634 mg COD/L and 554.3 mg COD/L during the treatment of Phases 5 and 6 wastewaters. The average stable heterotrophic biomass concentrations in the amended cSBR operation for treatment of Phases 5 and 6 wastewaters were 633.2 mg COD/L and 553.3 mg COD/L, respectively. There was a lower heterotrophic biomass concentration in the amended cSBR in comparison to that in the Run 1 cSBR. The heterotrophic biomass concentrations in the IASBR were much higher than in the amended cSBR during the treatment of Phase 5 and Phase 6 wastewaters.

(a) Phase 5 cSBR and IASBR Heterotrophic Biomass phase data
(b) Phase 6 cSBR and IASBR Heterotrophic Biomass phase data

**Figure 6.11.** Amended cSBR and IASBR heterotrophic biomass phase data during the treatment of Phase 5 (a) and Phase 6 (b) wastewaters

In the amended cSBR a slight reduction in autotrophic biomass concentrations was observed during the non-aeration period (Fig. 6.12) and concentrations at the beginning of the aeration period were 77.2 mg COD/L and 64.2 mg COD/L when treating Phase 5 and Phase 6 wastewaters, respectively. Autotrophic biomass gradually grew during the aeration period with peak concentrations of 79.2 mg COD/L and 65.9 mg COD/L for treatment of Phase 5 and Phase 6 wastewaters, respectively. The average autotrophic biomass concentrations were 78.2 mg COD/L and 65 mg COD/L, respectively. The autotroph concentrations were higher in the IASBR during the treatment of Phase 5 and Phase 6 wastewaters.
Run 2 Phase 5 Autotrophic Phase Data

(a) Phase 5 cSBR and IASBR Autotrophic Biomass phase data

Run 2 Phase 6 Autotrophic Phase Data

(b) Phase 6 cSBR and IASBR Autotrophic Biomass phase data

Figure 6.12. Amended cSBR and IASBR heterotrophic biomass phase data during the treatment of Phase 5 (a) and Phase 6 (b) wastewaters

Run 2 simulation indicates that extending the non-aeration period of the cSBR would significantly enhance denitrification and would reduce the effluent TON concentrations. The IASBR operation had slightly better effluent quality than the amended cSBR operation with higher heterotrophic and autotrophic biomass concentrations.
6.4.3.3 Run 3 Simulation

The average COD, TN and NH$_4^+$-N concentrations in Phase 5 wastewater measured in the pilot study were utilised in Run 3 simulation. During Run 3 the flexibility of the IASBR and amended cSBR operations over a range of HRT values (Table 6.6) was compared.

**Table 6.6. HRT value range**

<table>
<thead>
<tr>
<th>HRT (days)</th>
<th>Influent Flow Rate (L/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.6</td>
<td>1.52</td>
</tr>
<tr>
<td>5.2</td>
<td>1.92</td>
</tr>
<tr>
<td>4.3</td>
<td>2.32</td>
</tr>
<tr>
<td>3.7</td>
<td>2.72</td>
</tr>
<tr>
<td>3.2</td>
<td>3.12</td>
</tr>
<tr>
<td>2.8</td>
<td>3.52</td>
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</table>

When HRT was reduced, there were no significant variations in the effluent COD concentrations; however the effluent N concentrations increased in the amended cSBR and IASBR operation sequences. The variation of the effluent characteristics in the amended cSBR and IASBR operations is shown in Fig. 6.13.

(a) cSBR HRT variation effluent nutrient concentrations
The effluent COD concentrations did not vary in the cSBR and IASBR effluent as the influent loading increased. In both the amended cSBR and IASBR the maximum and minimum values of COD were 121 mg/L and 122 mg/L, respectively.

The NH$_4^+$-N effluent concentrations were increased as the HRT was reduced in the IASBR and amended cSBR because the increased loading rate inhibited the nitrification process. In the IASBR the NH$_4^+$-N concentrations were increased from 0 to 185 mg/L and in the amended cSBR the NH$_4^+$-N concentrations were increased from 0 to 188 mg/L, respectively. The obvious decline in the nitrification process was found after the HRT had been reduced from 3.8 days to 3.2 days. In the amended cSBR the TKN concentrations were increased from 0.75 mg/L to 188 mg/L, and in the IASBR the TKN concentrations were increased from 0.77 mg/L to 185 mg/L, respectively.

The effluent TN and TON concentrations in the IASBR and amended cSBR experienced a significant variation as the HRT was reduced. In the IASBR the TN concentrations were increased from 9.9 mg/L to 189 mg/L and in the amended cSBR the TN concentrations were increased from 14.3 mg/L to 197 mg/L when the HRT
was reduced from 6.6 days to 2.8 days. When the HRT was reduced from 6.6 days to 3.7 days TN concentrations were increased due to the increased TON production at higher NH$_4$$^+$-N influent loadings; after the HRT was reduced to 3.2 days and 2.8 days the increased effluent TN concentrations were due to a significant decline in the nitrification process which resulted in rising NH$_4$$^+$-N concentrations. When the HRT was reduced to 3.2 days the effluent TON concentrations were increased slightly from 14.8 mg/L to 15 mg/L in the IASBR and from 23.3 mg/L to 25 mg/L in the amended cSBR. However, the nitrification process declined significantly after the HRT was reduced to 2.8 days. In the IASBR the TON concentrations were reduced to 6.6 mg/L and in the amended cSBR the TON concentrations were reduced to 8.5 mg/L.

These results indicated that the operation sequences of the IASBR and amended cSBR used in this study were insufficient to handle a very low HRT. Further comparison of the operations was performed using the cycle phase data gathered from the simulations.

In the IASBR and the amended cSBR the nitrification trends were similar at the reduced HRT (Fig. 6.14); when the HRT was reduced to 3.2 days nitrification was seriously inhibited.
The TON concentrations changed significantly at various HRT (Fig. 6.15). In the IASBR, TON production occurred primarily in Aeration Periods 1, 2 and 3. Denitrification took place during Non-aeration Periods 2, 3 and 4 which reduced the effluent TON concentrations. In the amended cSBR, TON concentrations were increased during the aeration period and denitrification took place during the non-aeration period. The TON productions were increased in the IASBR and amended cSBR as the HRT was reduced to 3.2 days due to the increased NH$_4^+$-N loading rate; however due to the decline of the nitrification process when the HRT was reduced to 2.8 days the effluent TON concentrations were reduced significantly.
In the amended cSBR the heterotrophic biomass concentrations were increased significantly as the HRT was reduced due to the increased nutrients available for growth (Fig. 6.16). The heterotrophic biomass concentrations were increased from 745.7 mg COD/L to 1,194.7 mg COD/L. In the IASBR the heterotrophic biomass concentrations were increased from 1,088.7 mg COD/L to 1,576.5 mg COD/L as the HRT was reduced to 3.2 days. As the HRT was reduced to 2.8 days the reduced DO
concentrations in the reactors and declining treatment efficiency reduced the heterotrophic biomass concentrations to 1,480.3 mg COD/L.

**Figure 6.16.** Heterotrophic biomass concentrations in typical operation cycles in amended cSBR (a) and IASBR (b)

A significant variation was observed in the growth and decay of autotrophic biomass in the SBR cycles (Fig. 6.17). In the amended cSBR the autotrophic biomass concentrations were increased from 97 mg COD/L to 166.7 mg COD/L as the HRT
was reduced to 3.2 days; however after the HRT was reduced to 2.8 days the autotrophic biomass concentrations were reduced to 65.1 mg COD/L. In the IASBR the autotrophic biomass concentrations were increased from 100.7 mg COD/L to 172.1 mg COD/L as the HRT was reduced to 3.7 days; however as the HRT was reduced below 3.7 days the autotrophic biomass concentrations were reduced to 59.4 mg COD/L. The reduction in the autotrophic biomass concentrations was due to the reduced DO concentrations in the reactors.

Figure 6.17. Autotrophic biomass concentrations in typical cycles in amended cSBR

(a) cSBR HRT variation Autotrophic Biomass phase data

(b) IASBR HRT variation Autotrophic Biomass phase data
In order to overcome the poor nitrification efficiency at low HRTs, an amended IASBR layout and a corresponding cSBR layout, presented in Fig. 6.18, were assessed for the treatment of the slaughterhouse wastewater at a HRT of 2.8 days.

Figure 6.18. Proposed IASBR and cSBR operation sequences

The average effluent water characteristics obtained in the simulations for the proposed IASBR and cSBR operation sequences are shown in Table 6.8.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>IASBR</th>
<th>cSBR</th>
</tr>
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<tbody>
<tr>
<td>COD</td>
<td>121</td>
<td>121</td>
</tr>
<tr>
<td>TN</td>
<td>21.9</td>
<td>27.8</td>
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<tr>
<td>TON</td>
<td>21</td>
<td>26.6</td>
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<tr>
<td>NH$_4^+$-N</td>
<td>0.22</td>
<td>0.12</td>
</tr>
<tr>
<td>TKN</td>
<td>0.9</td>
<td>1.27</td>
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</table>

No variation was observed in the effluent COD concentrations in the proposed IASBR and cSBR operations in comparison to the original operation sequences (Fig. 6.19a). The proposed operations significantly affected the effluent N concentrations. The extended aeration period in the proposed operations enhanced the nitrification process and prevented the increase of effluent NH$_4^+$-N concentrations (Fig. 6.19e). In the proposed IASBR the effluent NH$_4^+$-N concentrations were reduced in comparison to the original operation from 185 mg/L to 0.2 mg/L. A similar trend was observed in the corresponding cSBR operation with a reduction in NH$_4^+$-N
concentrations from 188 mg/L to 0.15 mg/L. These reductions occurred due to the extended aeration periods, which enhanced nitrification.

Therefore, the TKN concentrations were reduced significantly (Fig. 6.19c). In the amended IASBR the effluent TKN concentrations were reduced in comparison to the original operation from 185 mg/L to 0.9 mg/L. A similar trend was observed in the corresponding cSBR operation with a reduction in TKN concentrations from 188 mg/L to 0.9 mg/L. TON concentrations were increased from 3.9 mg/L to 20.8 mg/L and from 8.5 mg/L to 28.6 mg/L in the proposed IASBR and cSBR operations (Fig. 6.19d). The increased effluent TON concentrations were mainly due to the increased nitrification rate.

The effluent TN concentrations were reduced from 189 mg/L to 21.7 mg/L for the proposed IASBR operation. In the corresponding cSBR operations the TN concentrations were reduced from 197 mg/L to 29.5 mg/L (Fig. 6.19b).

(a) Run 3 Amended cSBR and IASBR Operations COD effluent concentrations
(b) Run 3 Amended cSBR and IASBR Operations TN effluent concentrations

(c) Run 3 Amended cSBR and IASBR Operations TKN effluent concentrations
(d) Run 3 Amended cSBR and IASBR Operations TON effluent concentrations

(e) Run 3 Amended cSBR and IASBR Operations NH$_4^+$-N effluent concentrations

Figure 6.19. Effluent COD (a), TN (b), TKN (c), TON (d) and NH$_4^+$-N (e) concentrations

In the proposed IASBR operation the NH$_4^+$-N reduction was observed during Aeration Periods 1, 2 and 3, and nitrification was completed after an average of 45 minutes in the final aeration period (Fig. 6.20). The remaining time in Aeration Period 3 indicated that it was still feasible to increase the IASBRs operation performance. In the corresponding cSBR operation nitrification was completed after an average of 210 minutes in the aeration period.
Figure 6.20. NH$_4^+$-N in typical cycles for proposed cSBR and IASBR operations

Correspondingly, in the IASBR the TON production was observed during Aeration Periods 1, 2 and 3 (Fig. 6.21), and denitrification was observed during Non-aeration Periods 2 and 3. The denitrification reduced the effluent TON concentrations.

Figure 6.21. TON in typical cycles for proposed cSBR and IASBR operation sequences

A significant difference was observed in the heterotrophic biomass growth and decay in the proposed IASBR and cSBR operations (Fig. 6.22). In the proposed IASBR
operation the average heterotrophic biomass concentrations were 2,692 mg COD/L; and in the corresponding cSBR operation the average heterotrophic biomass concentrations were 2,408 mg COD/L.

**Figure 6.22.** Heterotrophic biomass concentrations in typical cycles for proposed IASBR and cSBR operations

In the proposed IASBR the average autotrophic biomass concentrations were 219.8 mg COD/L (Fig. 6.23), higher than that of the original operation due to the extended aeration periods. In the corresponding cSBR operation the average react phase autotrophic biomass concentrations were 212 mg COD/L.
### 6.5 Summary

This study examined and compared the efficiency of cSBR and IASBR operations for the treatment of slaughterhouse wastewater. The simulation found that the IASBR operations had much better denitrification efficiency due to the extended non-aeration periods. Amending the cSBR operation to increase the non-aeration period significantly reduced the effluent TN and TON concentrations.

The IASBR operation treatment efficiency declined as the HRT was reduced. However, amending the operation sequences prevented a decline in the nitrification and denitrification processes. This study clearly shows that the IASBR operations allowed for efficient treatment of slaughterhouse wastewater under various conditions due to the operation sequence flexibility. The simulation allowed for various operation sequences to be tested in order to achieve enhanced treatment of high strength wastewater.
Chapter Seven

Conclusions and Recommendations

7.1 Overview
During this study cSBR and IASBR operation sequences were used to treat various types of wastewater: (1) synthetic domestic wastewater with COD/N ratios of 10.5; (2) real municipal wastewater which had received primary clarification treatment with COD/N ratios of 6.5; (3) real municipal wastewater which had by-passed primary clarification treatment and had COD/N ratios of 14.9; and (4) real slaughterhouse wastewater. The primary objective of this study was to compare IASBRs with cSBRs with laboratory-scale experiment and mathematical modelling. Conclusions obtained with recommendations made are presented in the following sections.

7.2 Conclusions

7.2.1 Synthetic Domestic Wastewater Treatment
The performance of the IASBR and cSBR operation sequences regarding COD, N and P removals was studied and compared in a bench-scale study. Further comparisons were performed using mathematical modelling to study nutrient removals and heterotrophic, autotrophic and PAOs biomass concentrations under various scenarios. It was found that the IASBR operation sequence had better denitrification and EBPR efficiencies than cSBR operation. The results obtained are as follows:

1. At the aeration rate of 0.8 L air/min, removals of COD, TN and TP were up to 84%, 78% and 90%, and 86%, 62% and 89% in the IASBR and cSBR, respectively.
2. An increased aeration rate of 1 L air/min improved nitrification. Denitrification and P removal declined due to the increased DO concentrations in the react period. Removals of COD and TN were up to 90% and 70%, and 92% and 59% in the IASBR and cSBR, respectively.
3. The IASBR operation was considered better due to its enhanced denitrification and EBPR processes.

4. The mathematical modelling shows that the IASBR would achieve better performance than the cSBR. TN and TP removal efficiencies of 90.4% and 77.7%, and 60.3% and 22.7% were achieved in the IASBR and cSBR, respectively.

5. Simulated data demonstrates that heterotrophic biomass concentrations were 14% higher in the cSBR than in the IASBR, and the PAOs concentration was 71% higher in the IASBR.

7.2.2 By-passing Primary Clarification Treatment
In this study the performance of an IASBR for the treatment of PCE and PCI wastewater was compared to study the necessity of primary clarification for municipal wastewater treatment in an Irish context. Further comparison was performed using mathematical modelling to study the nutrient removal and heterotrophic, autotrophic and PAOs biomass concentrations. The results obtained are as follows:

1. By-passing primary clarification and utilising the IASBR technology to treat PCI wastewater improved the nitrogen removal efficiency. Removals of COD, TN, TP, and NH$_4^+$-N of 85%, 52%, 85% and 98%, respectively, were achieved.

2. The PCE wastewater significantly reduced the heterotrophic and autotrophic biomass concentrations within the IASBR. Heterotrophic and autotrophic biomass concentrations during the treatment of PCE wastewater were reduced by 58% and 12%, respectively, in comparison to the treatment of PCI wastewater.

3. The influent conversion from PCE to PCI after steady state operations resulted in immediate reduction of the TN, TON, TP and PO$_4^{3-}$-P concentrations in the effluent by 22%, 23%, 33% and 33%, respectively. Increased heterotrophic and autotrophic biomass concentrations were also observed.

4. The influent conversion from PCI to PCE after steady state operations resulted in the increase in the TN, TON, TP and PO$_4^{3-}$-P concentrations by
28.7%, 29.5%, 27% and 28%, respectively. The heterotrophic and autotrophic biomass concentrations were reduced in the reactor.

5. The length of the treatment period prior to influent wastewater variation was found to affect the shock load impacts; however effluent concentrations at the steady state were identical.

6. By-passing primary clarification can enhance the IASBR treatment efficiency. However loading variations can exceed the treatment capabilities of the secondary treatment unit, and therefore a by-passing system should be considered rather than omission of primary clarification.

### 7.2.3 Slaughterhouse Wastewater Treatment

In this study a mathematical model was developed to compare the performance of a cSBR and an IASBR for the treatment of slaughterhouse wastewater using the data gathered in the pilot-scale study of Norton (2009). The results obtained are as follows:

1. The operation conversion from the cSBR to the IASBR resulted in increased denitrification. Effluent TN concentrations were reduced by 90.6%.

2. Using an amended cSBR operation with an extended non-aeration period and a reduced aeration period the TN and TON concentrations in the effluent were reduced by 87% and 89%, respectively, in comparison to the pilot-scale study cSBR operation.

3. Decreasing the HRT negatively affected the IASBR and amended cSBR operations. Nitrification rates declined and there was a significant increase in the effluent TN concentrations. Altering the IASBR and cSBR operation sequences can improve the nitrification efficiency due to the flexibility of SBRs.

4. Use of the mathematical model allowed for testing and comparison of various operation sequences.

### 7.2.4 Summary

The findings of this PhD research indicate that N and P removal from wastewater treatment can be more efficiently achieved utilising the IASBR technology rather than the cSBR technology, and the IASBR system can be adapted to various conditions. The cSBR and IASBR systems were compared using laboratory-scale
studies and GPS-X models which were successfully developed for cSBR and IASBR systems treating various wastewaters. The laboratory-scale experimental data and mathematical modelling results indicate that IASBR technology can achieve enhanced denitrification and EBPR processes. The IASBRs ability to promote the nitrification, denitrification and EBPR processes can be utilised by WWTPs in order to reduce treatment complexity, space requirements and operation costs, and to increase operational flexibility. However, great care must be taken when choosing a proper aeration rate in order to prevent the negative impacts of excessive DO concentrations on the IASBR treatment performance. This study also shows that it is not necessary to have primary clarification for municipal wastewater treatment when utilising the IASBR technology. By-passing primary clarification can reduce a WWTP’s capital and operating costs and space requirements which allow for easier expansion of WWTPs. Due to the system flexibility, the IASBR technology can be adapted to maintain discharge standards during HRT and influent loading rate variations. An IASBR can be more easily altered to effectively treat high and low strength wastewaters. WWTPs utilising IASBRs could adapt the operations to suit the changes in wastewater strength which may change due seasonal weather variations or increased influent flows.

7.3 Recommendations

1. A step-feed method was proposed to improve the treatment of IASBRs for low COD/N ratio wastewaters. The regular introduction of COD into the reactors following the aeration periods will reduce carbon consumption in the aeration periods and provide more organic carbon for anoxic heterotrophic denitrification.

2. Weather variations as a result of climate change may affect the average influent volume treated and influent wastewater quality. Therefore research on effects of influent variations on IASBR efficiency needs to be studied so as to provide countermeasures to the climate change issue.

3. Analysis of nitrous oxide (N\textsubscript{2}O), nitric oxide (NO) and methane (CH\textsubscript{4}) emissions from the conventional SBR and the IASBR wastewater treatment systems should be carried out by measuring N\textsubscript{2}O, NO and CH\textsubscript{4} concentrations in the air phase and dissolved N\textsubscript{2}O and NO concentrations in
the liquid phase. N$_2$O, NO and CH$_4$ concentrations in the air phase can be collected using gas chromatography and dissolved N$_2$O and NO concentrations in the liquid phase can be measured using micro-sensors.

4. The EBPR process should be included in the mathematical model to simulate slaughterhouse wastewater treatment.
Bibliography


47. Fan, L., Xie, Y. (2011) ‘Optimization Control of SBR Wastewater Treatment Process Based on Pattern Recognition’ Procedia Environmental Sciences, 10, 20 – 25
slaughterhouse wastewater in a sequencing batch reactor: ASM2d modelling and verification’ Environmental Technology, 26, 1081 – 1100


70. Hatton, T.C. (1916a) ‘Activated Sludge experiments at Milwaukee, Wisconsin’ Engineering News 74, 134-137
141. MWE (2006a) ‘Technical Guidelines for the Use of Treated Sanitary Wastewater in Irrigation for Landscaping and Agricultural Irrigation’ Ministry of Water and Electricity, Kingdom of Saudi Arabia


characteristics of membrane sequencing batch reactors used for the treatment of synthetic petroleum refinery wastewater’ Bioresource Technology, 102, 7692 – 7699


Appendices

Appendix A: Publications and Presentations during PhD Research

Appendix B: Analytical Methods

The analysis methods and procedures utilised for testing the various contaminants were performed in the Environmental Engineering Laboratories, NUI Galway, and are presented below:

B1 – Chemical oxygen demand (COD)
COD was tested using the closed reflux method and a Hach DR/2010 portable data logging spectrophotometer in accordance with the standard APHA methods (APHA, 1995). Results are presented in units of mg COD/L.

B2 – 5-day biochemical oxygen demand (BOD₅)
BOD₅ was tested in accordance with the standard APHA methods (APHA, 1995) utilising a Lovibond BOD-System Oxidirect (Lovibond, UK). Results are presented in units of mg BOD₅/L.

B3 – Total Nitrogen (TN)
TN was measured utilising a TN kit and a Hach DR/2010 portable data logging spectrophotometer in accordance to the manufacturers protocol (Hach, USA). Results are presented in units of mg/L.

B4 – Ammonium-nitrogen (NH₄⁺-N)
NH₄⁺-N was tested utilising a Thermo Clinical Labsystem – Konelab 20 Nutrient Analyser (Thermo, USA) in accordance with standard APHA methods (APHA, 1995). Results are presented in units of mg/L.

B5 – Nitrite-nitrogen (NO₂⁻-N)
NO₂⁻-N was tested utilising a Thermo Clinical Labsystem – Konelab 20 Nutrient Analyser (Thermo, USA) in accordance with standard APHA methods (APHA, 1995). Results are presented in units of mg NO₂⁻-N/L.

B6 – Total oxidised nitrogen (TON)
TON was tested utilising a Thermo Clinical Labsystem – Konelab 20 Nutrient Analyser (Thermo, USA) in accordance with standard APHA methods (APHA, 1995). Results are presented in units of mg/L.
B7 – Total Phosphorus (TP)
TP was measured utilising a TP kit and a Hach DR/2010 portable data logging spectrophotometer in accordance to the manufacturers protocol (Hach, USA). Results are presented in units of mg/L.

B8 – Ortho-phosphate (PO$_4^{3-}$-P)
PO$_4^{3-}$-P was tested utilising a Thermo Clinical Labsystem – Konelab 20 Nutrient Analyser (Thermo, USA) in accordance with standard APHA methods (APHA, 1995). Results are presented in units of mg/L.

B9 – Dissolved oxygen (DO)
DO was measured utilising a HI-9828 multi-parameter electrode (Hanna Instruments, United Kingdom). Results are presented in units of mg DO/L.

B10 – pH
pH was measured utilising a HI-9828 multi-parameter electrode (Hanna Instruments, United Kingdom).

B11 – Suspended Solids (SS)
SS was tested in accordance with the standard APHA methods (APHA, 1995) utilising a vacuum pump and a 1.2 μm pore size GC/F filter papers (Whatman, UK). Results are presented in units of mg SS/L.

B12 – Sludge Volume Index (SVI)
SVI was tested in accordance with the standard APHA methods (APHA, 1995) utilising a vacuum pump and a 1.2 μm pore size GC/F filter papers (Whatman, UK) to measure the MLSS concentrations and a 1 litre graduated cylinder to measure the volume of settled sludge after a 30 minute settling period. Results are presented in units of mL/g.
Appendix C: Comparison of conventional SBR and IASBR operations for the treatment of synthetic domestic wastewater

Values of major parameters used in the calibration procedure of Chapter 4 Model

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<th>Chosen Value</th>
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<td>0</td>
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<td><strong>Kinetic Parameters</strong></td>
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storage \((K_{\text{per}})\) & gCOD/gCOD & 0.01 & 0.01 & - \\
PHA half saturation coefficient \((K_{\text{PHA}})\) & mgCOD/L & 0.2 & 0.2 & - \\
**General Half-Saturation Coefficients**

Oxygen half saturation coefficient \((K_{\text{O}_2})\) & mgO\(_2\)/L & 0.5 & 0.5 & - \\
Nitrate half saturation coefficient \((K_{\text{NO}_3})\) & mgN/L & 0.05 & 0.05 & - \\
Ammonium (as nutrient) half saturation coefficient \((K_{\text{NH}_4})\) & mgN/L & 0.01 & 0.01 & - \\
Phosphate (as nutrient) half saturation coefficient \((K_p)\) & mgP/L & 5 & 5 & - \\
Alkalinity half saturation coefficient \((K_{\text{ALK}})\) & mgCaCO\(_3\)/L & 25 & 25 & - \\

**Autotrophic Organisms \((X_{\text{AUT}})\)**

Autotrophic maximum specific growth rate \((\mu_{\text{A}})\) & d\(^{-1}\) & 1 & 1.2 & Pai et al. (2009) \\
Autotrophic Decay Rate \((b_{\text{A}})\) & d\(^{-1}\) & 0.15 & 0.08 & Jiang et al. (2008) \\
Oxygen half saturation coefficient for autotrophs growth \((K_{\text{O}_2})\) & mgO\(_2\)/L & 0.5 & 0.5 & - \\
Ammonium (as substrate) half saturation coefficient for autotrophs growth \((K_{\text{NH}_4})\) & mgN/L & 1 & 1 & - \\
Alkalinity half saturation coefficient for autotrophic growth \((K_{\text{ALK}})\) & mgCaCO\(_3\)/L & 25 & 25 & - \\

**Hydrolysis**

Hydrolysis rate \((K_h)\) & d\(^{-1}\) & 3 & 3 & - \\
Anoxic hydrolysis reduction factor \((\eta_{\text{NO}_3})\) & - & 0.6 & 0.6 & - \\
Anaerobic hydrolysis reduction factor \((\eta_{\text{fe}})\) & - & 0.4 & 0.4 & - \\
Slowly biodegradable substrate half saturation coefficient for hydrolysis \((K_X)\) & - & 0.1 & 0.1 & - \\

**Fermentation**

Fermentation maximum rate \((\mu_{\text{fe}})\) & d\(^{-1}\) & 3 & 3 & - \\
Fermentable substrate half saturation coefficient \((K_{\text{fe}})\) & mgCOD/L & 4 & 4 & - \\

**Precipitation**

P precipitation with metal hydroxides rate constant \((k_{\text{PRE}})\) & m\(^3\)/(g.min) & 0.0006944444 & 0.0006944444 & - \\
Redissolution of phosphates rate constant \((k_{\text{RED}})\) & d\(^{-1}\) & 0.6 & 0.6 & - \\
Alkalinity half saturation coefficient for phosphates redissolution \((K_{\text{ALK}})\) & mgCaCO\(_3\)/L & 25 & 25 & - \\

Data is stored on CD.
Appendix D: Efficiency of IASBRs treating municipal wastewater receiving and by-passing primary clarification

Values of major parameters used in the calibration procedure of Chapter 5 Model

<table>
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<tr>
<th>Parameters (symbol)</th>
<th>Units</th>
<th>Default Value</th>
<th>Chosen Value</th>
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<td>Heterotrophic Biomass ((X_H))</td>
<td>gCOD/gCOD</td>
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<td>0.63</td>
<td>Plazl et al. (1999)</td>
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<td>Heterotrophic yield ((Y_H))</td>
<td>gCOD/gCOD</td>
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<td>Poly-P accumulating biomass yield ((Y_{PAO}))</td>
<td>gCOD/gCOD</td>
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<td>0.4</td>
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<tr>
<td>PHA requirement for poly-P storage ((Y_{PHA}))</td>
<td>gCOD/gP</td>
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<td>Maximum ratio of poly-P in poly-P accumulating biomass</td>
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<td><strong>Phosphorus-accumulating Organisms ((X_{PAO}))</strong></td>
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<tr>
<td>Poly-P requirement per PHA stored ((Y_{PAO}))</td>
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<td>Rate constant for storage of PHA ((q_{PHA}))</td>
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<td>Rate constant for storage of poly-P ((q_{PP}))</td>
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<td>Poly-P accumulating biomass lysis rate ((b_{PAO}))</td>
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<td>Poly-P lysis rate ((b_{PP}))</td>
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<tr>
<td>PHA lysis rate ((b_{PHA}))</td>
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<tr>
<td>Reduction factor for anoxic activity ((\eta_{NO3}))</td>
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<tr>
<td>Volatile fatty acids half saturation coefficient ((K_A))</td>
<td>mgCOD/L</td>
<td>4</td>
<td>4</td>
<td>-</td>
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<tr>
<td>Volatile fatty acids half saturation coefficient ((K_P))</td>
<td>mgP/L</td>
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<tr>
<td>P half saturation coefficient for storage of poly-P ((K_P))</td>
<td>gCOD/gCOD</td>
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<tr>
<td><strong>Hydrolysis</strong></td>
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<td>Fraction of inert COD generated in biomass lysis ((f_{X1}))</td>
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<tr>
<td>Production of soluble inerts in hydrolysis ((f_{SI}))</td>
<td>gCOD/gCOD</td>
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<td><strong>Kinetic Parameters</strong></td>
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<td>Sin and Vanrolleghem (2006)</td>
</tr>
<tr>
<td>Heterotrophic Organisms ((X_H))</td>
<td>d⁻¹</td>
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<tr>
<td>Lysis and decay rate constant ((b_{H}))</td>
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<td>Denitification reduction factor ((\eta_{NO3}))</td>
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<td>0.8</td>
<td>0.4</td>
<td>Sin and Vanrolleghem (2006)</td>
</tr>
<tr>
<td>Fermentable substrate half saturation coefficient for heterotrophic growth ((K_F))</td>
<td>mgCOD/L</td>
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<tr>
<td>Volatile fatty acids half saturation coefficient ((K_A))</td>
<td>mgCOD/L</td>
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<td>Unit</td>
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<td>Value 2</td>
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<td>Inhibition coefficient for poly-P storage ((K_{IPP}))</td>
<td>gP/gCOD</td>
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<td>0.02</td>
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<tr>
<td>PHA half saturation coefficient ((K_{PHA}))</td>
<td>gCOD/gCOD</td>
<td>0.01</td>
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<td><strong>General Half-Saturation Coefficients</strong></td>
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<td>Oxygen half saturation coefficient ((K_{O2}))</td>
<td>mgO₂/L</td>
<td>0.2</td>
<td>0.2</td>
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<tr>
<td>Nitrate half saturation coefficient ((K_{NO3}))</td>
<td>mgN/L</td>
<td>0.5</td>
<td>0.5</td>
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<tr>
<td>Ammonium (as nutrient) half saturation coefficient ((K_{NH4}))</td>
<td>mgN/L</td>
<td>0.05</td>
<td>0.05</td>
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<td>Phosphate (as nutrient) half saturation coefficient ((K_{P}))</td>
<td>mgP/L</td>
<td>0.01</td>
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<tr>
<td>Alkalinity half saturation coefficient ((K_{ALK}))</td>
<td>mgCaCO₃/L</td>
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<td><strong>Autotrophic Organisms ((X_{AUT}))</strong></td>
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<td></td>
<td>Pai et al. (2009)</td>
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<tr>
<td>Autotrophic maximum specific growth rate ((\mu_A))</td>
<td>d⁻¹</td>
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<td>Autotrophic Decay Rate ((b_A))</td>
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<td>Jiang et al. (2008), Plazl et al. (1999)</td>
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<td>Oxygen half saturation coefficient for autotrophs growth ((K_{O2}))</td>
<td>mgO₂/L</td>
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<td>Ammonium (as substrate) half saturation coefficient for autotrophs growth ((K_{NH4}))</td>
<td>mgN/L</td>
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<td>Alkalinity half saturation coefficient for autotrophic growth ((K_{ALK}))</td>
<td>mgCaCO₃/L</td>
<td>25</td>
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<td><strong>Hydrolysis</strong></td>
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<td>Hydrolysis rate ((K_h))</td>
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<td>Anaerobic hydrolysis reduction factor ((\eta_{NO3}))</td>
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<td>0.6</td>
<td>0.6</td>
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<td>Anaerobic hydrolysis reduction factor ((\eta_{fe}))</td>
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<td>0.4</td>
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</tr>
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<td>Slowly biodegradable substrate half saturation coefficient for hydrolysis ((K_X))</td>
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<td>0.1</td>
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<td><strong>Fermentation</strong></td>
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<tr>
<td>Fermentation maximum rate ((\mu_{fe}))</td>
<td>d⁻¹</td>
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<tr>
<td>Fermentable substrate half saturation coefficient ((K_{fe}))</td>
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<td><strong>Precipitation</strong></td>
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<td>P precipitation with metal hydroxides rate constant ((k_{PRE}))</td>
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<tr>
<td>Alkalinity half saturation coefficient for phosphates redissolution ((K_{ALK}))</td>
<td>mgCaCO₃/L</td>
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Data is stored on CD.
Appendix E: Mathematical Modelling of Slaughterhouse Wastewater Treatment using the IASBR and cSBR systems

Values of major parameters used in the calibration procedure of Chapter 6 Model

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<th>Parameters (symbol)</th>
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<td><strong>General Fractions</strong></td>
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<td>Fractions of biomass loading to particulate products</td>
<td>gCOD/gCOD</td>
<td>0.08</td>
<td>0.08</td>
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<tr>
<td><strong>Heterotrophic Biomass (X_H)</strong></td>
<td>gCOD/gCOD</td>
<td>0.666</td>
<td>0.67</td>
<td>Norton (2009)</td>
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<tr>
<td>Heterotrophic yield (Y_H)</td>
<td>gCOD/gCOD</td>
<td>0.67</td>
<td>0.67</td>
<td>Norton (2009)</td>
</tr>
<tr>
<td><strong>Autotrophic Biomass (X_AUT)</strong></td>
<td>gCOD/gN</td>
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<td>0.24</td>
<td>-</td>
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<tr>
<td>Autotrophic yield (Y_A)</td>
<td>gCOD/gN</td>
<td>0.24</td>
<td>0.24</td>
<td>-</td>
</tr>
<tr>
<td><strong>Kinetic Parameters</strong></td>
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<tr>
<td>Heterotrophic maximum specific growth rate (µ_H)</td>
<td>d^{-1}</td>
<td>6</td>
<td>10</td>
<td>Mulas (2005)</td>
</tr>
<tr>
<td>Readily biodegradable substrate half saturation coefficient (K_s)</td>
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<td>20</td>
<td>20</td>
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<td>Oxygen half saturation coefficient (K_O)</td>
<td>mgO_2/L</td>
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<td>Nitrate half saturation coefficient (K_NO3)</td>
<td>mgN/L</td>
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<td>Anoxic growth factor (η_g)</td>
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<td>Heterotrophic decay rate</td>
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<td>Boulkroune et al. (2009)</td>
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<td>Autotrophic decay rate (b_A)</td>
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<td>Mulas (2005)</td>
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<td>Oxygen half saturation coefficient for autotrophs growth (K_O2)</td>
<td>mgO_2/L</td>
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<td>0.4</td>
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<td><strong>Hydrolysis</strong></td>
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<td>Maximum specific hydrolysis rate (K_h)</td>
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<td>Del Pozo et al. (2003)</td>
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<td><strong>Ammonification</strong></td>
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<tr>
<td>Ammonification rate (k_a)</td>
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<td>0.08</td>
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</tbody>
</table>

Data is stored on CD.
Appendix F: Activated Sludge Model equations and PLC Ladder Logic

ASM1 Processes and Components

The ASM1 model utilises eight processes and thirteen components for the AS simulation. The processes included in the ASM1 model are:

- **Aerobic growth of heterotrophs**: this process is the growth of heterotrophic biomass, when oxygen is present in the treatment system. It is the main contributor to the production of new biomass and the removal of COD. The kinetics of this process is represented by:

  \[
  \frac{dX_{B,H}}{dt} = \bar{u}_H \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{S_O}{K_{O,H} + S_O} \right) X_{B,H}
  \]  

  (F.1)

  The rate equations include a number of kinetic parameters and all kinetic parameters in ASM1 are explained in Table F.1. Five components are considered in this process: \( S_S, X_{B,H}, S_O, S_{NH}, \) and \( S_{ALK} \). These and all other components in ASM1 are shown in Table 2.4. The parameter \( Y_H \), heterotrophic yield, is utilised to describe the \( S_S \) reduction kinetics by Equation F.2.

  \[
  \frac{dS_S}{dt} = -\frac{1}{Y_H} \frac{dX_{B,H}}{dt}
  \]  

  (F.2)

- **Anoxic growth of heterotrophs**: This process is the growth of heterotrophic biomass under anoxic conditions where nitrate is used as the terminal electron acceptor, leading to denitrification. The kinetic equation for this process is:

  \[
  \frac{dX_{B,H}}{dt} = \bar{u}_H \left( \frac{S_S}{K_S + S_S} \right) \left( \frac{S_O}{K_{O,H} + S_O} \right) \left( \frac{S_{NO}}{K_{NO} + S_{NO}} \right) \eta_0 X_{B,H}
  \]  

  (F.3)

  Denitrification allows for the release of alkalinity. The variation of the alkalinity can be expressed as Equation F.4.

  \[
  \frac{dS_{ALK}}{dt} = \left( \frac{1 - Y_H}{14 \cdot 2.86Y_H} - \frac{i_{XB}}{14} \right) \frac{dX_{B,H}}{dt}
  \]  

  (F.4)

- **Aerobic growth of autotrophs**: This process is the growth of autotrophic biomass under aerobic conditions. This process results in nitrification within the model. The kinetic equation for this process is:

  \[
  \frac{dX_{B,A}}{dt} = \bar{u}_A \left( \frac{S_{NH}}{K_{NH} + S_{NH}} \right) \left( \frac{S_O}{K_{O,A} + S_O} \right) X_{B,A}
  \]  

  (F.5)
The kinetics of the $S_{NH}$ reduction can be expressed by Equation F.6 by using $Y_A$, the yield coefficient for autotrophic biomass.

$$\frac{dS_{NH}}{dt} = \left(-i_{XB} - \frac{1}{Y_A}\right) \frac{dX_{B,A}}{dt} \tag{F.6}$$

The nitrification process reduces alkalinity and its variation can be expressed as:

$$\frac{dS_{ALK}}{dt} = \left(-i_{XB} \frac{1}{14} - \frac{1}{7Y_A}\right) \frac{dX_{B,A}}{dt} \tag{F.7}$$

- Decay of heterotrophs: This process is the decay of heterotrophic biomass under aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

$$\frac{dX_{B,H}}{dt} = -b_{H}X_{B,H} \tag{F.8}$$

- Decay of autotrophs: This process is the decay of autotrophic biomass during the aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

$$\frac{dX_{B,A}}{dt} = -b_{A}X_{B,A} \tag{F.9}$$

- Ammonification of soluble organic nitrogen: This process converts biodegradable soluble organic nitrogen ($S_{ND}$) to ammonia ($S_{NH}$). The kinetic equation of this process is:

$$\frac{dS_{NH}}{dt} = k_{SP}S_{ND}X_{B,H} \tag{F.10}$$

- Hydrolysis of entrapped organics: This process controls how readily biodegradable substrate is produced when the slowly biodegradable substrate contained in the sludge mass is broken down extracellularly. The readily biodegradable substrate becomes available to the organisms for growth. The kinetic equation of this process is:

$$\frac{dS_{S}}{dt} = k_{h} \frac{X_{S}/X_{B,H}}{K_{X} + (X_{S}/X_{B,H})} \left\{ \frac{S_o}{K_{O,H} + S_o} + \eta_h \frac{K_{O,H}}{K_{O,H} + S_o} \left( \frac{S_{ND}}{K_{NO} + S_{NO}} \right) \right\} X_{B,H} \tag{F.11}$$

$S_{S}$ is produced in this process and $X_{S}$ is consumed.

- Hydrolysis of entrapped organic nitrogen: This process is the breakdown of biodegradable particulate organic nitrogen to soluble organic nitrogen. The kinetics of this process is shown in the equation:

$$\frac{dS_{ND}}{dt} = \rho_f (X_{ND}/X_{S}) \tag{F.12}$$
ρ7 represents the rate equation of the hydrolysis of entrapped organics process.

Table F.1. ASM1 kinetic parameters (Scientific and Technical Report No. 9, 2000)

<table>
<thead>
<tr>
<th>Parameter Symbol</th>
<th>Kinetic parameter description</th>
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<tr>
<td>û_H</td>
<td>Maximum specific growth rate for heterotrophic biomass</td>
</tr>
<tr>
<td>û_A</td>
<td>Maximum specific growth rate for autotrophic biomass</td>
</tr>
<tr>
<td>K_s</td>
<td>Half-saturation coefficient for heterotrophic biomass</td>
</tr>
<tr>
<td>K_NO</td>
<td>Nitrate half-saturation coefficient for denitrifying heterotrophic biomass</td>
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<tr>
<td>K_NH</td>
<td>Ammonia half-saturation coefficient for autotrophic biomass</td>
</tr>
<tr>
<td>K_O,A</td>
<td>Oxygen half-saturation coefficient for autotrophic biomass</td>
</tr>
<tr>
<td>K_O,H</td>
<td>Oxygen half-saturation coefficient for heterotrophic biomass</td>
</tr>
<tr>
<td>K_X</td>
<td>Half-saturation coefficient for hydrolysis of slowly biodegradable substrate</td>
</tr>
<tr>
<td>η_f</td>
<td>Correction factor for û_H under anoxic conditions</td>
</tr>
<tr>
<td>η_h</td>
<td>Correction factor for hydrolysis under anoxic conditions</td>
</tr>
<tr>
<td>k_s</td>
<td>Ammonification rate</td>
</tr>
<tr>
<td>b_s</td>
<td>Maximum specific hydrolysis rate</td>
</tr>
<tr>
<td>b_H</td>
<td>Decay coefficient for heterotrophic biomass</td>
</tr>
<tr>
<td>b_A</td>
<td>Decay coefficient for autotrophic biomass</td>
</tr>
</tbody>
</table>

ASM2 Processes and Components

The ASM2 model utilises 19 processes and 20 components for simulations. The processes included in the ASM2 model are:

- Aerobic/anoxic/anaerobic hydrolysis of slowly biodegradable substrates: The three processes consider the same seven components: S_F, S_NH4, S_PO4, S_I, S_ALK, X_S and X_TSS. These and all other components in ASM2 are shown in Table 2.5. The stoichiometric coefficients for S_NH4, S_PO4, S_ALK and X_TSS are calculated for each process using continuity equations and specific conversion factors. The kinetic equation under aerobic conditions is:

\[
\frac{dX_S}{dt} = -K_O \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{X_S/X_H}{X + X_S/X_H} \cdot X_H \quad (F.13)
\]

All kinetic parameters utilised in ASM2 are explained in Table F.2. The kinetic equation under anoxic conditions is:

\[
\frac{dX_S}{dt} = -K_s \eta_{NO3} \cdot \frac{K_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NO3}}{K_{NO3} + S_{NO3}} \cdot \frac{X_S/X_H}{X + X_S/X_H} \cdot X_H \quad (F.14)
\]

The kinetic equation under anaerobic conditions is:
Aerobic growth on \( S_F \) and \( S_A \): This presents the growth of heterotrophic organisms under aerobic conditions utilising soluble substrates (\( S_S \)), which have been broken down to fermentable, readily biodegradable organic substrates (\( S_F \)) and fermentation products (\( S_A \)). This breakdown is necessary as fermentation has been included as a biological process so fermentable products must be modelled separately from other soluble organic materials. The kinetic equation for growth on \( S_F \) is:

\[
\frac{dX_H}{dt} = \frac{\mu_H}{S_{NH4}} \cdot \frac{S_{F}}{S_{PO4}} \cdot \frac{S_F}{S_ALK} \cdot X_H
\]  

\( S_F \) is the rate limiting substrate for this process and is consumed for heterotrophic growth. The kinetic equation for growth on \( S_A \) is:

\[
\frac{dX_H}{dt} = \frac{\mu_H}{S_{NH4}} \cdot \frac{S_{A}}{S_{PO4}} \cdot \frac{S_A}{S_ALK} \cdot X_H
\]  

The substrate utilisation kinetics can be expressed by Equation F.18 using the coefficient \( Y_H \).

\[
\frac{dS_F}{dt} = \frac{1}{Y_H} \cdot \frac{dX_H}{dt}, \quad \frac{dS_A}{dt} = \frac{1}{Y_H} \cdot \frac{dX_H}{dt}
\]

Anoxic growth on \( S_F \) and \( S_A \) (denitrification): This describes the growth of heterotrophic organisms under anoxic conditions using \( S_F \) and \( S_A \) as the electron donor, respectively. Denitrification occurs as this process uses nitrate as the electron acceptor. The kinetic equation for growth on \( S_A \) is:

\[
\frac{dX_H}{dt} = \frac{\mu_H}{S_{NH4}} \cdot \frac{S_{F}}{S_{PO4}} \cdot \frac{S_F}{S_ALK} \cdot X_H
\]  

The kinetic equation for growth on \( S_A \) is:

\[
\frac{dX_H}{dt} = \frac{\mu_H}{S_{NH4}} \cdot \frac{S_{A}}{S_{PO4}} \cdot \frac{S_A}{S_ALK} \cdot X_H
\]

Fermentation: This process is the conversion of fermentable substrates (\( S_F \)) to fermentation products (\( S_A \)) under anaerobic conditions. The kinetic equation is:
Lysis: This process is the decay of heterotrophic organisms in aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

\[
\frac{dS}{dt} = -\frac{q_{fe} \cdot K_{O_2} \cdot S_{NO_3} \cdot S_F}{K_{fe} + S_F \cdot K_{ALK} + S_{ALK} \cdot X_H} \tag{F.21}
\]

Storage of cell internal storage products of PAOs (X\textsubscript{PHA}): this process is the storage of the X\textsubscript{PHA} material using the energy produced by the release of phosphate from polyphosphate by PAOs. The kinetic equation of this process is:

\[
\frac{dX_{PHA}}{dt} = \frac{q_{PHA} \cdot S_A \cdot S_{ALK} \cdot X_{pp}/X_{PAO} \cdot Y_{PO4}}{K_A + S_A \cdot K_{ALK} + S_{ALK} \cdot K_{pp} + X_{pp}/X_{PAO} \cdot X_{PAO}} \tag{F.23}
\]

Y\textsubscript{PO4}, poly-phosphate requirement for PHA storage, is used to represent phosphate release kinetics through Equations F.24 and F.25, respectively.

\[
\frac{dS_{PO4}}{dt} = Y_{PO4} \cdot \frac{dX_{PHA}}{dt} \tag{F.24}
\]

\[
\frac{dX_{pp}}{dt} = -Y_{PO4} \cdot \frac{dX_{PHA}}{dt} \tag{F.25}
\]

Storage of X\textsubscript{PP}: This process is the storage of orthophosphates in the form of polyphosphate. The PAOs obtain energy from the respiration of X\textsubscript{PHA}. The kinetic equation of this process is:

\[
\frac{dX_{pp}}{dt} = q_{pp} \cdot \frac{S_{O_2} \cdot S_{PO4} \cdot S_{ALK} \cdot X_{PHA}/X_{PAO} \cdot X_{PP}/X_{PAO} \cdot X_{PAO}}{K_{O_2} + S_{O_2} \cdot K_{pp} \cdot S_{PO4} \cdot S_{ALK} \cdot X_{PHA}/X_{PAO} \cdot X_{PP}/X_{PAO} \cdot X_{PAO}} \tag{F.26}
\]

The respiration kinetics of X\textsubscript{PHA} is described in Equation F.27 by using Y\textsubscript{PHA}, which is the PHA requirement for poly-phosphate storage.

\[
\frac{dX_{PHA}}{dt} = -Y_{PHA} \cdot \frac{dX_{pp}}{dt} \tag{F.27}
\]

Aerobic growth of X\textsubscript{PAO}: This process is the growth of PAOs under aerobic conditions. The kinetic equation of this process is:
The change rate of $X_{PHA}$ due to the growth of $X_{PAO}$ is described using Equation F.29.

\[
\frac{dX_{PHA}}{dt} = \frac{1}{Y_H} \cdot \frac{dX_{PAO}}{dt}
\]

- **Lysis of $X_{PAO}$**: This process is the decay of P accumulating organisms under aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

\[
\frac{dX_{PAO}}{dt} = -b_{PAO} \cdot X_{PAO} \cdot \frac{S_{ALK}}{K_{ALK} + S_{ALK}}
\]

- **Lysis of $X_{PP}$**: This process is the decay and decline of poly-phosphates under aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

\[
\frac{dX_{PP}}{dt} = -b_{PP} \cdot X_{PP} \cdot \frac{S_{ALK}}{K_{ALK} + S_{ALK}}
\]

- **Lysis of $X_{PHA}$**: This process is the decay of PHAs during aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

\[
\frac{dX_{PHA}}{dt} = -b_{PHA} \cdot X_{PHA} \cdot \frac{S_{ALK}}{K_{ALK} + S_{ALK}}
\]

- **Aerobic growth of $X_{AUT}$**: This process is the growth of nitrifying organisms under aerobic conditions. The kinetic equation of this process is:

\[
\frac{dX_{AUT}}{dt} = \mu_{AUT} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NH4}}{S_{NH4} + S_{NH4}} \cdot \frac{S_{PO4}}{K_{P} + S_{PO4}} \cdot \frac{S_{ALK}}{K_{ALK} + S_{ALK}} \cdot X_{AUT}
\]

- **Lysis of $X_{AUT}$**: This process controls the decay of nitrifying organisms under aerobic, anaerobic and anoxic conditions. The kinetic equation of this process is:

\[
\frac{dX_{AUT}}{dt} = -b_{AUT} \cdot X_{AUT}
\]

- **Phosphorus precipitation**: this process is the precipitation of P via chemical P removal. The kinetic equation of this process is:

\[
\frac{dS_{PO4}}{dt} = -k_{PRE} \cdot S_{PO4} \cdot X_{MeOH}
\]

- **Redissolution**: this process is the redissolution of P in the chemical P removal process. The kinetic equation for this process is:
Table F.2: ASM2 and ASM2d kinetic parameters (Scientific and Technical Report No. 9, 2000)

<table>
<thead>
<tr>
<th>Parameter Symbol</th>
<th>Kinetic parameter description</th>
</tr>
</thead>
<tbody>
<tr>
<td>µ_{AUT}</td>
<td>Maximum growth rate for nitrifiers</td>
</tr>
<tr>
<td>µ_H</td>
<td>Maximum growth rate on substrate for heterotrophic organisms</td>
</tr>
<tr>
<td>µ_{PAO}</td>
<td>Maximum growth rate for PAOs</td>
</tr>
<tr>
<td>K_A</td>
<td>Saturation coefficient for S_A (acetate)</td>
</tr>
<tr>
<td>K_{ALK}</td>
<td>Saturation coefficient for alkalinity</td>
</tr>
<tr>
<td>K_F</td>
<td>Saturation coefficient for growth on S_F</td>
</tr>
<tr>
<td>K_e</td>
<td>Saturation coefficient for fermentation of S_F</td>
</tr>
<tr>
<td>K_h</td>
<td>Hydrolysis rate constant</td>
</tr>
<tr>
<td>K_{IPP}</td>
<td>Inhibition coefficient for X_PP storage</td>
</tr>
<tr>
<td>K_{MAX}</td>
<td>Maximum ratio of X_PP/X_{PAO}</td>
</tr>
<tr>
<td>K_{NH4}</td>
<td>Saturation coefficient for ammonium</td>
</tr>
<tr>
<td>K_{NO3}</td>
<td>Saturation/inhibition coefficient for nitrate</td>
</tr>
<tr>
<td>K_O2</td>
<td>The definition of this parameter varies depending on the processes involved. It is the saturation/inhibition coefficient for oxygen for hydrolysis and heterotrophic organisms, is the saturation coefficient for S_O2 parameter for P-accumulating organisms, and is the saturation coefficient for oxygen for nitrifiers.</td>
</tr>
<tr>
<td>K_P</td>
<td>The definition of this parameter varies depending on the processes involved. It is the saturation coefficient for P for nitrifiers and heterotrophic organisms, and is the saturation coefficient for P in growth for the P-accumulating organisms.</td>
</tr>
<tr>
<td>K_{PHA}</td>
<td>Saturation coefficient for PHA</td>
</tr>
<tr>
<td>K_{PP}</td>
<td>Saturation coefficient for poly-phosphate</td>
</tr>
<tr>
<td>K_{PS}</td>
<td>Saturation coefficient for P in PP storage</td>
</tr>
<tr>
<td>K_X</td>
<td>Saturation coefficient for particulate COD</td>
</tr>
<tr>
<td>k_{PRE}</td>
<td>Rate constant for P precipitation</td>
</tr>
<tr>
<td>k_{RED}</td>
<td>Rate constant for P redissolution</td>
</tr>
<tr>
<td>q_fe</td>
<td>Maximum rate for fermentation</td>
</tr>
<tr>
<td>q_{PHA}</td>
<td>Rate constant for storage of PHA (base: X_PP)</td>
</tr>
<tr>
<td>q_PP</td>
<td>Rate constant for storage of PP</td>
</tr>
<tr>
<td>η_{fe}</td>
<td>Anaerobic hydrolysis reduction factor</td>
</tr>
<tr>
<td>η_{NO3}</td>
<td>The definition of this parameter varies depending on the processes involved. It is the anoxic hydrolysis reduction factor and the reduction factor for denitrification for heterotrophic organisms. It is also the reduction factor for anoxic activity for the ASM2d model.</td>
</tr>
<tr>
<td>b_{AUT}</td>
<td>Decay rate for nitrifiers</td>
</tr>
<tr>
<td>b_H</td>
<td>Rate constant for lysis for heterotrophic organisms</td>
</tr>
<tr>
<td>b_{PAO}</td>
<td>Rate constant for lysis of X_{PAO}</td>
</tr>
<tr>
<td>b_{PHA}</td>
<td>Rate constant for lysis of X_PHAs</td>
</tr>
<tr>
<td>b_{PP}</td>
<td>Rate constant for lysis of X_PP</td>
</tr>
</tbody>
</table>

ASM2d Processes and Components

In comparison to the ASM2 model, some new processes have been added in ASM2d. The storage of X_PP process has been divided into two separate processes, which are:

- Aerobic storage of X_PP: This process utilises the same components, stoichiometric coefficients and rate equation as the ASM2 model's storage of X_PP process. This process is to be differentiated from the newly added anoxic storage of X_PP process.
• Anoxic storage of X_{PP}: This process is the storage of poly-phosphate in the absence of oxygen and when nitrate is used as the electron acceptor. The kinetic equation of this process is:

\[
\frac{dX_{PP}}{dt} = \rho_{11} \cdot \eta_{N03} \cdot \frac{K_{O2}}{S_{O2} + S_{N03}} S_{N03}
\]  

(F.37)

All the kinetics of ASM2d are described in Table 2.8. The Y_{PHA} stoichiometric parameter is used to link the X_{PP} storage kinetics to the kinetics of respiration of X_{PHA} using Equation F.38.

\[
\frac{dX_{PHA}}{dt} = -Y_{PHA} \frac{dX_{PP}}{dt}
\]  

(F.38)

The growth of X_{PAO} under anoxic conditions is also considered.

• Anoxic growth of X_{PAO}: This process is the growth of PAOs in the absence of oxygen and when nitrate is used as an electron acceptor. The rate equation for this process is:

\[
\frac{dX_{PAO}}{dt} = \rho_{13} \cdot \eta_{N03} \cdot \frac{K_{O2}}{S_{O2} + S_{N03}} S_{N03}
\]  

(F.39)

The X_{PHA} utilisation kinetics is described in Equation F.40.

\[
\frac{dX_{PHA}}{dt} = -\frac{1}{Y_{H}} \frac{dX_{PAO}}{dt}
\]  

(F.40)

Some changes are also made to the stoichiometric coefficients of certain components.

**ASM3 Processes and Components**

ASM3 consists of 13 components and 12 processes. These processes and their kinetics are:

• Hydrolysis: This process is the release of all slowly biodegradable substrates (X_{S}) under the aerobic, anaerobic and anoxic conditions. This process is associated with six components; S_{I}, S_{S}, S_{NH4}, S_{ALK}, X_{S} and X_{SS}. These and all other components in ASM3 are shown in Table F.3. The coefficients for S_{S}, S_{NH4} and S_{ALK}, x_{j}, y_{j} and z_{j}, are calculated using a conservation equation for the three conservatives: theoretical oxygen demand (ThOD), nitrogen and ionic charge. This equation is:

\[
\sum_{i} v_{j,i} \cdot i_{k,i}
\]  

(F.41)

The i_{k,i} values are found in the composition matrix, shown in Table 2.7. The kinetic
equation of this process is:
\[
\frac{dX_S}{dt} = -k_H \cdot \frac{X_S/X_H}{K_X + X_S/X_H} \cdot X_H
\]  \hspace{1cm} (F.42)

All the kinetic parameters of ASM3 are shown in Table 2.10.

- **Aerobic Storage of S\(_S\)**: This process is the storage of S\(_S\) as cell internal storage products (X\(_{STO}\)) under aerobic conditions. The energy necessary for this process is provided through aerobic respiration. The kinetic equation of this process is:
\[
\frac{dS_S}{dt} = -k_{STO} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_S}{K_S + S_S} \cdot X_H
\]  \hspace{1cm} (F.43)

The production kinetics of X\(_{STO}\) are described in Equation F.44 by using Y\(_{STO,O2}\), which is the aerobic yield of stored products per S\(_S\).
\[
\frac{dX_{STO}}{dt} = -Y_{STO,O2} \cdot \frac{dS_S}{dt}
\]  \hspace{1cm} (F.44)

- **Anoxic Storage of S\(_S\)**: This process is the storage of S\(_S\) as cell internal storage products (X\(_{STO}\)) under anoxic conditions, where nitrate is used as the electron acceptor. The energy necessary for this process is provided through denitrification.

The kinetic equation of this process is:
\[
\frac{dS_S}{dt} = -k_{STO} \cdot \eta_{NOX} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot \frac{S_S}{K_S + S_S} \cdot X_H
\]  \hspace{1cm} (F.45)

Similarly to the aerobic storage of S\(_S\) process, the production kinetics of X\(_{STO}\) is described in Equation F.46 using the parameter Y\(_{STO,NOX}\), which is the anoxic yield of stored products per S\(_S\).
\[
\frac{dX_{STO}}{dt} = -Y_{STO,NOX} \cdot \frac{dS_S}{dt}
\]  \hspace{1cm} (F.46)

The stoichiometric coefficients for the X\(_SS\) component are calculated using the composition equation. This equation is:
\[
\sum_i v_{j,i} \cdot t_{k,i}
\]  \hspace{1cm} (F.47)

- **Aerobic growth of X\(_H\)**: This process is the growth of heterotrophic organisms under aerobic conditions. The substrate utilised for growth is assumed to be X\(_{STO}\), thereby simplifying the ASM3 model. The kinetic equation for this process is:
The kinetics of the consumption of X STO is described in Equation F.49 by using \( Y_{H,02} \), which is the aerobic yield of heterotrophic biomass.

\[
\frac{dX_H}{dt} = \mu_H \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NH4}}{K_{NH4} + S_{NH4}} 
\]

\[
\frac{S_{ALK}}{K_{ALK} + S_{ALK}} \cdot \frac{X_{STO}/X_H}{K_{STO} + X_{STO}/X_H} \cdot X_H
\]

(F.48)

(F.49)

- Anoxic growth (denitrification): This process is the growth of heterotrophic organisms under anoxic conditions, in which nitrate is the electron acceptor. This results in denitrification. The kinetic equation for this process is:

\[
\frac{dX_H}{dt} = \mu_H \cdot \eta_{NOX} \cdot \frac{K_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NH4}}{K_{NH4} + S_{NH4}} 
\]

\[
\frac{S_{ALK}}{K_{ALK} + S_{ALK}} \cdot \frac{X_{STO}/X_H}{K_{STO} + X_{STO}/X_H} \cdot X_H
\]

(F.50)

(F.51)

- Aerobic endogenous respiration (heterotrophic organisms): This process is biomass loss under aerobic conditions, not related to growth processes. The kinetic equation for this process is:

\[
\frac{dX_H}{dt} = -b_{H,O2} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot X_H
\]

(F.52)

- Anoxic endogenous respiration (heterotrophic organisms): This process is biomass loss under anoxic conditions which are not related to growth processes. The kinetic equation of this process is:

\[
\frac{dX_H}{dt} = -b_{H,NOX} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot X_H
\]

(F.53)

- Aerobic respiration of X STO: This process corresponds to the endogenous respiration process. It is the decay of X STO together with biomass. The kinetic equation of this process is:

\[
\frac{dX_{STO}}{dt} = -b_{STO,O2} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot X_{STO}
\]

(F.54)
• Anoxic respiration of $X_{STO}$: This process corresponds to the aerobic respiration process and is the decay of $X_{STO}$ together with biomass under anoxic conditions. The kinetic equation of this process is:

$$\frac{dX_{STO}}{dt} = -b_{STO, NOX} \cdot \frac{S_{O2}}{K_{O2} + S_{O2}} \cdot \frac{S_{NOX}}{K_{NOX} + S_{NOX}} \cdot X_{STO}$$ (F.55)

• Aerobic growth of $X_A$: This process is the growth of autotrophic organisms under aerobic conditions. The kinetic equation of this process is:

$$\frac{dX_A}{dt} = v_A \cdot \frac{S_{O2}}{K_{A, O2} + S_{O2}} \cdot \frac{S_{NH4}}{K_{A, NH4} + S_{NH4}}$$

$$\cdot \frac{S_{ALK}}{K_{A, ALK} + S_{ALK}} \cdot X_A$$ (F.56)

• Aerobic endogenous respiration (autotrophic organisms): This process is biomass loss of autotrophic organisms under aerobic conditions, not related to the autotrophic growth process. The kinetic equation of this process is:

$$\frac{dX_A}{dt} = -b_{A, O2} \cdot \frac{S_{O2}}{K_{A, O2} + S_{O2}} \cdot X_A$$ (F.57)

• Anoxic endogenous respiration (autotrophic organisms): This process is biomass loss of autotrophic organisms under anoxic conditions, not related to the growth process of autotrophic organisms. The kinetic equation of this process is:

$$\frac{dX_A}{dt} = -b_{A, NOX} \cdot \frac{S_{O2}}{K_{A, O2} + S_{O2}} \cdot \frac{S_{NOX}}{K_{A, NOX} + S_{NOX}} \cdot X_A$$ (F.58)
Table F.3. ASM3 kinetic parameters (Scientific and Technical Report No. 9, 2000)

<table>
<thead>
<tr>
<th>Parameter Symbol</th>
<th>Kinetic parameter description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_H )</td>
<td>Hydrolysis rate constant</td>
</tr>
<tr>
<td>( k_{STO} )</td>
<td>Storage rate constant</td>
</tr>
<tr>
<td>( K_{A,ALK} )</td>
<td>Bicarbonate saturation for nitrifiers</td>
</tr>
<tr>
<td>( K_{A,NH4} )</td>
<td>Ammonium substrate saturation for ( X_A )</td>
</tr>
<tr>
<td>( K_{A,NOX} )</td>
<td>Saturation constant for ( S_{NOX} ) for ( X_A )</td>
</tr>
<tr>
<td>( K_{A,O2} )</td>
<td>Oxygen saturation for nitrifiers</td>
</tr>
<tr>
<td>( K_{ALK} )</td>
<td>Saturation constant for alkalinity for ( X_H )</td>
</tr>
<tr>
<td>( K_{NH4} )</td>
<td>Saturation constant for ammonium, ( S_{NH4} )</td>
</tr>
<tr>
<td>( K_{NOX} )</td>
<td>Saturation constant for ( S_{NOX} )</td>
</tr>
<tr>
<td>( K_{O2} )</td>
<td>Saturation constant for ( S_{NO2} )</td>
</tr>
<tr>
<td>( K_{S} )</td>
<td>Saturation constant for ( S )</td>
</tr>
<tr>
<td>( K_{STO} )</td>
<td>Saturation constant for ( X_{STO} )</td>
</tr>
<tr>
<td>( K_X )</td>
<td>Hydrolysis saturation constant</td>
</tr>
<tr>
<td>( \eta_{NOX} )</td>
<td>Anoxic reduction factor</td>
</tr>
<tr>
<td>( \mu_A )</td>
<td>Autotrophic maximum growth rate of ( X_A )</td>
</tr>
<tr>
<td>( \mu_H )</td>
<td>Heterotrophic maximum growth rate of ( X_H )</td>
</tr>
<tr>
<td>( b_{A,O2} )</td>
<td>Aerobic endogenous respiration rate of ( X_A )</td>
</tr>
<tr>
<td>( b_{A,NOX} )</td>
<td>Anoxic endogenous respiration rate of ( X_A )</td>
</tr>
<tr>
<td>( b_{H,O2} )</td>
<td>Aerobic endogenous respiration rate of ( X_H )</td>
</tr>
<tr>
<td>( b_{STO,O2} )</td>
<td>Anoxic endogenous respiration rate of ( X_H )</td>
</tr>
</tbody>
</table>

PLC Networks

The PLC networks contained the ladder logic code for activating the various PLC actions. Network 1 acted as the activation control for all PLC operation controls (Fig. F.1). It contained an open contact which was a single input (I0.3), and a control output (V0.1). The input was an on/off switch which turned on/off the output. The V0.1 output was a memory slot within the PLC. It activated the operation control networks in the software.

![Figure F.1. Network 1](image)

Networks 2, 3 and 4 controlled the timing within the cSBR and IASBR units (Fig. F.2 a, b and c, respectively). Network 2 contained a single memory bit input (M0.0) which was deactivated when receiving a signal, and a timer output (T33). When the input signal was on the timer was activated every second. Network 3 contained an open contact input connected to T33. When the timer was activated this contact was on, which activated the READ_RTC_I output. The READ_RTC_I function was a sub program which converted the real time PLC clock from a binary-coded decimal format to an integer format. The activation of T33 sent the time in the integer format to a memory location. The memory location was named VB10 and
stored real clock data in the format shown in Table F.4. Network 4 contained the T33 input which activated M0.0. This caused T33 in Network 2 to reset and restart the process.

**Figure F.2.** Networks 2 (a), 3 (b) and 4 (c)

**Table F.4.** Real clock data storage format

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>VB10</td>
<td>Year</td>
</tr>
<tr>
<td>VB11</td>
<td>Month</td>
</tr>
<tr>
<td>VB12</td>
<td>Day</td>
</tr>
<tr>
<td>VB13</td>
<td>Hour</td>
</tr>
<tr>
<td>VB14</td>
<td>Minute</td>
</tr>
<tr>
<td>VB15</td>
<td>Second</td>
</tr>
</tbody>
</table>

Networks 5 and 6 contained the original control code for solenoid valves withdrawing
effluent from the cSBR and IASBR units. These were replaced with a timer plug for a
peristaltic pump which utilised two pump heads to decant treated wastewater from each
reactor. The timer was activated at the end of each settle period and was deactivated 10
minutes later. The volume decanted was controlled by fixing the effluent tubing into a proper
position. Networks 7 and 8 controlled magnetic stirrers in the IASBR and cSBR units,
respectively (Fig. F.3 a and b). The network contained the open contact V0.1 input and VB13
comparison contact inputs. The mixer was activated at the specified time input. When the
stirrer was turned off, the settle period began.

![Network 7](image1)

(a)

![Network 8](image2)

(b)

**Figure F.3.** Network 7 (a) and Network 8 (b)

Networks 9 and 10 controlled the air pumps for the IASBR and cSBR units, respectively
(Fig. F.4 a and b). The network contained the open contact V0.1 input and the VB13 and
VB14 comparison contact inputs. The air pumps were activated at the specified time inputs.
Network 11 controlled the influent pump drive for both reactor units (Fig. F.5) and was the final network in the PLC code. The network contained the open contact V0.1 input, the VB13 and VB14 comparison contact inputs and a closed contact float switch input from each unit. The float switches were positioned to close when the working volume had been reached, in order to prevent accidental overflow. The VB14 contact was used if the float switches had not been activated after 11 minutes to prevent accidental overflow.

Figure F.4. Network 9 (a) and Network 10 (b)

Figure F.5. Network 11