Economic and Environmental Cost Assessment of Wastewater Treatment Systems

A Life Cycle Perspective

By

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DECLARATION

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Abstract

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Wastewater treatment systems have economic and environmental costs associated with their construction and operation. These costs vary with location because of the specific conditions under which a treatment plant must be built and operated. A challenge for authorities is selecting the most appropriate treatment system for a given location. This requires an understanding of how competing systems will perform in a given scenario, and how variations in performance influence the associated costs. Small agglomerations in particular face unique challenges during system selection. These are often rural communities where access to resources and wastewater treatment expertise may be minimal, or come at a higher cost. It is, therefore, evident that appropriate system assessment tools are required to assist in the selection process. The objective of this study was to present a methodology to assess system performance under changing conditions, and elucidate the trade–offs that can occur between capital and operational costs, environmental impact categories, and ultimately between the overall economic and environmental costs. A review of the literature has determined that the life cycle approach provides a holistic understanding of the actual cost of system implementation. Thus, life cycle costing and life cycle assessment were the analytical frameworks selected for the study. A decision support tool that integrated both frameworks was developed to facilitate system analysis in user-defined, site-specific scenarios. Life cycle inventories were compiled with data collected from a selection of wastewater treatment plants, and from life cycle assessment process datasets. The life cycle cost data were compiled from a variety of academic and industry sources. To assess the methodology, ten wastewater treatment systems were evaluated under a range of predetermined site-specific scenarios that varied in scale, loading, discharge limits, and method of sludge disposal. In general, system analyses showed that treatment systems with the capacity to mitigate energy and chemical consumption exhibited more favourable economic and environmental life cycle profiles. The methodology illustrated the importance of conducting system assessment from a life cycle perspective and highlighted system processes and components that provide the greatest potential for system improvement and cost savings.

Abbreviations used in this thesis

AAO	Anaerobic Anoxic Oxic
ADPe	Abiotic Depletion Potential element
ADPf	Abiotic Depletion Potential fossil
ANP	Analytic Network Process
AO	Anoxic Oxic
AOB	Ammonia Oxidising Bacteria
AP	Acidification Potential
AS	Activated Sludge
bCOD	biodegradable Chemical Oxygen Demand
BOD	Biochemical Oxygen Demand
CAS	Conventional Activated Sludge
CBA	Cost Benefit Analysis
CCI	Construction Cost Index
CED	Cumulative Energy Demand
CHP	Combined Heat and Power
CMAS	Complete Mix Activated Sludge
CML	Centre for Environmental Science (Leiden University)
COD	Chemical Oxygen Demand
CW	Constructed Wetlands
DAF	Dissolved Air Flotation
DESASS	Design and Simulation of Activated Sludge Systems
DL	Discharge Limit
DS	Dry Solids
DSC	Dry Solids Concentration
EA	Exergy Analysis
EBPR	Enhanced Biological Phosphorus Removal

EF	Ecological Footprint
EIA	Environmental Impact Assessment
EP	Eutrophication Potential
ERA	Environmental Risk Assessment
FAETP	Freshwater Aquatic Eco-toxicity Potential
GHG	Green House Gas
GWP	Global Warming Potential
HFBR	Horizontal Flow Biofilm Reactors
HLR	Hydraulic Loading Rate
HTP	Human Toxicity Potential
ICW	Integrated Constructed Wetlands
IFAS	Integrated Fixed Film Activated Sludge
iTSS	inert Total Suspended Solids
LCA	Life Cycle Assessment
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LCCI	Life Cycle Cost Inventory
LCI	Life Cycle Inventory
LCIA	Life Cycle Impact Assessment
LCM	Life Cycle Management
MAETP	Marine Aquatic Eco-toxicity Potential
MBBR	Moving Bed Biofilm Reactor
MBR	Membrane Bioreactor
MLD	Million Litres Day
MLE	Modified Ludzack-Ettinger
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
nbVSS	nonbiodegradable Volatile Suspended Solids

NDFA	National Development Finance Agency
NEIWPCC	New England Interstate Water Pollution Control Commission
NOB	Nitrite-Oxidising Bacteria
NPV	Net Present Value
OD	Oxidation Ditch
ODP	Ozone Depletion Potential
OLR	Organic Loading Rate
PAH	Polycyclic Aromatic Hydrocarbons
pBOD	particulate Biochemical Oxygen Demand
PCB	Polychlorinated Biphenyls
PCOP	Photo-chemical Oxidation Potential
PD	Positive Displacement
PFBR	Pump Flow Bioreactor
PFT	Picket Fence Thickeners
PV	Present Value
RAS	Return Activated Sludge
RBC	Rotating Biological Contactor
RO	Reverse Osmosis
sBOD	soluble Biochemical Oxygen Demand
SBR	Sequence Batch Reactor
SCADA	Supervisory Control and Data Acquisition
sCOD	soluble Chemical Oxygen Demand
SETAC	Society of Environmental Toxicology and Chemistry
SMBR	Submerged Membrane Bioreactor
SPPWF	Single Payment Present Worth Factor
SPV	Single Present Value
SRT	Solids Retention Time
SS	Suspended Solids

TDH	Total Dynamic Head
TETP	Terrestrial EcoToxicity Potential
TF	Trickling Filter
TIC	Total Inorganic Carbon
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TOC	Total Organic Carbon
TP	Total Phosphorus
TSS	Total Suspended Solids
UBOD	Ultimate Biochemical Oxygen Demand
UPV	Uniform Present Value
VFA	Volatile Fatty Acid
VOC	Volatile Organic Carbons
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WLCC	Whole Life Cycle Cost
WSP	Waste Stabilisation Pond
WWT	Wastewater treatment
WWTP	Wastewater treatment Plant
WWTS	Wastewater treatment System

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Publications resulting from this research

Journal

McNamara, G., Horrigan, M., Phelan, T., Fitzsimons, L., Delaure, Y., Corcoran, B., Doherty, E. and Clifford, E., (2014), "Life cycle assessment of waste water treatment plants in Ireland", *Journal of Sustainable Development of Energy, Water and Environment Systems,* Vol.4 (3), 216-233.

McNamara, G., T., Fitzsimons, L., Doherty, E. and Clifford, E., (2016), "The evaluation of technologies for small, new design, wastewater treatment systems", *Journal of Desalination and Water Treatment,* Vol. 91, 12-22.

Conference

McNamara, G., Horrigan, M., Phelan, T., Fitzsimons, L., Delaure, Y., Corcoran, B., Doherty, E. and Clifford, E., (2014), "Life cycle assessment of waste water treatment plants in Ireland, SEE SDEWES Ohrid 2014, 1st South East European Conference on Sustainable Development of Energy, Water and Environment Systems, 29 June - 3rd July, Ohrid, Macedonia.

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Report

Fitzsimons, L., Clifford, E., McNamara, G., Doherty, E., Phelan, T., Horrigan, M., Delauré, Y. and Corcoran, B., (2016), "*Increasing Resource Efficiency in Wastewater Treatment Plants*", Strive Report 2012-W-MS-10, Environmental Protection Agency, Wexford, Ireland.

1 Introduction

1.1 Background

Wastewater treatment (WWT) describes the process whereby pollutants that are harmful to humans and the environment are removed from wastewater through a series of unit processes that make up a wastewater treatment system (WWTS). Conventional WWTSs achieve pollutant removal by different means. Natural systems remove pollutants by mimicking natural occurring WWT processes, which require minimal human interaction, energy or resources, but require large surface areas. Electro-mechanical WWTSs are more compact but can require significant energy, resources, and process control. Each type of WWTS has particular strengths and limitations that make them more applicable to a given set of site-specific conditions than others. The conditions under which WWTSs must operate vary with location. The scale of the agglomeration being served is a key factor because some systems are more suited to small scale agglomerations, while other systems exhibit significant economies of scale. Some systems are better equipped to handle high organic and inorganic loading, while others perform optimally when loading is low and at a relatively steady state. In Ireland, one of the most influential site-specific conditions that plant operators have to contend with is the final effluent discharge limits. The discharge limits define the type and quantity of substrate to be removed from the wastewater and are determined by the sensitivity of the final effluent receiving waters. Receiving waters in Ireland vary from inland freshwater bodies for which effluent concentrations of nitrogen and phosphorus are required to be reduced to some predetermined level, to estuary and coastal waterbodies where the impact of nutrients has been deemed less critical, discharge limits are often at their least stringent, and in many cases, particularly for small systems, only biochemical oxygen demand (BOD) and solids are required to be removed.

In general, most modern WWTSs can achieve high levels of pollutant removal. However, the economic and environmental performance of each treatment system will vary depending on the type and quantity of substrate to be removed. Schumacher [1] stated that the appropriate technology is always contextual and situational. This suggests that there is no ‘one size fits all’ solution applicable to every location, or more specifically, for every location there is one system, or system configuration that will outperform all others. The problem here is how to determine which system is most appropriate for a particular location. According to Molinos-Senante et al. [2] the selection of the most appropriate wastewater treatment technology is the biggest challenge faced by wastewater treatment management. Historically, the initial capital expenditure has often been the deciding factor in the system selection process; however, in more recent times, there is an awareness that the operational costs over a system’s lifetime can be much greater than the initial capital investment, and that both the initial capital and operational costs need to be assessed together in order to understand the actual cumulative cost over a system’s lifetime.

In addition to the direct economic cost assessment, society, business, and government have become more environmentally aware. It is widely understood that the environmental profile of a product or system extends far beyond the immediate point of manufacture or operation, and that these indirect environmental consequences can also have financial implications. The cost of global abiotic resources will increase with an increasing global population, and carbon tax creates a direct link to greenhouse gases emissions. This has changed the nature of the procurement process from a solely economic exercise, to include sustainability factors. Pasqualino et al. [3] state that

> “*...the goals for wastewater treatment systems need to move beyond the protection of human health and surface waters to also minimizing the loss of resources, reducing the use of energy and water, reducing waste generation, and enabling the recycling of nutrients.”*

However, the inclusion of environmental factors adds another layer of complexity to the decision making process, and requires the appropriate tools to evaluate system performance. Environmental assessments can be costly and time consuming exercises that require large amounts of data. The provision of an environmental assessment tool that limits the extent of data acquisition may result in their use becoming more amenable to decision makers.

The population spread in Ireland is such that there are 587 wastewater treatment plants (WWTPs) that serve agglomerations of below 2,000 PE[1] (population equivalent). For small agglomerations, the challenge of selecting the most appropriate WWTS is even more difficult. Small WWTPs are often unmanned and located in isolate or rural locations. There may be issues with the availability of skilled labour. Operational costs may be higher because of lower energy efficiencies, lower sludge disposal and chemical cost discount opportunities. Safety factors may also be unnecessarily high in order to mitigate the risk of compliance failure. Conversely, capital expenditure for small systems is often the dominant cost factor, which puts the economic and environmental costs in direct conflict with each other as it is a system's operational phase that has the most significant environmental cost.

1.2 Objectives

The objectives of this thesis are presented in two parts: 1) a life cycle assessment (LCA) of a selection of WWTPs, and 2) the development of a WWTS selection methodology and software tool.

1.2.1 Preliminary LCA study

The challenge of controlling WWTP operational cost has grown as discharge limits have become more stringent. These limits are decided upon through an assessment of a WWTP's receiving water body that determines acceptable levels of eutrophication and aquatic toxicity. However, eutrophication and aquatic toxicity are only two parts of the broad environmental spectrum that is affected by the WWT process. It is postulated that contributions to other non-aquatic environmental compartments (air and soil) are often increased as a result of efforts to control pollution of receiving water bodies; thereby, reallocating environmental impact both

[1] 1 PE (person equivalent) is estimated to be 0.2 m^3 of waste water influent and 60 g of BOD (biological oxygen demand) [4]

regionally and globally. Furthermore, variations in WWTP scale and organic loading will affect energy use and resource consumption to the extent that it can change a WWTP's environmental profile. Finally, studies have shown that the method of sludge treatment and disposal can also have varying environmental consequence. The heavy metal concentrations in sludge that is applied to agricultural farmland has been widely reported as the primary source of terrestrial ecotoxicity, and therefore, it is postulated that methods of sludge treatment and disposal that can reduce concentrations of heavy metals will produce a more favourable environmental profile.

The novelty in this part of study relates to its regional application. While there are many studies international LCA-WWT studies, to the best of the authors knowledge, no such study has been carried out in Ireland to date. Legislation, environmental conditions, and WWT practices will vary internationally, and therefore, it is necessary to conduct an environmental assessment of plants in Ireland in order to understand the impact from treatment in an Irish context. Hence, the objectives of the preliminary LCA study are

- to conduct energy audits of a selection of WWTPs in Ireland for the purpose of identifying the primary energy sinks within the systems and determining the extent to which energy consumption effects the overall environmental profile of a system;
- to determine the extent to which variations in scale, discharge limits and organic loading have on energy use, resource consumption, and environmental impact;
- to assess the environmental consequence of variations in the method of sludge treatment and disposal; and
- to determine suitable boundary definitions, process flows, functional units, and impact assessment methodology for integration into a WWTS decision support tool
- Evaluate LCA as an environmental assessment tool.

1.2.2 Wastewater treatment system assessment methodology and toolkit development for small wastewater treatment systems

Reviews of academic literature have highlighted the constant evolution of WWTS assessment and selection methods. These methods have ranged from simple capital cost comparisons to more complex multi-criteria decision making processes. It is generally understood that capital cost comparisons do not provide the most accurate representation of the cost of system ownership over its lifetime. Furthermore, most procurement processes require some level of sustainability evaluation. Conversely, multi-criteria decision making processes consider a range of economic, environmental, and performance related parameters such as capital and operational expenditure, sustainability, operational expertise, ease of use, robustness, reliability, and social acceptance. These types of assessment methodology generally involve assigning weights to each of the parameters and aggregating all of the weighted values into a single score. The issue with this approach is that the weighting system is generally a qualitative measure that is often subjective or opinion based. Furthermore, the aggregation of weighted values into a single score makes it difficult to identify aspects of system performance that have the potential for improvement. Additionally, WWTS energy use is central to both economic and environmental cost, and estimations of energy use for many system assessment methods are generally average values based on empirical data collected from existing systems. This approach may provide more realistic estimations of energy use because it includes inefficiencies that can occur within a system; however, because of data aggregation it does not allow for variations in loading and discharge limits that can occur between different systems in different locations. Finally, small scale WWTSs often forego any onsite sludge treatment because of the additional capital and operational costs involved. In some cases the sludge can be stored on-site and then delivered to a larger parent plant for treatment and ultimate disposal. In other situations plant management may choose instead to pay an external contractor to remove untreated sludge from site at a significant cost.

The hypothesis pertaining to the second part of the study is thus; there are economic, energetic, environmental, and in some cases, social costs associated with the implementation of wastewater treatment systems. These costs will vary with system and location, and therefore, must be assessed under the site-specific conditions. This requires a methodology that accounts for the multitude of parameters that influence system performance. Furthermore, these costs must be assessed from a lifecycle perspective because this is the best way to understand the true cost of system ownership. Hence, the objectives of this part of the study are

- to select appropriate tools and develop an economic and environmental assessment methodology for WWTSs serving small agglomerations. The methodology must account for variations in several key site-specific parameters, namely; scale, organic loading, discharge limits, and sludge treatment and disposal;
- using the developed methodology, design a WWTS decision support tool that accepts user-defined site-specific data and outputs system specific economic, environmental, and energy information; and
- using the developed software, investigate how variations in the site-specific conditions affect the economic and environmental life cycle costs.

1.3 Structure of thesis

The literature review is presented in Chapter 2 and begins with a brief introduction to the history and development of wastewater treatment. This includes an overview of various international water pollution and WWT acts that lead to the water quality regulations that are in place today. A brief overview of some common WWTSs currently in operation is provided to show how changes in conditions affect their performance. A review of the development of system assessment and selection methods is provided. The key aspects of current economic and environmental assessment tools are identified and discussed, and rationale is provided for the selection of the respective costing models. Chapter 3 contains the preliminary LCA study. It is in this phase of the research that the LCA methodology and assumptions are assessed. This phase of the research was the catalyst for many of the lines of investigation that would follow in the subsequent work. Chapter 4 presents the methodology adopted for the study beginning with an overview of the rationale for choosing the systems that were to be included in the study. Details relating to the LCA component of the decision support tool are provided in this chapter including additional information relating to the functional unit, system boundaries, and flows that were not relevant to the preliminary study. The final section of the chapter presents the life cycle cost (LCC) procedure and related cost information. Chapter 5 presents details of system energy modelling. Chapter 6 presents the decision support tool user interface and program architecture. Chapter 7 presents the method and results from systems analyses. Chapter 8 presents the conclusions, thesis contributions, and further work.

2 Literature Review

2.1 Introduction

The literature review includes a brief introduction to the history and development of wastewater treatment. An overview of water pollution and WWT legislation is provided to illustrate how tighter regulations lead to an increase in WWTP operational cost. Conventional WWTSs are reviewed, and additional background information is provided to help identify the key parameters that contribute to individual system performance. Wastewater treatment system selection methods are reviewed and evaluated. This is followed by a review of economic and environmental assessment tools.

2.2 Wastewater treatment

Wastewater treatment can be defined as the removal of harmful pollutants from a wastewater stream by physical, chemical or biological means, or by a combination of some or all of them. There is historical evidence to suggest that the concept of wastewater management dates back to the Mesopotamian Empire (3500–2500 BC). Babylonian ruins show dwellings with drainage systems designed to carry away wastewater [5]. In the period from 800 BC to 100 AD, Roman engineers implemented a system of sewer networks to transport wastewater in public latrines away from population centres in an effort to avoid the spread of diseases associated with human effluent [6]. However, after the collapse of the Roman Empire wastewater management went into decline, and throughout the Middle Ages (450 – 1750), all water was deemed unhealthy [7]. By the 1800s, many of the large cities throughout Europe had some form of sewer network, but the treatment of sewage was limited to removing solids from waste ponds or cesspits for use in agriculture. It was not until the 20th century that wastewater treatment in its conventional form began to develop. The first biological filter was installed in Wisconsin in the United States in 1901 [8]. This was a basic rock filter with algal growth formed in a riverbed. In the 1960s, eutrophication (EP) of surface water became an issue, and it prompted intensive research into methods for removing nitrogen and phosphorus from discharged effluent streams. This led to the use of Monod kinetics to model nitrification in WWT [9], a process that is still used today. It was becoming clear that greater control over the composition of effluent being discharged into water bodies was required. The 1960s and 1970s saw the introduction of various water pollution acts in many of the developed nations around the world. In Europe, the East German Government introduced *Das Wassergesetz 1963* (The 1963 Water Act) [10]. Similar measures were adopted by the French government in 1964 [11]. In the United States, the *1972 Clean Water Act* established the framework to control pollution of water [12]. In 1973 the U.K. government passed the *1973 Water Act.* [13]. On the 21st of May 1991 the then European Economic Community (EEC) issued the 91/271/EEC Urban Wastewater Treatment Directive (UWWTD) with the aim of protecting the environment from the adverse effects of effluent being discharged from wastewater treatment plants [14]. The directive made recommendations on the collection, treatment, and discharge of urban wastewater. One of the key recommendations was that WWTPs serving agglomerations greater than 2,000 PE discharging final effluent into freshwater estuaries, and all other agglomerations greater than 10,000 PE employ secondary treatment (Table 2-1 and Table 2-2). In Ireland, the discharge limits recommended in the UWWTD were adopted as the benchmark for systems serving agglomerations down to 500 PE. Some small WWTPs can be subject to even more stringent limits in areas of particular sensitivity. These tighter discharge limits puts pressure on the resources that are available to small WWTP operators, and makes the choice of the most appropriate system even more important.

Table 2-1: Regulations concerning discharge from urban wastewater treatment plants [14]

Parameter	*Concentration (mg/l)*	*Removal percentage*
BOD_5	25	70 - 90
COD	125	75
TSS (> 10,000 PE)	35	90
TSS (2,000 < PE < 10,000)	60	70

Table 2-2: Nutrient limits for sensitive areas [14]

Parameter	*Concentration (mg/l)*	*Removal percentage*
Total Phosphorous (10^4 < PE < 10^5)	2	80
Total Phosphorous (> 10^5)	1	
Total Nitrogen (10^4 < PE < 10^5)	15	70 - 80
Total Nitrogen (> 10^5)	10	

2.3 Wastewater treatment systems

Conventional WWTSs generally fall under one of four categories: suspended growth, attached growth, hybrid and natural systems. Table 2-3 presents some of the systems most commonly found in operation today. A general overview of each treatment system category is presented in this chapter, and additional system-specific information is included where it has been considered necessary to provide a clearer understanding. Mechanisms of nutrient removal are discussed in relation to the additional energy, capital, and operational resources required. The discussion begins with a review of natural systems.

Table 2-3: Categories of wastewater treatment systems

Suspended growth	**Attached growth**	**Hybrid**	**Natural**
Conventional Activated Sludge (CAS)	Rotating Biological Contactors (RBC)	Membrane Bioreactor (MBR)	Constructed Wetlands (CW)
Anoxic Oxic (AO)	Trickling Filter (TF)	Moving Bed Biofilm Reactor (MBBR)	Reed Bed (RB)
Anaerobic Anoxic Oxic (AAO)	Membrane Aerated Biofilm Reactor (MABR)	Integrated Fixed-Film Activated Sludge (IFAS)	Waste Stabilisation Pond (WSP)
Sequence Batch Reactor (SBR)	Pumped Flow Biofilm Reactor (PFBR)	CAS/TF	Aerated Lagoon
Extended Aeration (EA) - Oxidation Ditch (OD)	Horizontal Flow Biofilm Reactors (HFBR)	RBC/RB	

2.3.1 Natural systems

Natural WWTSs are low energy consumers that require large surface areas in which to operate. Although they are often referred to as low-tech systems, the mechanism by which pollutant removal is carried out is complex and specialised. There is a wide range of macrophytes and plants that are responsible for removing specific substances in specific environments and climates. Natural systems are particularly suited to rural, decentralised locations with small populations. However, low operational costs and low expertise requirements make them a feasible option wherever land availability is not an issue. The types of natural system currently in use include: reed beds, which are often used as a tertiary treatment stage for low nutrient removal requirements, waste stabilisation ponds (WSP), free water surface constructed wetlands (FWS CW), sub-surface horizontal flow constructed wetlands (HF CW), vertical-flow constructed wetlands (VF CW), soil and sand filters [15]. Integrated constructed wetlands (ICW) are a variation of FWS CW designed to function as more than just a wastewater treatment system. The systems are designed to integrate into the natural landscape, provide a habitat for a diversity of flora and fauna, and in some cases provide amenities for the local community, and visiting tourists [16].

Each natural treatment system has specific strengths and limitations that make them suitable to particular locations and conditions. The choice of system will depend largely on the required effluent quality and land availability, and in some cases, there may be a social aspect to be considered. Combinations of natural systems are often integrated to produce a particular effluent quality by utilising pollutant removal mechanisms specific to a particular system type [17]. The Irish landscape and population distribution is particularly suited to the implementation of natural systems. More than 42% of the population live in rural areas [18], and 71% of centralised treatment systems serve agglomerations of less than 2,000 PE (personal communication, 2015). Despite this, natural systems account for less than 0.5% of all WWTSs currently in operation in the country, the majority of which are constructed wetlands.

Constructed wetland systems utilise the natural treatment processes that occur in ground water, wetland vegetation, and soil. The removal of pollutants is achieved through a combination of microbial activity, vegetation filtration, and sedimentation. Constructed wetlands have excellent pollutant removal efficiencies and frequently achieve biochemical oxygen demand (BOD) and total suspended solids (TSS) removal greater than 95% [19]. Other studies have reported BOD and TSS removal rates of 99% [20].

Nitrogen removal in CW systems is achieved through a variety of pathways and is dependent on the CW type. Nitrification and denitrification are the primary nitrogen removal mechanisms for most types of wetlands (description provided in Appendix A). Vymazal [21] reported that total nitrogen (TN) reduction requires a combination of system types. The report stated that vertical flow constructed wetlands (VF-CW) provide the best option for nitrification but had a low capacity for denitrification. Conversely, horizontal flow constructed wetlands (HF-CW) have high and low capacities for denitrification and nitrification respectively. Therefore, for systems required to reduce TN a hybrid VF-HF system is proposed [22]. There are conflicting reports of phosphorus removal efficiencies in the literature. According to Kayranli et al. [19], consistent molybdate reactive phosphorus (MRP) removal rates of over 99% are being achieved at an ICW site in Ireland. However, the removal rates presented in the study were from the first year of operation. The other CW site in the study is older and reported a decline in MRP removal rates in the third and fourth years of operation, however, it was reported that this may have been due to overloading. In the study conducted by Costello [23] it was found that the average MRP percentage of TP ranged from 43.5% to 68 %, which may suggest that CW TP removal rates could be even lower. Vymazal [15] concluded that phosphorus retention in all types of CWs is low and that wetlands are generally not built with phosphorus as the main pollutant target. The survey of 386 FWS CW carried out by Vymazal reported an average TP removal efficiency of just less than 40%. Lüderitz and Gerlach [24] reported lower P removal rates for VF-CW of 27%, but reported 99% P removal with HF-CW that had iron filings added to the filter material.

2.3.1.1 Land requirements

A primary limiting factor involved in the selection of CWs is the large surface area requirements. Constructed wetlands are reported as being ideally suited to small, rural, decentralised communities [25]. Some studies have suggested agglomerations sizes of less than 12,000 PE [15]; however, there are larger systems in operation that exceed this value [26]. The required land, availability, and cost of land, is central to discussions of constructed wetlands. Sizing of CWs is normally based on organic loading and required effluent quality. Table 2-4 presents loading rates and required surface areas for three CW types achieving final effluent BOD of less than 25 mg/l [27]. The VF-CW has the obvious advantage of a lower required surface area in BOD removal only scenarios. However, for nutrient removal, it is evident that the required surface area is dependent on the range of nutrients to be removed. This creates a direct link between the system's discharge limits and its cost.

Table 2-4: Surface area requirements for FWS, HF and VF CW systems

CW type	*Required effluent BOD (mg/l)*	*Recommended BOD loading (g/m^2)*	*Surface area (m^2/PE)*	*Reference*
Free surface water	25	3	20	[27]
Sub-surface Horizontal flow	30	6	10	[27]
Sub-surface Vertical flow	25	20	3	[27]

2.3.1.2 Summary

Natural WWTSs are low energy consumers, with minimal OPEX when compared with conventional electro-mechanical systems. The main issue with their implementation is the large surface area that is required. Constructed wetlands have demonstrated reliability and good BOD and TSS removal rates. High levels of nutrient removal can be achieved through combinations of systems with specific substrate removal mechanisms. However, high levels of P removal may require additional material. The expertise needed to operate and maintain CW systems is minimal, which makes them particularly suited to rural, decentralised locations.

2.3.2 Electro-mechanical systems

Conventional electro-mechanical WWTPs are, in general, material and energy intensive when compared with natural systems. The complexity of the system may change depending on the size of the plant and the desired effluent quality. The generic WWTP layout presented in Figure 2-1 represents the most common system configuration for medium to large-scale WWTPs.

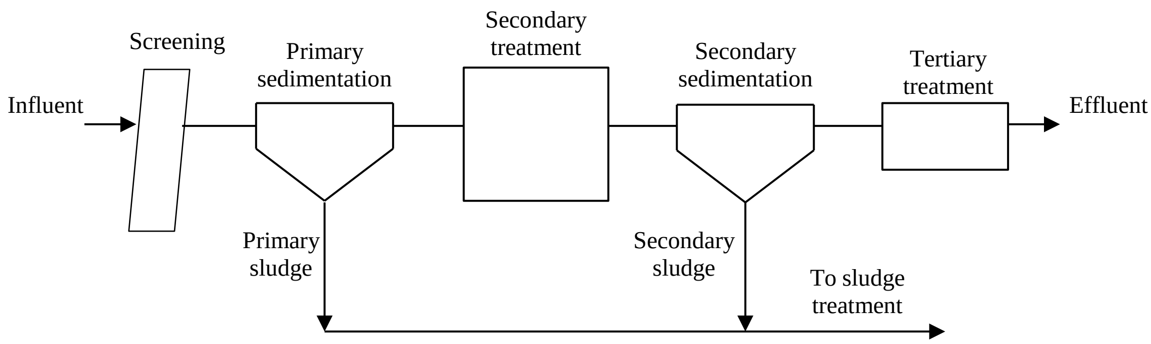

Figure 2-1: Generic wastewater treatment plant layout

Wastewater influent is screened as it enters the system to remove large debris (plastics, rags) that may cause damage to downstream processes. Screen designs vary from simple manually cleaned fixed-bar screens, to mechanically driven rake type or drum type screens. Primary treatment, also referred to as primary settling, is the earliest form of wastewater treatment. Up until 1992, when the U.S. Clean Water Act was introduced, primary treatment was the main WWT process in the United States. The objective of primary treatment is to remove the readily settleable suspended solids (SS) from the wastewater through gravity separation. Around 50 - 70% of SS and 25 - 30% of biochemical oxygen demand (BOD) can be removed with primary treatment [28]. Smaller plants may choose to omit the primary treatment stage and rely on inlet-works and screening. However, this can increase the loading to the secondary process, which can lead to an increase in energy consumption. Furthermore, there is a risk of inert materials being carried through to aeration tanks in CAS systems, which can have an adverse effect on particular aeration diffusers. The most significant and variable unit process within conventional electro-mechanical WWTSs is the secondary treatment process. Secondary treatment is generally a biological process that falls under one of three categories: suspended growth (activated sludge), attached growth (biofilm), or hybrid.

2.3.2.1 Suspended growth

It is widely accepted that the introduction of the activated sludge process took place on the 3rd of April 1914 with a presentation to the Society of Chemical Industry by Edward Arden and William Lockett [29]. The process involves the use of microorganisms to stabilise the organic content of wastewater. Primary treatment effluent flows into an aeration tank that hosts a mass of heterotrophic bacteria referred to as *activated sludge* or *mixed liquor* [28]. The activated sludge needs a continuous supply of oxygen to complete the stabilisation process and maintain solids suspension in the tank. Aeration can be achieved by submerged diffusers, surface aerators, or mechanical mixing, or by a combination of methods. After a period of contact between the wastewater and the activated sludge, the bacteria form flocs that are readily settleable. The bacterial flocs then flow into secondary settlement tanks where they are removed from the effluent by gravity separation. Depending on the discharge limits, suspended growth systems can be configured to achieve different levels of effluent quality. Variations of AS systems are too numerous to discuss individually. The following sections give a brief overview of common configurations for carbon, ammonia, total nitrogen, and phosphorus removal.

2.3.2.1.1 Conventional activated sludge – carbon removal

Figure 2-2 presents the basic BOD removal conventional activated sludge (CAS) configuration. The most significant elements of CAS systems in terms of economic and environmental cost are the energy consumed by the oxygen delivery systems and the sludge produced. For plants that require BOD and TSS removal only, solid retention time (SRT) can be kept to a minimum. This will result in large quantities of wasted sludge, but will avoid nitrification and reduce energy demand. Conversely, ammonia removal is achieved with CAS systems by increasing SRT. Endogenous decay will reduce sludge volume, but the increase in SRT will increase energy demand.

There is some evidence to suggest that simultaneous nitrification and denitrification is achievable in single stage CAS systems, but the results have been mixed [30], and high TN removal rates generally require a separate anoxic zone. Phosphorus removal in single stage CAS systems is only achievable with chemical precipitation.

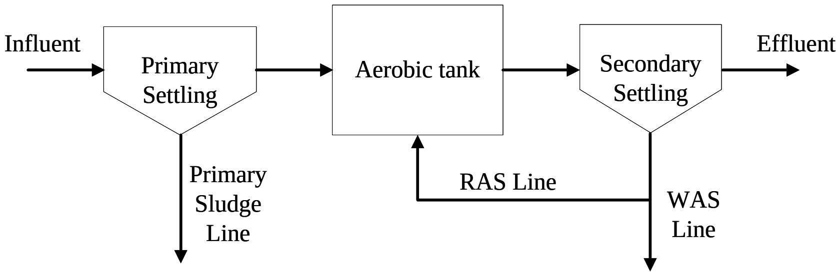

Figure 2-2: Basic CAS configuration

2.3.2.1.2 Anoxic oxic – total nitrogen removal

The anoxic-oxic (AO) configuration is used to achieve denitrification when TN reduction is required. Anoxic zones can be positioned post-anoxic or pre-anoxic. Pre-anoxic zone configurations [also referred to as the *modified Ludzack-Ettinger process* (MLE)] (Figure 2-3) are more common because the influent substrate can be used as a carbon source for denitrification, whereas a post-anoxic zone configuration may require the addition of an external carbon source. The use of a pre-anoxic zone can also reduce aeration energy demands. During denitrification, oxygen is released from nitrogen compounds in the anoxic tank prior to aeration.

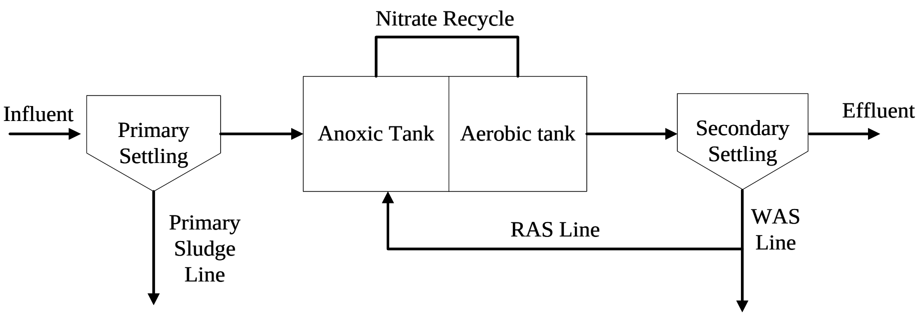

Figure 2-3: The AO system is used when denitrification is required

2.3.2.1.3 Anaerobic anoxic oxic – phosphorus removal

The process of removing phosphorus from wastewater through biological means is referred to as enhanced biological phosphorus removal (EBPR). Phosphorus accumulating organisms (PAOs) have an advantage over heterotrophic bacteria in anaerobic conditions because they are able to consume rbCOD (ready biodegradable chemical oxygen demand) in the form of volatile fatty acids (VFA) using energy from stored phosphorus, whereas heterotrophic bacteria require an electron acceptor in the form of oxygen, nitrate or nitrite to consume rbCOD [28]. Conventional activated sludge systems can be configured to include an anaerobic zone. A typical AAO system layout is presented below (Figure 2-4). The anaerobic tank is positioned prior to the anoxic zone. A portion of the flow (30 – 50% of flowrate) [31] is returned from the secondary settling to the anaerobic tank. The nitrate recycle line is maintained between the aerobic tank and the anoxic tank (100 – 300% of flowrate). Reports of achievable EPBR effluent phosphorus concentrations vary in the literature from < 1 mg/l [28] to < 0.3 mg/l [32]. Phosphorus limits below 0.5 mg/l generally require the addition of chemical coagulants such as ferric chloride.

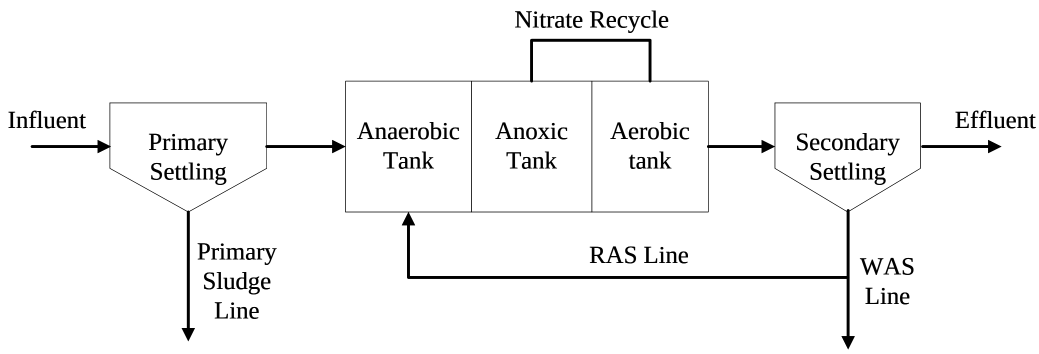

Figure 2-4: The AAO system is used for biological phosphorus and nitrogen removal

2.3.2.1.4 Extended aeration

Extended aeration (EA) is a particular variation of the activated sludge process that uses long SRTs and high MLSS concentrations (3000 to 6000 mg/l) to achieve high quality effluent. The extended SRTs (20 – 40 days) result in the destruction of most of the sludge with the remainder consisting of inert or non-biodegradable material. The process is particularly suited to treating small volumes of wastewater where strict final effluent discharge limitations are required, but is often used in large-scale installations. Many EA systems omit primary sedimentation but employ significant pre-treatment (fine mesh screening, maceration, and grit removal). The aeration tank is much larger than CAS systems to allow for the longer SRTs. Ideally, the aeration tank should be large enough so as not to exceed loading rates greater than 650 g BOD/day/m^3. Nitrification will occur naturally with long SRTs, and denitrification can be achieved with cyclical aeration regimes, or through tank design. Oxidation ditches (OD) are a specific configuration of EA developed in the Netherlands by Pasveer in 1953 [33]. Pasveer's design was simple and inexpensive. Primary treatment is not required and simultaneous nitrification and denitrification can be achieved in a single unit. The Orbal design and continuous fluid motion promotes the growth of ammonia oxidising bacteria (AOB), nitrite-oxidising bacteria (NOB), and phosphate accumulating organisms (PAO) at different stages of the cycle (Figure 2-5). Aeration is usually achieved with rotary aerators that provide oxygen transfer and maintain fluid motion around the tank.

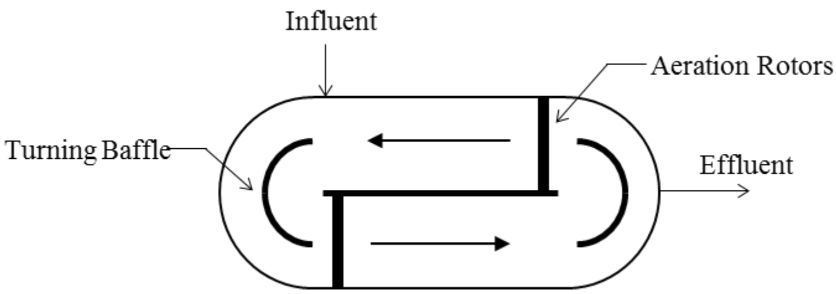

Figure 2-5: Pasveer type oxidation ditch

Many variations of Pasveer's OD have been developed since the original design. The design presented below (Figure 2-6) consists of concentric racetrack type channels enclosing secondary sedimentation tanks in the centre [34]. Aeration is provided with rotating perforated discs as per conventional RBC systems, which also serves as mixers and maintain fluid motion. The outer channel is the largest in volume and carries out the function of primary treatment. The influent flows from one channel to the next through interconnected ports designed in such a way that there can be no short-circuiting of flow directly across a channel. The versatility of the process is due to the arrangement of channels acting as sub-compartments. This allows the plant to be configured as a complete-mix or stepped aeration system. Nitrification is also achievable in compartmentalised systems such as these. Orbal systems can also be scaled up easily with additional outer channels.

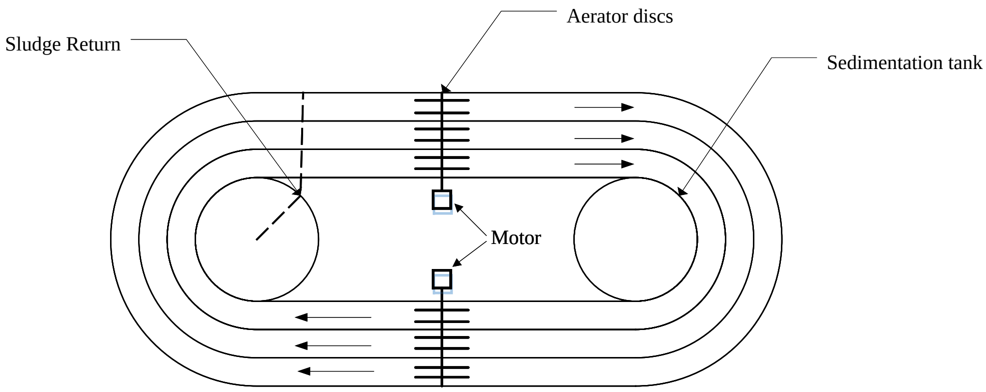

Figure 2-6: Orbal EA configuration with concentric channels

2.3.2.2 Attached growth

One of the fundamental differences between suspended-growth and attached-growth systems is the method by which oxygen is transferred to the microorganisms. In suspended growth systems, energy intensive blowers deliver oxygen to free-moving bacteria, while in traditional attached growth systems, the bacteria form a biofilm on a fixed growth media that is exposed to atmospheric air. Two of the oldest and well-established attached growth systems are trickling filters and rotating biological contactors.

2.3.2.2.1 Trickling filters

Trickling filters are one of the oldest forms of fixed-film or fixed-growth biological reactors. The process was born out of research carried out in the Lawrence Experimental Station in the United States in the late 1800s [28], and was used extensively in the first half of the 20th century [35]. The first reported application of a TF for use in a large centralised system was in the United States in 1908 [36]. The process and a number of variations of the process are still widely used today.

A tricking filter is a non-submerged fixed film biological reactor [28]. Although the process is described as a *filtration*, there is no actual physical filtration [37]. The removal of pollutants from the influent is achieved through biological degradation. Influent that has passed through a primary sedimentation or pre-treatment stage is distributed evenly over a biological growth medium. The wastewater trickles down slowly through the growth medium where it comes into contact with the microorganisms that breakdown the organic matter (Figure 2-8).

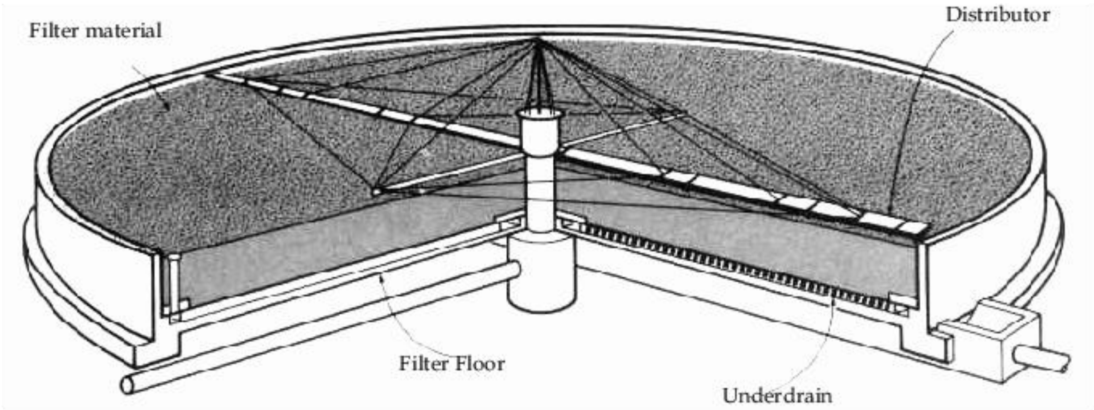

Figure 2-7: Basic trickling filter design [38]

The treated wastewater is collected in an underground drainage system where it is transferred to secondary settling. The filter material typically used in early models was rock (slag) or redwood. However, the use of redwood as a growth medium has decreased in recent years. The development of synthetic materials for use as a growth medium has enhanced the performance and removal efficiency of trickling filters. Biotowers that use light synthetic materials can be built much higher than traditional rock based systems. This means that TF footprints can be reduced for locations where surface area is an issue. Figure 2-8 presents a basic TF system configuration. Variations of TF system configuration include TF + AS [39], 2-stage TFs for high strength wastewaters where nitrification is required [40], and TFs used for tertiary treatment. High levels of nitrification are possible with single stage TF systems by controlling recycling ratios. The biofilm in the top 0.6 - 1.2 m depth of growth media is primarily responsible for BOD removal. As the wastewater travels down below this depth the nitrifying bacteria begin to thrive in lower soluble BOD (sBOD) concentrations. The sBOD loading rate is a limiting factor for nitrification. Akker et al. [41] reported a 55% decrease in nitrification when the sBOD loading was increased from 0.75 – 2.1 g sBOD/m^{2}d. The sBOD concentrations can be reduced by increasing recycling ratios, but this will require increasing capacity and will increase energy demand.

The main energy sink in TF systems is the pumping system. In hydrostatic circular TF systems the distributor arms are propelled by the force of the wastewater as it is expelled through the nozzles. This needs to be a continuous process because of minimum required wetting rates, problems with pests, and in lower temperatures to avoid freezing and loss of biomass.

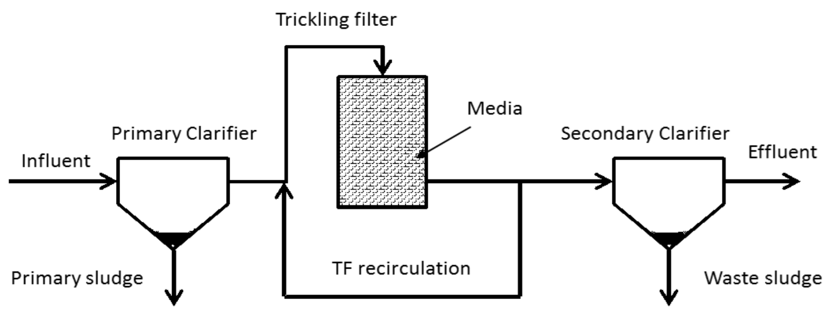

Figure 2-8: Basic trickling filter system configuration

2.3.2.2.2 Rotating biological contactors

In RBC systems, the bacteria form a biofilm on one, or a series of closely spaced, shaft-mounted rotating discs (Figure 2-9). Contact between the bacteria and the substrate takes place in a biological reactor. Substrate oxidation occurs through passive aeration as the discs rotate out of the influent and the biofilm is exposed to atmospheric air. The discs are partially submerged in the wastewater influent, usually to a depth of about 40% of the disc diameter.

Design configurations for RBC systems will vary depending on the scale of the treatment plant and on the desired effluent quality. Staging is a very important design specification that can be defined as the compartmentalisation of individual RBC units for increasing substrate removal rates. Each stage has different microbial growth characteristics with variations in biofilm thickness and growth rate. Studies have shown that a four-stage system can produce a higher quality effluent than a two-stage system having the same overall surface area [42].

The process consists of a *train* of RBC units or *stages* mimicking a plug-flow system and avoiding any potential short-circuiting of the wastewater stream. The BOD removal efficiency is at its highest in the first stage and decreases through subsequent stages. The percentage values of BOD removal in the first stage will vary depending on loading and operating parameters but is generally about 50% [43]. For smaller systems, a single RBC shaft positioned parallel to the direction of flow can be divided into individual stages by introducing baffles at desired intervals (Figure 2-9). For larger systems, it is common practice to arrange the disc shafts perpendicular to the direction of flow (Figure 2-10). For BOD removal only, two to four stages may be required depending on the final effluent requirements.

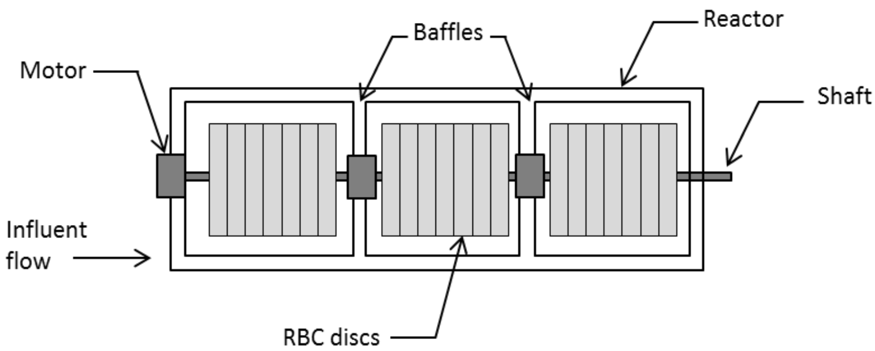

Figure 2-9: Small RBC systems with baffle configuration

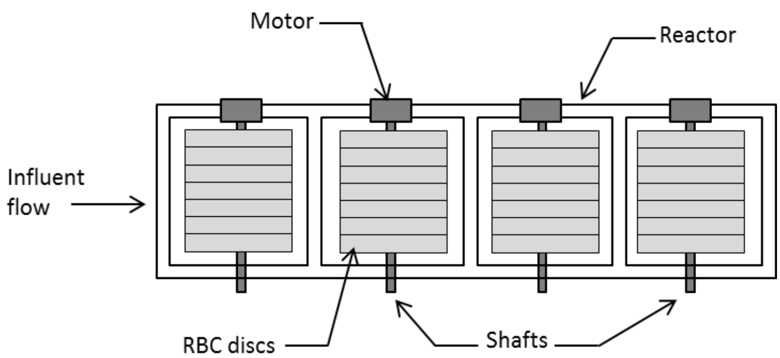

Figure 2-10: Large systems with individual shafts perpendicular to the direction of flow

Treatment plants that are required to reduce ammonia can achieve nitrification by the addition of several stages. The number of stages required will depend on the ammonia discharge limit and the concentration of sBOD in successive stages. Various studies have been carried out to assess the effective staging for ammonia removal. Lin and Shackleford [44] assessed a five-stage system and concluded that the majority of the 83% ammonia removal efficiency occurred in stages three and four and practically no nitrification took place in stage five. Denitrification with RBC systems is reported to be achievable with full submergence of the discs in the wastewater and the addition of an external carbon source. Gupta et al. [45] reported successful simultaneous nitrification/denitrification in high strength synthetic wastewater by the introduction of a sulphur oxidising bacterium (*Thiosphaera Pantotropha*) with the capacity for heterotrophic nitrification and aerobic denitrification. The study found that there was no need for an external carbon source for denitrification. However, although ammonia and TN removal efficiencies were good (90-99% and 49-82% respectively), the final effluent concentrations were high (30 mg NH_3/l, and 19 – 27 mg TN/l).

2.3.2.3 Hybrid systems

The term *hybrid system* can be used to describe any combination of treatment processes. The purpose of hybrid systems is to utilise the strengths of specific processes together in one system with the aim of achieving a particular quality of final effluent. In the TF + AS configuration mentioned previously, a TF removes the bulk of the sBOD from the influent, which lowers the aeration energy requirement of the AS process [39]. Electro-mechanical processes can be combined with natural processes. Upton et al. [46] demonstrated excellent BOD and ammonia removal rates using a RBC/reed-bed hybrid. These types of hybrid systems involve process combinations in series with one another. The following section examines three *integrated* hybrid systems.

2.3.2.3.1 Membrane bioreactors

Membrane bioreactors (MBRs) are the combination of the CAS process and a crossflow membrane-filtration (micro or ultra) loop. The development of MBR systems for treating municipal wastewater began over 30 years ago [47]. The basic concept of the MBR process is that solids separation is accomplished through filtration rather than traditional gravity settling methods. Earlier versions of the systems involved a separate stand-alone filtration unit that was external to the aeration tank, but these were energy intensive systems [48]. In 1989, Yamamoto et al. [49] designed a system with the membrane directly submerged in the aeration tank (Figure 2-11). This integrated configuration was found to be more energy favourable (> 80% reduction in kWh/m^3 [50]). However, MBRs continue to be one of the most energy intensive systems currently in operation. Krzeminski et al. [51] reported energy consumption values ranging from 0.4 – 4.3 kWh/m^3. There are three main reasons for this: firstly, elevated MLSS concentrations (typically 8000 – 14,000 mg/l) mean that dissolved oxygen (DO) levels in MBR systems are generally higher than CAS systems [52]. Faust et al. [53] concluded that higher DO concentrations (4 mg/l) resulted in higher COD removal efficiencies, better flocculation and lower supernatant turbidity. Although Chen et al. [54] demonstrated that high COD removal efficiencies were achievable with DO concentrations below 1mg/l, the same issues related to flocculation were observed. Secondly, one of the main operational issues associated with MBR systems is membrane fouling. The contraflow air scouring methods used to prevent fouling are energy intensive. Krzeminski et al. [51] reported membrane cleaning energy values of 0.5 – 0.6 kWh/m^3 wastewater treated. Lastly, the suction head required to maintain flux across the membrane is an additional energy sink not found in CAS systems.

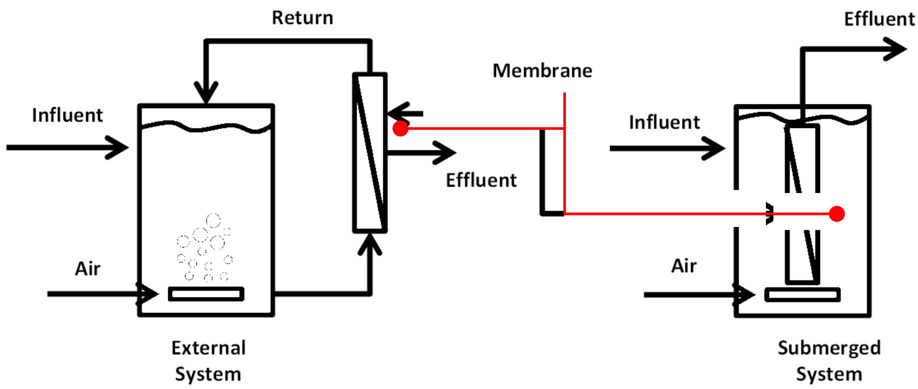

Figure 2-11: External and submerged MBR configurations

The compact structure of MBRs makes them suitable for locations with space restrictions. The high effluent quality eliminates the need for secondary clarification. Solids retention times in MBRs are much longer than CAS systems (15 - 45 days), and therefore, typically produce less sludge. Moreover, MBRs do not have the problem of poor sludge settleability associated with long SRTs in CAS systems. The initial construction costs for MBRs are higher than CAS systems. In the early days of their development, there were high costs associated with replacing the membranes, but these costs have now been largely reduced (Figure 2-12).

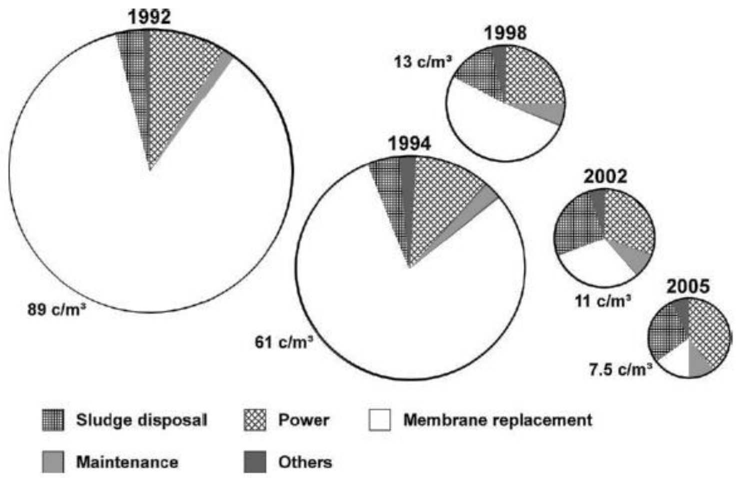

Figure 2-12: Distribution of MBR OPEX from 1992 to 2005. Adapted from [55]

Membrane bioreactor system configurations for biological nutrient removal (BNR) are similar to those of CAS systems, and can achieve very high nutrient removal rates. Low effluent suspended solids values achievable with MBR lower the particulate TN and total phosphorus. Galil et al. [56] reported effluent concentrations of 5.9 – 7.6 mg TN/l, 0.07 – 0.15 mg NH4+/l, and 0.4 – 2.3 mg TP/l, with an average value of 0.8 mg TP/l without the addition of coagulants.

2.3.2.3.2 Integrated fixed-film activated sludge

The concept of the IFAS system, as it is known in its current form, was introduced in the late 1990s [57]. The advantage of this system is that it provides the stability of fixed-film technology – in that it is more resistant to microbial washout - and the flexibility and removal efficiencies associated with CAS systems. The process is ideally suited to medium to high strength wastewaters in locations where surface area availability is an issue. The addition of a growth media is reported to have the capacity to provide an equivalent MLSS of up to six times that of CAS suspended growth [28], which reduces the required aeration tank volume. Existing CAS systems are often retrofitted with IFAS media when an ammonia discharge limit has been introduced, or where an agglomeration is experiencing a significant population increase.

Integrated fixed film activated sludge system variations are similar to CAS variations for different quality final effluents i.e. the inclusion of anoxic and anaerobic zones for TN and TP removal respectively.

There are two categories of media: fixed and dispersed. Fixed media systems consist of flexible fabric or PVC sheeting connected to rigid frames that are fixed to the aeration basin structure. These systems are relatively cheap to install and maintain. The PVC sheets in particular perform well, promote mixing and have good oxygen transfer to the biofilm. Dispersed media systems consist of a mass of sponge or plastic biofilm carriers dispersed in the aeration tank. The carriers are kept in suspension by the oxygen being supplied by floor-mounted diffusers. Air sparging is required to keep the carriers rotating around the tank and to avoid build up at the exit of the aeration tank. Air sparging also acts to control biofilm build-up on the growth media. Dispersed media systems require a sieve to restrain them in the aeration tank. They also require adequate pre-treatment as the media can suffer a loss of material due to abrasion from inert material.

Nitrification and denitrification in IFAS systems can be achieved in much the same way as CAS systems. The addition of the carrier media to the aerobic zone has been found to increase nitrification capacity and stability due to a greater percentage (>70%) of AOB and NOB residing on the carrier media. However, for TN reduction, it has been found that denitrifying bacteria are more likely to reside in the suspended mixed liquor [58]. Furthermore, difficulties related to mixing are introduced when anoxic zones are fitted with fixed-film media. Onnis-Hayden et al. [59] reported that in IFAS-EBPR systems, over 90% of EBPR activity takes place in the suspended mixed liquor, and concluded that it is possible to decouple conflicting SRT for phosphorus and nitrogen removal, allowing for greater SRT control and process optimisation.

2.3.2.3.3 Moving bed biofilm reactors

Moving bed biofilm reactor (MBBR) technology was developed in Norway in the late 1980s, early 1990s [60]. The systems operate in much the same way as IFAS systems and many of the design characteristics for media carrier specifications and retention sieves are the same. They also provide many of the same advantages such as low surface-area requirements, and enhanced process stability. However, there is no return activated sludge line in MBBR systems to maintain suspended microbial populations, which results in negligible MLSS concentrations (100 – 250 mg/l [61]). This reduces the level of expertise needed to operate the system, as the operator does not have to control SRTs, sludge wasting or recycling. One of the disadvantages of not having a RAS line is that while MBBR systems can achieve nitrification in the same mode as IFAS and CAS systems, denitrification must be a post-anoxic process, and will require an external carbon source for TN reductions below 3 mg/l [28].

2.3.3 Summary

Wastewater treatment systems currently in operation exhibit varying degrees of complexity and specific expertise requirements. These range from low input natural systems to more sophisticated hybrids that require specialised expertise, energy, and material input. Most systems can achieve high levels of BOD, COD and TSS removal, and can be configured to achieve good levels of ammonia removal, but at a significant operational cost increase. Constructed wetlands can achieve TN reduction with a hybrid HF-VF CW system. Total nitrogen reduction at electro-mechanical plants can be achieved through cyclical aeration, or with the addition of pre or post anoxic zones. Some suspended growth configurations such as the AAO system can achieve EBPR, but most other systems employ chemical P removal.

It is evident that the site-specific conditions under which systems are required to operate will affect their performance. Material, energy, and labour inputs required to reach desired final effluent quality can vary significantly depending on locational factors. This means that some WWTSs are more suited to given locations and conditions than others. It is, therefore, necessary to be able to evaluate system performance under changing conditions to make informed decisions on their possible implementation. The following sections review approaches and methods of system evaluation and selection.

2.4 Economic cost assessment

2.4.1 Introduction

It is widely accepted that the total economic cost of a given system is best determined by assessing both the capital and operational costs together over the entire life cycle of the system [62-64]. The following sections review life cycle costing methodologies and provide background to their development.

2.4.2 History and development

The term life cycle cost (LCC) was first introduced in 1965 in a report entitled '*Life Cycle Costing in Equipment Procurement'* [65]. The report was prepared for the U.S. Department of Defence who determined that the cost of system acquisition may be small in relation to the cost of ownership [66]. Dhillon [67] reported that the cost of system ownership could range from 10 to 100 times the cost of acquisition. This gave weight to the idea of compiling and analysing all associated costs over the lifetime of a system rather than basing procurement decisions solely on the initial bidding price. The concept of LCC introduced a new level of transparency to costing, and exposed hidden costs that were not immediately apparent with traditional costing methods. In his review of the LCC technique Harvey [68] described the LCC of an item as

> *"...the sum of all funds expended in support of the item from its conception and fabrication through its operation to the end of its useful life".*

This approach makes it possible to determine the most cost effective solution amongst a range of alternatives by considering all cash flows over the lifetime of the system, and allows practitioners to identify potential trade-offs between initial capital investment costs and long-term cost savings. Woodward et al. [64] state that

> *"LCC is concerned with quantifying different options so as to ensure the adoption of the optimum asset configuration."*

Flanagan and Norman [69] determined that the four main objectives of LCC are

- to enable objective options to be more effectively evaluated;
- to consider the impact of all costs rather than only initial capital costs;
- to assist in the effective management of completed buildings and projects; and
- to facilitate choice between competing alternatives.

Despite the apparent benefits of LCC, the concept has had varying degrees of implementation. Research in the U.S. found that only 40% of administrations applied LCC to construction projects [63]. In Europe, the adoption of the practice has been varied. The Swedish building industry reported that 66% of the countries' building industry employed LCC [70], while in Finland the figure is only 5% [71]. Since 1988, the Norwegian process of public procurement has been subject to the NS 3454 standard '*Life-Cycle Costs for Buildings and Civil Work, Principles and Classification*', which details procedures for life-cycle costs and economic evaluation [72]. In the United Kingdom, the British Standards Institute (BSI) and the British Cost Information Service (BCIS) issued a standardised method for the application of LCC in the construction industry. In Ireland the Capital Works Management Framework (CWMF) makes reference to the importance of LCC stating that it '*should be integrated at every stage in cost plan development*', but does not outline any details or methodology for its implementation [73]. Since the conception of LCC, there have been many who advocate that the practice, and generally agree that the application, of LCC at an early design stage will result in better system design and operation [69]. However, there are others who question the cost-benefit credentials, and claim that the level of detail required and the extent of the LCC model can result in the process being 'overcomplicated and laborious' [74]. The U.S National Research Council (U.S. NRC) [75] concluded that

> *'One of the most difficult problems is the shortage of reliable information on historical costs and performance, which is needed for accurate estimation of costs.'*

While there is little doubt that LCC data acquisition is a significant challenge, it is difficult to envisage alternatives that offer the same level of completeness or transparency.

2.4.3 LCC Procedure

Since its conception, several LCC procedures have been developed; however, to date, standardisation has been limited to a subsection of ISO 15686 (2008) [76], pertaining to the LCC procedure for building and constructed assets. One of the earliest LCC procedures was the simple four-step approach proposed by Harvey [68].

1. Define the cost elements of interest
2. Define the cost structure to be implemented
3. Establish the cost estimating relationships
4. Establish the method of LCC formulation.

The *cost elements* are the cash flows that occur over the life of the system. The *cost structure* describes the allocation of costs into groups i.e. engineering and development, construction, operation, disposal/salvage. The *cost estimating relationships* are the mathematical relationships between cost and a given parameter. Finally, the *establishment of the method* of LCC refers to the choice of the most appropriate method. The procedure presented by Harvey is generic and can be broadly applied to most costing problems. Greene and Shaw [77] proposed the procedure presented below (Figure 2-13). Two of the additional and key stages in this procedure are the *sanity check* of inputs and outputs, and the *sensitivity analysis and risk assessment stages*. Although presented as separate stages, input-output and sensitivity analyses could be included under the heading of *inventory analysis*. These two steps are closely linked due to uncertainties that may exist in both the input quantities and the associated specific costs, and should be carried out in parallel. This is particularly significant for processes with large material and energy inventories.

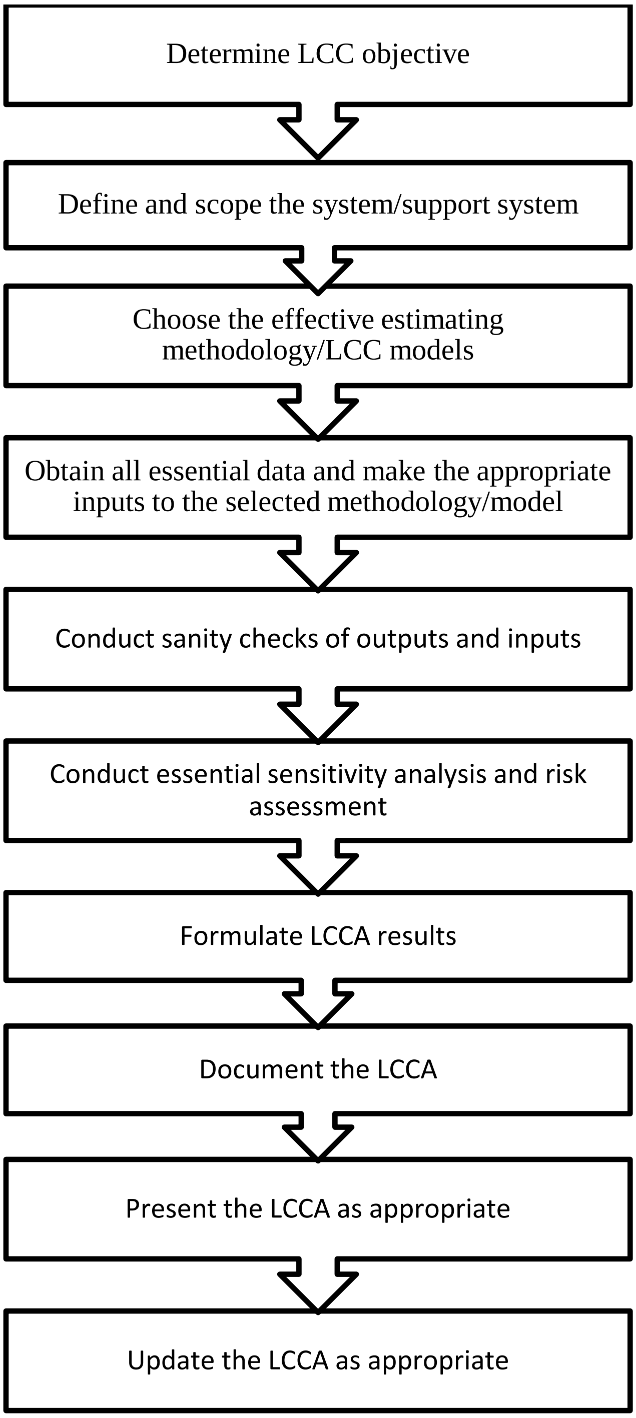

Figure 2-13: Life cycle costing procedure developed by Greene and Shaw [77]

2.4.4 Life cycle cost methodology

2.4.4.1 Present value

Costs that occur at different times in the future cannot be compared directly because of changes in the time value of money, and, therefore, must be calculated to represent their value at a common base date. This approach provides a platform for a fair evaluation of alternatives. The adjusted value is commonly referred to as the present value (PV). Present values are calculated by applying a discount rate *d*, to the future value FV, which occurs *n* years in the future. The basic formula is presented below (Eq. 1) [74]. The term $\frac{FV}{(1+d)^n}$ is commonly referred to as the single payment present worth factor (SPPWF).

$$PV = \frac{FV}{(1+d)^n} \qquad \text{(Eq.1)}$$

The Task Group 4 (TG4) report commissioned by the EU [78] adopted Eq. 2 for calculating the accumulated future costs in construction projects.

$$NPV = \sum \frac{C_n}{(1+d)^n} \qquad \text{(Eq. 2)}$$

Where C_n is the cash flow occurring in year *n*. This formula is referred to as the net present value (NPV) formula. In situations where systems experience a single recurring cost (A_O), over a particular time period (*n*), the uniform present value (UPV) formula (Eq. 3) can be used to calculate the present value of the accumulated cost.

$$UPV = A_O \frac{(1+d)^n - 1}{d(1+d)^n} \qquad \text{(Eq. 3)}$$

There are other LCC models for specific variations of cash flow such as the uniform gradient present worth (UGPW) method that is used to account for regular payments that increase or decrease by a fixed amount. Combinations of these formulae are often used when carrying out

LCCs as there are generally a variety of cash flow types within the economic structure of an asset or system. There are other methods of economic evaluation such as the equivalent annual cost (EAC) [74]. The EAC method estimates the cost of owning and operating a system or asset over its lifetime, and assumes that the system will be replaced by an identical system. The discount payback period (DPP) method calculates the length of time is will take for the investment cash flows to equal its costs. The critical variable common to most of these methods is the discount rate.

2.4.4.2 Discounting

It is important to understand the difference between the *discount rate* and *the rate of inflation*. The discount rate represents the time value of money, whereas the rate of inflation describes the decrease in purchasing power and increase in operating costs. There are two types of discount rate used in NPV calculations: the real discount rate and the nominal discount rate. The main difference between the two is that the nominal discount rate accounts for inflation and deflation, whereas the real discount rate does not. The choice of discount rate to be used will depend on the purpose of the costing exercise. If the purpose of the LCC is to estimate the actual cash flow it is important to include interest rates, and thus, adopt a nominal discount rate. However, if the purpose of the LCC is to compare alternative systems then the real discount rate is usually sufficient. The most commonly adopted discount rate in the literature is 3.5%. The Irish National Development Finance Agency (NDFA) currently recommend using a nominal discount rate of 3.96% for projects lifetimes of between 10 and 20 years, and suggest a 5% test discount rate (TDR), which is a real discount rate, for use in cost benefit analysis (CBA) [79]. One further consideration in relation to discount rates in NPV calculations is the use of multiple rates. The NPV model presented by the American Society for Testing and Materials (ASTM) [80] includes a separate discount rate for energy, which makes sense considering volatility in oil markets, advancements in energy saving technologies, and a fundamental change in attitudes towards energy use. It is also particularly applicable to LCCs of energy intensive systems such as electro-mechanical wastewater treatment.

2.4.5 Life cycle costing and wastewater treatment

The application of LCCA to WWTSs is particularly appropriate because of the significant cost variability that exists between different locations. Individual systems may have different CAPEX and OPEX profiles depending on location, and therefore, should be assessed on a case by case basis. The significance of this was recognised at an early stage by the U.S. EPA. Shortly after the introduction of the U.S. Water Act, the EPA commissioned a series of reports to examine several aspects of WWT cost, such as '*Operation and Maintenance Costs of Municipal Wastewater Treatment Plants'* [81], '*Estimating Sludge Management Costs'* [82]; and '*Biological Nutrient Removal Processes and Costs'* [83], they also published '*A guide to the selection of; Cost-Effective Wastewater Treatment Systems'* [84]. The EU adopted similar measures in commissioning studies to assess WWTP operational costs [85], and sludge management alternatives [86]. The study conducted by Foes et al. [87] could be considered one of the seminal pieces of research regarding WWT life cycle costing. The report was entitled '*Cost and Performance Evaluation of BNR Processes',* and consisted of a compilation of CAPEX and OPEX for nine WWTS alternatives. Included in the analysis was the uniform annual cost (UAC) of each system. Although system selection was ultimately determined by a weighting mechanism that included both quantitative and qualitative criteria, it was apparent that if UAC had been the basis for system selection, the results would have been different from those where CAPEX or OPEX had been the selection criterion. Gratziou et al. [88] carried out an assessment of small WWTSs using the LCC method and found that in most scenarios natural systems were the optimum choice in locations where land availability and cost were not an issue. Similar findings were presented later by Rawal and Duggal [89] in their LCC evaluations of TF, AS and WSP systems. Lim et al. [90] recognised the potential of LCC to identify trade-offs within a single WWTS where a reduction of cost in one area can result in an increase in another and that these trade-offs need to be optimised to reduce the total sum. The study successfully applied the LCC methodology as a tool for developing a mathematical optimisation model for total wastewater treatment network system (TWTNS).

2.4.6 System lifetime

The projected lifetime of WWT systems will vary between different system types. This is an area that is often overlooked, and values used in LCC studies vary widely in the literature from 20 [88, 91] to 40 years [88]. According to Sundaravadivel and Vigneswaren [92], the ‘highly technical and mechanical nature of concrete and steel’ used in construction of conventional WWTPs results in system lifetimes of less than 25 to 30 years. It could be argued that for the purpose of system comparison a single lifetime value will suffice; however, it should also be noted that systems with large OPEX will be more sensitive to variations in the nominal lifetime value.

2.4.6.1 Capital expenditure

Wastewater treatment project CAPEX refers to the cost of the initial investment in materials, planning, construction, engineering, electrical and mechanical equipment. Some literature may include the cost of land acquisition, and there is generally a 15 – 20% contingency included to account for uncertainty. Table 2-5 outlines the general capital cost breakdown from a cohort of of surveyed WWT projects [93], and highlights the large variability that exists in the overall CAPEX profile. The type of treatment system being considered will, to a large extent, determine the CAPEX distribution profile. Systems that require large structures such as EA, OD and TF will incur higher construction costs. Complex hybrid systems such as IFAS, MBRs and MBBRs will have higher specialised material and labour costs. Natural systems such as CWs will have a much greater civil works cost than conventional electro-mechanical systems due to the large surface areas involved. The location of the potential site can have a large influence over several areas of cost. For example, the distance to suppliers, availability of labour, access to utilities (water, electricity, gas) will vary by location, and will inevitably affect cost. The cost of civil works can rise depending on the site topography and soil geology. There can be costly legal challenges from public or private interest groups. Proximity to residential areas can result in additional investment in expensive odour and noise restriction equipment.

Table 2-5: Breakdown of typical new-build WWTP capital expenditure [93]

Cost type	Percentage of total CAPEX (excluding land cost and infrastructure)	Description
1 Preparation		
Site acquisition	Not included	Building site, legal fees, public relations
Infrastructure	Not included	Access roads, sewer lines and effluent discharge pipelines, power supply
Site preparation	0.5 – 2%	Demolishing, ground work, rerouting pipes & cables
2 Construction		
Civil	23 – 29%	Construction of concrete structures – tanks, buildings.
Mechanical	21 – 27%	Process plant e.g. aerators, pumps
Electrical	10 – 16%	Motors, process-specific technical electrics
Piping	2 – 5%	Sewers, utilities, tracing
Process control	2 – 5%	Control units, software installation, substation, cabling
Contingency	10 – 20%	Unforeseen costs
3 Start up		
Equipment	1 - 3%	Maintenance and lab equipment
Start-up supplies		Chemicals, first fills (activated carbon, filter material). Fittings, cables.
Personnel		Hiring and training employees
4 Additional		
Initial studies	10 – 20%	Feasibility study, system selection, soil survey
Design and engineering		Design and engineering inputs, revisions, procurement
Project management		Planning and budget control
Construction management		Site supervision, testing and commissioning
Miscellaneous		Permits, insurance

The combination of these issues presents a significant challenge to providing accurate estimations. According to [28], there are three levels of accuracy that can be achieved:

- At the highest level (lowest accuracy), *order of magnitude* estimates can be attained from cost curves and published project costs
- Budget level estimates can be derived from historical bid information and manufacturers' quotations
- Generally, the highest level of accuracy is attained from a detailed bill of quantities. However, estimations of this nature are laborious and time-consuming and contractors are generally reluctant to undertake them unless there is a realistic potential for sale.

2.4.6.2 Operation and maintenance expenditure

Although the type of technology chosen will generally dictate OPEX distribution, it is the location of the treatment plant that will ultimately determine the type of treatment technology that should be used. This is based on the predication that the most appropriate system will be chosen for a given location. Figure 2-14 presents a comparison of the OPEX profiles of three different activated sludge systems [upflow anaerobic sludge blanket (UASB) + TF, CAS, and EA] operating in three different regions [94]. It is unclear whether the variation in the OPEX distribution presented here is due to the type of system, or location. The large labour cost for the UASB system in Brazil could be attributed to the lack of available local expertise. The large energy cost for the EA plant in Tunisia is likely due to the heavy aeration demand. Finally, the large sludge management cost at the German plant could be due to stricter sludge disposal regulations in Europe, or to the culture of sludge incineration in Germany (up to 55% of total sludge disposal in 2011), which is a more expensive disposal option. The main point here is that the total OPEX, and OPEX distribution profile of a WWTS will vary because of location-related factors.

Typical OPEX profiles are dominated by four main cost components: energy, chemicals, labour and sludge disposal (maintenance is often accounted for under labour and replacement materials). Depending on the system type, these four cost elements can account for up to 90% of the total OPEX in electro-mechanical systems [85, 95].

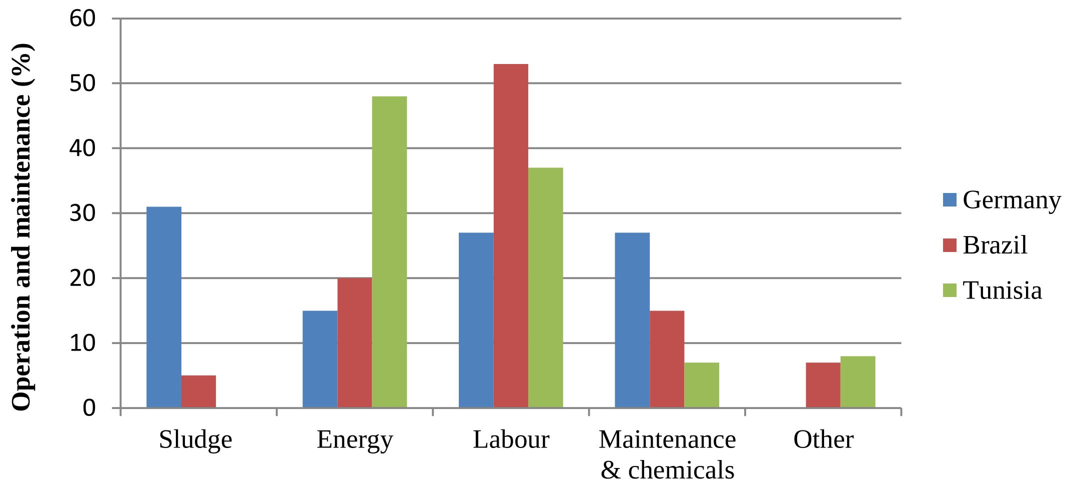

Figure 2-14: Operation and maintenance expenditure profile comparison of three different treatment systems in three different regions

2.4.6.2.1 Labour

Properly trained and skilled personnel are essential for WWTP operational efficiency [96]. In a study carried out by Hegg et al. [97], 30 WWTPs were evaluated to determine the factors affecting plant performance. It was found that the top two factors limiting performance were:

1. Operator application of concepts and testing to process control
2. Wastewater treatment understanding.

According to the New England Interstate Water Pollution Control Commission (NEIWPCC), in the 1999 review of the 104(g) program[2], the U.S EPA found that inadequate staffing was third in the top five causes of WWTP compliance failure in the United States [98].

Kemper et al. [99] reported that the ratio of labour costs to overall OPEX is much lower in EU countries when compared with some less developed countries globally (Figure 2-15). It is difficult to disaggregate the contributing causes of this. It may be attributed in part to a scarcity of experienced, technical professionals, necessitating the import of more expensive foreign personnel. The level of automation and control of European systems may be higher than in

[2] The 104(g) (1) is a section of the Clean Water Act (CWA) in the U.S. The "Wastewater Operator Training Program" was set up specifically to assist small community WWTPs to achieve compliance with regulatory requirements.

some developing nations. It may simply be that other operational costs such as energy, chemicals, and sludge disposal are lower than they are in Europe.

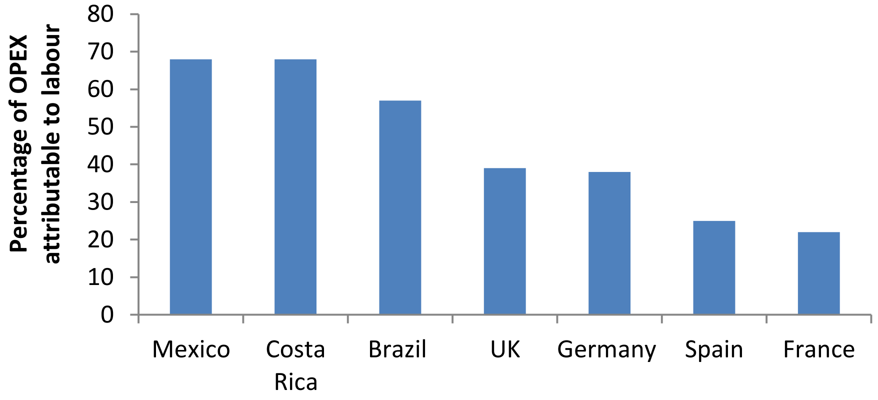

Figure 2-15: Labour cost to OPEX ratio (adapted from [99])

The percentage of OPEX attributable to labour is reported to be higher for small WWTPs (Table 2-6) [100]. It is conceivable that the labour percentage will rise even further as the plant size falls below 2,000 PE and optimisation of labour resources becomes more challenging. For small plants that are manned infrequently, the ratio of hours spent travelling to and from the plant, to hours spent operating a plant increases.

Table 2-6: Percentage of OPEX attributable to labour for a range of plant sizes [100]

PE	*Percentage of OPEX attributable to labour*
< 10,000	35 - 40
10,000 – 100,000	25
> 100,000	15

2.4.6.2.2 Sludge management

Sludge management is a central issue in WWTP operation because of the high cost of treatment and disposal. Population growth and the implementation of the UWWTD have resulted in an increase in sludge quantities in Europe. In 2010, the quantity of sewage sludge produced in the EU exceeded 10 million tonnes. The European Commission (EC) has been active in trying to manage the impact on human health and the environment, and has published a number of directives concerning sludge disposal [101-103]. The EC has also commissioned research related to the economics of sludge management [86, 104]. Traditionally, methods of sludge disposal in Europe have been land spreading, incineration, or landfill. The EU directive on the landfilling of waste (1991/31/EEC) recommends a reduction in the quantities of sewage sludge going to landfills [105]. In Germany, landfilling with sludge is prohibited unless in the form of ash from incineration, and in some countries such as Sweden the practice has been banned completely since 2005 [106]. In 2003, Irish WWTPs with agglomerations greater than 500 PE collectively produced 42,298 DS tonnes sludge, 63% of which was recycled for agricultural use, and 35% sent to landfill, and 8% to incineration [107]. Land spreading reported at a cost of just over €150/tonne DS (dry solids) is the least expensive method of disposal in the Europe Union and accounts for over 75% of sludge disposed of in countries such as Portugal and the United Kingdom [108]. However, this practice may change as regulations relating to land spreading become more stringent and drive up costs. Incineration is the primary sludge disposal route in countries such as Malta, and Bosnia and Herzegovina at a cost of just under €250/t DS [108]. Figure 2-16 presents the costs of common sludge disposal methods in the European Union [109].

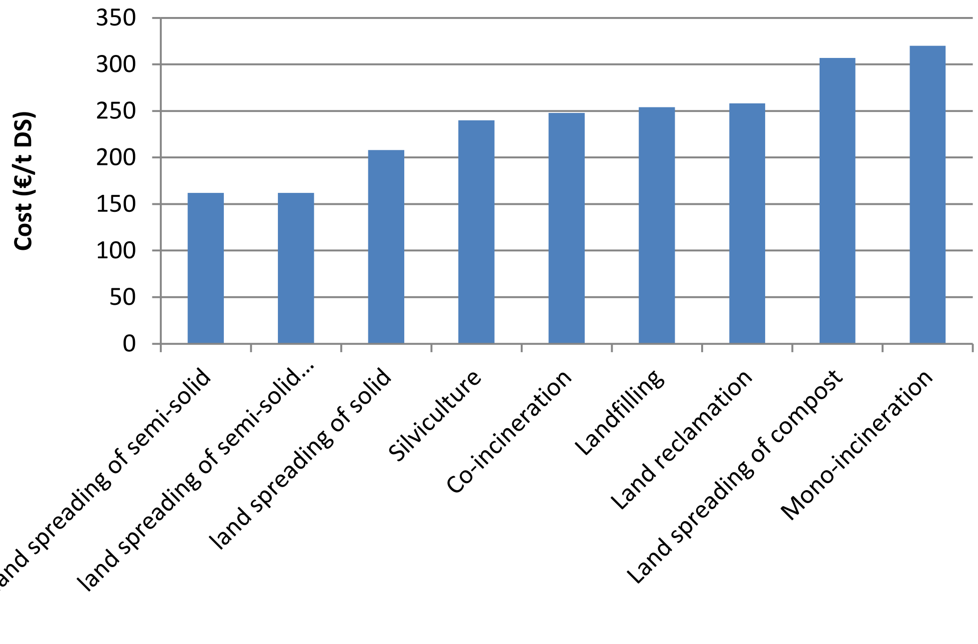

Figure 2-16: Cost comparison of sludge recycling and disposal routes

The cost of sludge management can be divided broadly into two categories: treatment cost, and disposal cost. In Europe, treated sludge is defined as

> "…*having undergone biological, chemical or heat treatment, long-term storage, or any other appropriate process so as to significantly reduce fermentability and any health hazards resulting from its use"* [110].

Whichever the method of treatment, the associated costs for process plant, materials, labour and energy are significant. The percentage of OPEX attributable to sludge treatment and disposal depends on plant size, location, and disposal or recycling route. Sludge cost will vary with plant location because of the disposal options that are available and their relative distance. The size of a treatment plant will ultimately dictate whether it is economically feasible to treat sludge on-site. For large plants, the input costs (materials, energy and labour) can be offset by energy gained from anaerobic digestion (AD) [111]. It has been reported by Caldwell [112] that AD could potentially generate enough energy to meet the demand necessary to operate the entire system, and that there may even be a net-positive energy production. However, this claim has been disputed by Gude [113] who argues that AD of municipal wastewater sludge alone cannot achieve net-positive energy production, and states that current systems are producing a maximum of 50% of the energy required, and only at large scale plants. However, Gude does state that net positive energy production is achievable with co-digestion of municipal sludge with food, brewery or dairy wastes.

In addition to any potential energy that may be gained from AD, the quality of the treated sludge may be of a high enough standard to be sold as a biosolid [114]. In the year 2000, farms in the UK were incurring mineral fertilizer costs of £0.36 /kg N and £0.26 /kg P_2O_5. In 2011, costs rose to £1.00 /kg N, £0.93 /kg P_2O_5 [115]. Hence, for large WWTPs, in addition to generating energy to operate the system, the end products of the sludge treatment process could provide a source of revenue, or at least offset a percentage of the environmental impact by reducing synthetic fertiliser production. The sludge treatment economics for smaller plants is very different. Anaerobic digestion requires a minimum feedstock for economically feasible operation. Because of the high capital and operational costs associated with AD, it has been estimated that a minimum agglomeration of 40,000 PE is required '*in order to realise energetic benefits within a reasonable time horizon*' [116]. Therefore, treatment plants below this agglomeration size will not only lose out on any energetic benefits to be gained from AD, but also will have the extra cost associated with higher sludge volumes. Furthermore, as plant sizes reduce, new questions over investment in sludge treatment processes begin to arise. There are obvious gains to be had from investment in thickening and dewatering equipment, as an increase of 1% in sludge dry solids concentration results in a 50% reduction in sludge volume. In Ireland, the cost of sludge disposal by contractor ranges from €45/m^3 to €75/m^3 for digested sludge and €60/m^3 to €90/m^3 for undigested sludge (Enva Ireland[3], sales representative, personal communication, November 15, 2016). However, at a certain WWTP scale, the capital investment required for sludge thickening and dewatering equipment when added to the additional OPEX in energy, labour, and chemicals, will outweigh the reduction in sludge handling costs to a point where it becomes more economical to outsource sludge treatment and disposal to an external contractor as opposed to treating sludge onsite.

2.4.6.2.3 Energy

Reports on the percentage of OPEX attributed to energy consumption vary widely in the literature, and can range from 0 – 60% depending on system type [117]. The specific energy consumption values for different treatment systems vary from 0 kWh/m^3 for ICW systems [118] to over 4 kWh/m^3 for MBR systems [51]. Because of the minimal energy used by natural systems, the remainder of the discussion here will be limited to electro-mechanical systems.

The total energy cost and distribution across processes within a WWTS will vary with system type, scale, location, hydraulic, organic and inorganic load, discharge limits, and operational efficiency. A typical WWTP energy distribution profile for an activated sludge system is presented below in Figure 2-17 [28]. The profile presented here is typical of medium to large-

[3] Enva is a waste management company in Ireland that provides sludge stabilisation and disposal services.

scale systems. Small systems may not include units for sludge thickening and dewatering, or primary clarifiers. It would also be unlikely that 7% of the energy would be used for heating.

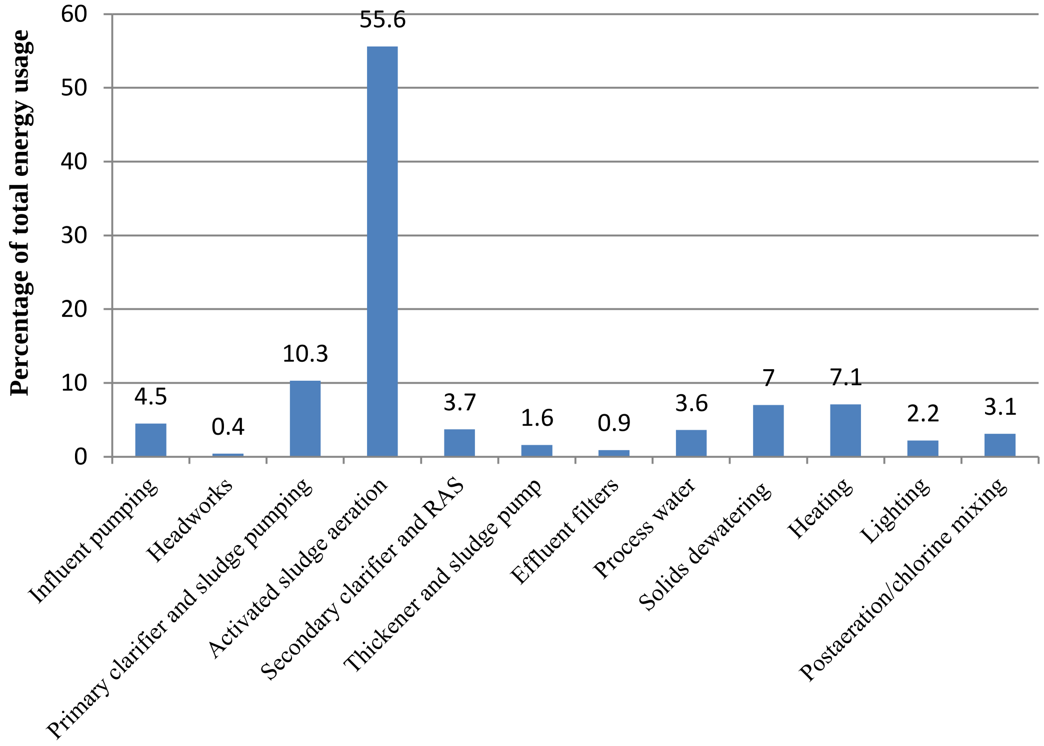

Figure 2-17: Typical energy distribution profile for an activated sludge system [28]

The scale of a WWTP can have an effect on specific energy consumption. Economies of scale have been widely reported throughout the literature [119]. However, there is little reported about the apparent causes of these economies. Aeration systems used in the activated sludge process exhibit a reduction in specific energy consumption with increases in flowrate, because system components such as motors and pumps generally exhibit higher efficiencies with increased capacity [120]. Furthermore, motors and pumps operate more efficiently when their size is matched correctly to their loading requirements [121]; therefore, large variations in hydraulic load will reduce efficiency. This is particularly relevant to small systems because the relative magnitude of hydraulic load variations increase with decreasing plant size (Figure 2-18) [28].

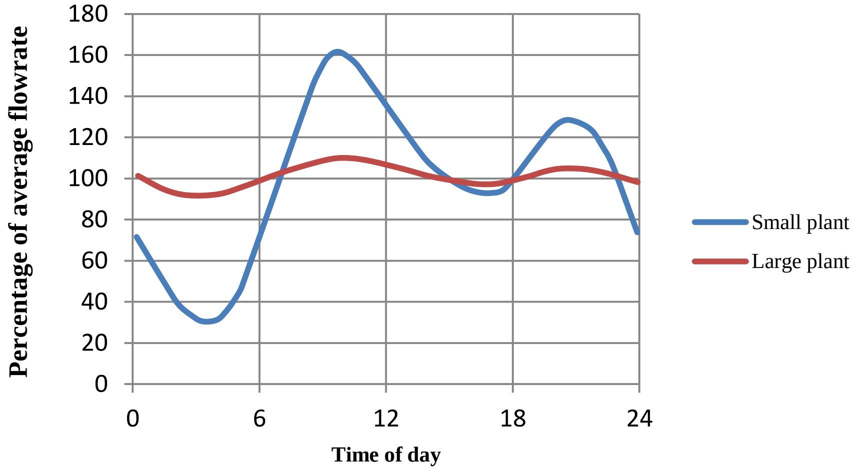

Figure 2-18: Comparison of the percentage variation in hydraulic loading between large and small [28]

Larger pipe diameters produce less fluid frictional drag [122], and increased aeration tank depths will have better oxygen transfer efficiencies due to extended bubble-substrate contact time [123]. Activated sludge aeration energy values reported by [28] range from 0.12 – 0.23 kWh/m^3, while values reported by Foladori et al. [124] for systems below 10,000 PE range from 0.68 – 0.79 kWh/m^3. Figure 2-19 presents energy consumption for activated sludge systems as a function of influent flowrate [119]. It can be seen here that there is a significant increase in the rate of change of energy use with respect to flowrate below 5,000 m^3/day.

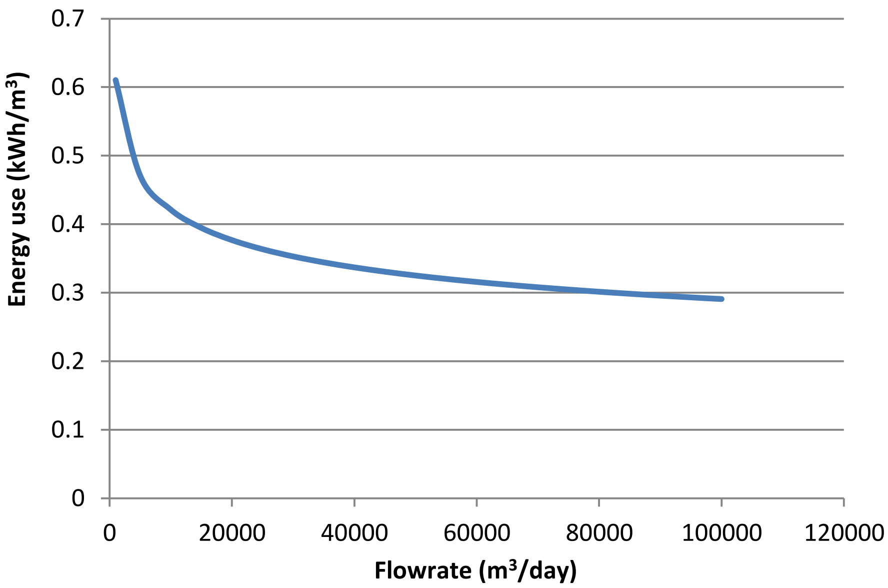

Figure 2-19: Typical energy use as a function of flowrate for activated sludge system [119]

Final effluent discharge limits will affect energy consumption in a number of ways. For activated sludge systems, low BOD limits require a longer solids retention time (SRT). In EA systems, oxygen demand can reach up to 1.5 kg O_2/kg BOD removed. The oxygen demand for oxidation of ammonia to nitrate is 4.6 kg O_2/kg NH_3 removed. Therefore, an ammonia reduction requirement can increase O_2 demand by up to 300%. Denitrification in AO systems will require power for mixing and additional pumping for the nitrate recycle line. In trickling filter (TF) systems, pumping energy increases when ammonia removal is required due to increases in recycling rates. Rotating biological contact systems require several additional stages in the process train to remove ammonia, which requires additional disc rotational power. In many small systems, phosphorous removal is achieved through chemical precipitation with additional energy requirements for dosing pumps.

Operational efficiency can have an impact on energy consumption. Preventative maintenance schedules on system components such as motors, pumps, blowers and diffuser heads will improve performance and energy efficiency. Cost savings can be achieved by taking advantage of off-peak energy rates. Energy rates in Ireland can vary by over 80% in a single day (max price €197.01/MWh – min price €36.06/MWh, Dec. 2016 [125]). However, diurnal flow patterns tend to mimic energy utility system demand; that is, the peak flows into a WWTP occur at the same time as peak energy demand [126]. This may necessitate additional influent and sludge storage to defer treatment times until off-peak hours. Other cost saving measures such as the installation of variable frequency drives (VFDs) and load balancers on pumps and blowers have been found to reduce energy consumption by up to 30% [127]. Reducing SRTs will reduce energy consumption in situations where nitrification is not a requirement, but this needs to be weighed up against the cost of additional sludge handling.

2.4.6.2.4 Chemicals

The specific cost of chemicals will vary with plant location and supplier. Chemical quantities are heavily influenced by the plants' discharge limits. Plants with low phosphorus limits (< 2mg/l) will require the addition of chemicals. The principal chemicals used are aluminium chloride ($AlCl_3$), ferric chloride ($FeCl_3$) and calcium hydroxide ($Ca(OH)_2$); however, the addition of calcium hydroxide can require recarbonation of the fluid stream to reduce the pH value. Because of the additional cost involved with using calcium hydroxide, metal salts are generally the preferred option [28]. An inexpensive alternative for phosphorus precipitation is the use of pickle liquor. Spent pickle liquor is a by-product of the steel making and metal finishing industry. Due to the high metal content of this waste product disposal can be difficult and costly; therefore, sending it for use in WWT is beneficial for both parties [128]. This is a cheap alternative to other phosphorous precipitation compounds [129]. However, there will be an additional oxygen demand in the aeration basin to oxidise ferrous ions to ferric ions before it reacts with the phosphate ions (Eq. 4-5), thus, any savings made may be slightly offset by an increase in energy costs.

$$2Fe^{2+}+O_2 \rightarrow 2Fe^{3+}+O_2^{2-} \qquad \text{(Eq. 4)}$$

$$Fe^{3+}+PO_4^{3-} \rightarrow FePO_4 \qquad \text{(Eq. 5)}$$

In addition, pickle liquor can sometimes introduce metal contaminants into the sludge line; therefore, sludge quality needs to be monitored for adverse effects, or otherwise the costs are being transferred rather than reduced. The precipitation performance of the pickle liquor is quite poor and the phosphorus removal efficiency is 70%, which means that much higher molar dosages are required than for virgin ferric chloride.

Systems with sludge treatment processes may require sludge conditioning chemicals. Up to the 1970s, metal salts addition followed by $Ca(OH)_2$ was widely used for sludge thickening and dewatering [114]. However, in recent times, organic polyelectrolytes (polymers) have become more popular because they are easier to handle, require less space for administration, produce better sludge densities, but can more expensive than inorganic conditioners [130]. The addition of $Ca(OH)_2$ also serves a secondary role of sludge stabilisation. Larger plants may have anaerobic digesters for stabilisation, but for smaller plants this is not economically feasible and lime stabilisation is generally preferred.

Treatment plants with post-anoxic systems or weak influent wastewater may need additional carbon from an external carbon source. Historically, methanol and ethanol have been the carbon source of choice, although there are a number of other options such as corn syrup or molasses that may not provide the same rates of denitrification, but are easier and safer to handle.

2.5 Environmental cost assessment

The economic cost associated with implementing a given system is generally the primary concern for business in both public and private sectors. However, over the last half century the concept of sustainability has grown from being simply, a good idea, to being fully integrated into design standards and management ethos. Sustainability may not carry the same weight of importance in product design specifications (PDS) as robustness or reliability, but it can be a powerful marketing tool in societies with a sense of environmental awareness. In parallel with the emergence of environmental thinking there has been an evolution of the tools and methods needed to assess product or system sustainability, and environmental impact. In the 1980s, Burton and White [131] advocated the use of *environmental risk assessment* (ERA) to assess not only the consequence of an environmental hazard, but also societal attitude towards risk. Early ERA models tended to focus on the immediate regional impact of single substances, with secondary consideration being given to upstream and downstream interventions; however, subsequent studies have looked to address this issue by providing frameworks for the inclusion of a more holistic global impact assessment [132]. *Environmental impact assessment* (EIA) is defined by the International Association for Impact Assessment (IAIA) [133] as

> *"..a process of identifying, predicting, evaluating, and mitigating the biophysical, social, and other relevant effects of proposed projects or plans and physical activities prior to major decisions and commitments being made"*

Environmental impact assessment and ERA are often used interchangeably. The main difference between them is the scope of the assessment, where the EIA scope extends to assess the wider social and environmental impact of a project, and in many cases ERA is used as a supplement to EIA. The e*cological footprint* methodology is limited to measuring resource depletion by area of wilderness or natural capital required to supply a system's energy and materials, and sequester its emissions [134]. *Cumulative energy demand* (CED) is one of the oldest forms of environmental impact assessment [135]. The CED represents the total energy content of all resources used in a product or system over its entire life cycle. Although often overlooked as an environmental assessment tool because of the focus on energy [136], this can be a relief for non-technical commissioners of environmental impact studies who may find that other methodologies produce subjective and over-complicated results. However, CED does not account for the impact of waste streams. To overcome this limitation several authors have proposed the use of e*xergy analysis* (EA) as a method of measuring both resource use and waste emissions [134, 137]. Ayres [138] postulates that '*thermodynamics offers a means of accounting both for resources and wastes in a systematic and uniform way*'. Exergy is a thermodynamic property defined by Moran et al. [139] as

> "..*the maximum theoretical work obtainable from an overall system consisting of a system and the environment as the system comes into equilibrium with the environment*"

In many cases the reference environment is the surrounding, or natural environment. It is therefore, possible to calculate the exergy (both physical and chemical) of any waste stream. The principle being that the greater the magnitude of the exergy value, the further the state of the system is from equilibrium with the surrounding environment and thus, the greater the environmental impact. Furthermore, all natural resources have an intrinsic exergy value. Therefore, it is possible to produce a single aggregated value for the exergy of both the natural resources used, and the waste emissions. However, there are other EA practitioners who do not believe that EA is suitable for environmental applications. Gaudreau et al. [140] make reference to inconsistencies and contradictions related to reference environment formulation and question whether it is appropriate to apply thermodynamic analysis to non-thermodynamic properties such as scarcity.

Life cycle assessment (LCA) is an analytical tool that provides a holistic approach to assessing the environmental performance of a product or system from cradle to grave [141] (Figure 2-20). The LCA concept encapsulates many of the methods employed by the previously mentioned environmental assessment tools. The LCA methodology has been widely accepted as a valid environmental assessment tool for government, local authorities, and areas of the private sector

[142]. The application of LCA to WWTS is particularly appropriate due to the nature of the relationship between a plant's technosphere (sphere or realm containing processes controlled by humans) and the surrounding ecosphere (sphere containing naturally occurring processes).

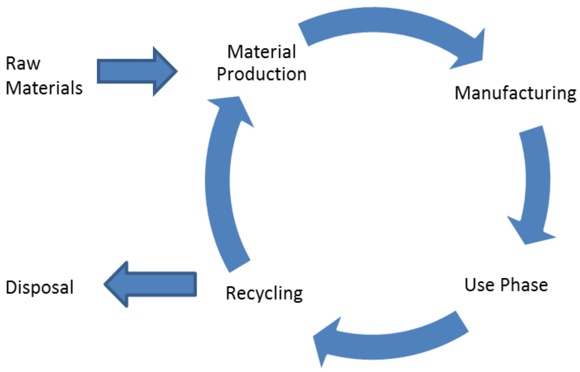

Figure 2-20: Life cycle of a product or system

2.5.1 History and development of life cycle assessment

Life Cycle Assessment (LCA) found its roots in the late 1960s. There is a general acceptance that the Coca-Cola Company was the first to carry out a full LCA study. The company was examining the feasibility of manufacturing its own drinks containers and was looking at alternatives to the traditional glass bottles. A study was conducted by Darney, Hunt and Franklin [143], in which one of the main outcomes was that the company had acquired a scientifically robust defence to any negative public perception on the use of plastic as an alternative to glass. At the same time in the UK, Dr. Ian Boustead had carried out his own research into the energy consumption of beverage containers manufactured from a variety of materials, and in 1979 published the "*Handbook of Industrial Energy Analysis*" [144]. In the United States, between the years 1970 and 1975, the process of analysing energy, resource use, and environmental emissions was referred to as *Resource and Environmental Profile Analysis* (REPA). In Europe, the process was called *Ecobalance*. The term LCA was not defined until 1991 and the first scientific journal on LCA was not published until 1996 [145]. It was during the period from 1990 to 1993 that a series of workshops conducted by the Society for Environmental Toxicology and Chemistry (SETAC) that the LCA methodology and framework began to take shape. The result of these workshops was the 1993 Code of Practice which formed the basis of the ISO 14040 series of standards pertaining to life cycle assessment [146-149].

2.5.2 Life cycle assessment and wastewater treatment

The application of LCA to a wastewater treatment plant or system was first reported in The Netherlands in 1997. A study was conducted by Roeleveld *et al*. [150] to examine *the sustainability of municipal wastewater treatment*. The study concluded that improvements in the environmental performance of WWT should focus on minimizing effluent discharge pollutants and sludge production, and that the impact from energy consumption was negligible. In Spain, Gallego et al. [151] concluded that the impact from energy production was one of the main contributors to a system's overall environmental profile. The disparity between studies highlights an important aspect of LCA interpretation. The Roeleveld study placed greater emphasis on regional terrestrial and aquatic impact which may be more significant in a water rich landscape such as The Netherlands. Energy generation has a much greater influence on global impact categories such as global warming and acidification. Spain is the most arid country in the EU and is more susceptible to rising temperatures resulting from the GHG emissions associated with electricity production. Most contemporary studies agree that electricity production provides the largest potential for environmental impact. Pasqualino et al. [3] concluded that *"The highest environmental impacts of the water line are due to the energy consuming equipment"* and recommended *"reducing energy consumption, use energy efficiently, and use more renewable forms of energy."* However, the environmental impact resulting from energy use will vary between countries because the magnitude of impact is not only dependant on the amount of energy used, but also on the method of energy generation. For example, the impact from electricity generation in Norway where over 90% is hydroelectric power will be much less than that of Italy where over 60% of electricity is generated from fossil fuels [152].

Assertions in the literature as to the sludge disposal method with the least environmental impact will vary between countries due to specific regional sensitivities. In Spain, the study conducted by Pasqualino *et al*. [3] examined sludge composting and disposal to a cement plant, and concluded that landfilling was the least desirable option in all impact categories except for acidification and eutrophication. The EU Directive on the landfilling of waste (1991/31/EEC) recommends a reduction in the quantities of sewage sludge going to landfills [105]. In Germany, landfilling with sludge is prohibited unless in the form of ash from incineration, and in some countries such as Sweden the practice has been banned completely since 2005 [106]. Houillon and Jolliet [153] found that incineration in fluidised beds and agricultural spreading are the best choice based on energy and global warming balance, but stress that it is impossible to draw conclusions on the global environmental impact without including other impact categories. Lundin et al. [154] expanded the impact assessment of sludge disposal to a wider range of impact categories and found that incineration had environmental restrictions, but agreed that land application was the least favoured method. Suh and Rousseaux [155] were among the few that found land application to have a better environmental profile than the other alternatives.

Since the first study by Roeleveld *et al*. [150], there have been over forty LCA WWT studies of published in peer-reviewed journals [156]. These studies covered a variety of objectives which included assessing changes in system configuration [157], variations in boundaries and scale [158], structural changes [159], and competing technologies [160]. In recent times, there has been a paradigm shift in environmental assessment of treatment systems from considering not only water quality and human health, but also energy and resource recovery [156].

2.5.3 Limitations

In some sectors of industry there can be a level of scepticism surrounding the results of an LCA depending on background of the group involved. Public scepticism is often borne out of misunderstanding of the methods and aims of life cycle assessment. A common assertion is that companies are "cooking their books", by setting their own boundaries, choosing their own methodologies and indicators which make their product or system seem more environmentally favourable [142]. Product comparison can be contentious with some claims that the LCA process lacks transparency, that data are inconsistent, or that it is too confusing for non-scientific professionals [161]. However, even within the scientific community there is a degree of discord over the interpretation of LCA results. An area of particular concern amongst LCA practitioners is the reporting of variability and uncertainty. In a review carried out by Stuart et al. [162] regarding how LCA uncertainty is dealt with, less than 50% of the studies that were examined reported any uncertainty. Of those who had reported uncertainty, only 3% made reference to any quantitative analysis of uncertainty thresholds, and only 7% reported carrying out any qualitative analysis. The study concluded that while LCA can effectively assess resource use and efficiency, uncertainties must be made transparent to policy makers, and that there should be at least a qualitative description of uncertainty and variability.

The sources of uncertainty and variability are numerous and have been well documented throughout the literature. In general, variability in LCA comes from variations in the natural world i.e. temporal and spatial variability, whereas uncertainties can come from a number of sources such as choice of functional unit and boundaries, model assumptions, lack of site-specific data and inaccurate measurements. Uncertainties due to choices that have to be made in LCA are unavoidable as there are several at the start of every project: the type of study; the extent of boundaries; time horizons of emissions; and the LCIA methodology. The choice of functional unit can introduce a degree of uncertainty to an LCA study. The problem with environmental loadings being expressed per a single functional unit is that there is no information about existing background concentration of emissions, nor is there any temporal information included [163]. Practitioners conducting LCAs of WWTPs often choose volume per time unit, e.g. m^3/day of influent treated [164], but this metric does not consider influent constituents.

It is important to highlight some of the limitations of the LCA methodology. Moreover, it is incumbent on LCA practitioners to provide as much clarity on assumptions, uncertainty, and variability as is practicable. It should also be understood that the extent, range and quantity of data required for an LCA means that there will always be a degree of uncertainty, but this has to be weighed up against the value of the information that is being provided. Guinee et al. [165] state that "*The core characteristic of LCA is its holistic nature, which is both its major strength and, at the same time, its limitation. The broad scope of analysing the complete life cycle of a product can only be achieved at the expense of simplifying other aspects*".

2.6 System selection

2.6.1 Introduction

Methods of selecting the most appropriate WWTS have evolved over time. Original system selection was often based solely on the required initial capital investment. Whichever system that could achieve the required results for the lowest cost would generally be one that was chosen. Over time it became apparent that the costs associated with operating a given system could outweigh the initial investment costs and would require due consideration during the selection process. More criteria were also being considered such as the expertise required to operate a system, the land requirements, and in more recent times the system's environmental performance. System selection became a more complex problem and required a new approach that could integrate multiple objectives and criteria into a single decision making process.

2.6.2 Multi criteria decision making

The application of the *multiple attribute decision-making* (MADM) method to WWTS selection was originally conceived by Tecle et al. [166]. In this approach, a selection of treatment systems was assessed using *non-dominated solution* and *game theoretic* concepts. The criteria included level of influent pollution, required effluent quality, capital and operational costs, reliability, compatibility, flexibility, resilience, manpower and land use. The criteria are assigned weightings and combined to provide a single score for each system. The three MADM techniques used in the study produced consistent recommendations. The criteria did not include specific environmental factors, but this may have had more to do with the time of the study (1988) when sustainability was not at the forefront of many of the modern design specifications that are present today. Capital and operational expenditure factors were treated individually in the methodology, which makes understanding the actual total cost more difficult. An analytical hierarchy process (AHP) was adopted by Ellis and Tang [167] . In this method, a hierarchy model for system selection was developed with data gathered from several WWTPs. An extensive set of criteria was used to evaluate a selection of treatment alternatives. The criteria used in the study included many of those presented by Tecle et al [166], but also included were several subjective, qualitative criteria such as “ability of local administration to adequately support the work's operation” and “willingness and enthusiasm of community/politicians to improve the existing wastewater treatment facilities.” Qualitative parameters such as these can be difficult to assess for several reasons. The weighting of these types of criteria is opinion based, and can be subject to small temporal variations. Public opinion can change very quickly in reaction to a negative event such as a water contamination or a bathing restriction. Similar to the study by Tecle et al. [166], capital and operational costs are treated separately and there is no reference to environmental sustainability. This is particularly relevant because of the absence of a sludge treatment criterion overlooks the impact that sludge treatment/disposal can have on a system’s economic and environmental inventories. The multi-criteria decision analysis (MCDA) technique proposed by Rawal and Duggal [89] addresses WWTS economics from a life cycle perspective with the application of present value (PV) methods, but fails to include any other non-economic criteria.

2.6.3 Whole life cycle costing

There are different interpretations of the term ‘whole life cycle costing’ (WLCC). The term traditionally referred to the practice of considering both the LCC and LCA of a project. Nogueira et al. [168] proposed a parallel economic and environmental assessment approach to WWTS selection. Unlike previous evaluation methods that attempt to combine criteria through a weighting mechanism, the economic and environmental factors are analysed separately but in parallel with each other. To illustrate the method, an LCA was carried out for three alternative systems; in conjunction, investment and operational cost functions were developed. However, the combination of the two fell short of more recent formats of life cycle cost analysis. Pretel et al. [169] went further by conducting a full LCA and LCC using the PV method to assess alternative systems during high influent loading. As with the Nogueira study, both the economic and environmental assessments were treated separately.

In recent times, the scope of WLCC has been extended to include additional indirect costs, or externalities, that are often qualitative and difficult to include in a performance or cost evaluation (Figure 2-21). Societal factors such as public acceptance, visual appearance, or community benefit are examples of externalities that are often included in the scope of whole life cycle costing.

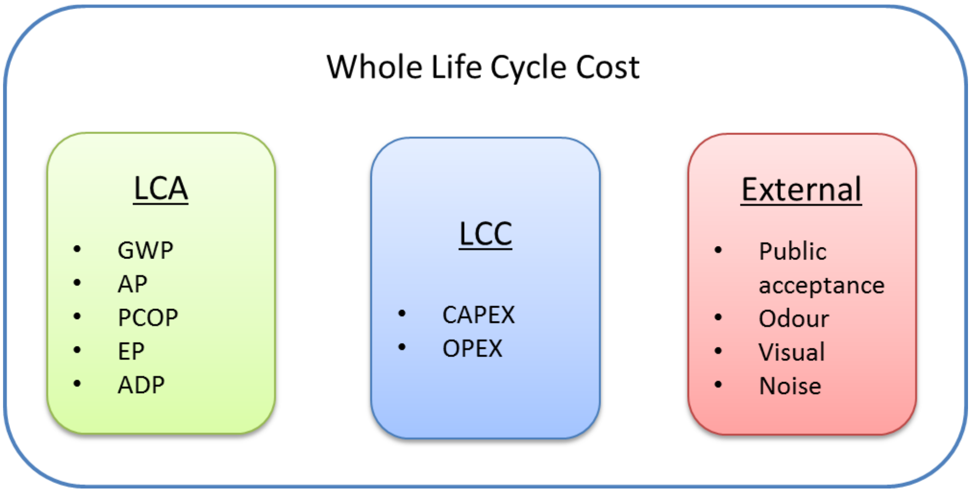

Figure 2-21: Whole life cycle cost of wastewater treatment systems

Building on previous work, an innovative approach was developed by Pradip et al. [170] to address the problem of WWTS selection in India. The methodology presented is MADM based; however, unlike the aforementioned methodologies that use a list of criteria, this method includes the six specific scenarios most commonly found in India. Each scenario has three levels of information. The first level defines the location type: urban, sub-urban and rural. The next level provides a choice between locations with and without land restrictions, and lastly between systems that discharge to a water body and systems that require water reuse. The six scenarios are then evaluated with a set of weighted criteria. The criteria include life cycle costs presented as net present worth (NPW), land requirement, and LCA is accounted for with global warming and eutrophication inventories. There are a number of qualitative criteria such as reliability, durability and acceptability. The main issue with the application of this methodology in Ireland is that, as it will be shown in this study, small variations in scale, loading and discharge limits can have a large effect on the economic and environmental performance of a system. The variations of these factors in WWTSs in Ireland are too numerous to be defined under a limited selection of scenarios. In other words, systems selection in the Irish landscape requires a methodology that allows the input of more detailed site-specific information.

2.6.4 Summary

Methods proposed to determine the most appropriate WWTS vary in complexity from basic economic evaluation to WLCC that includes economic, environmental, and social factors. Some of the economic evaluation methods treat capital and operational costs individually. This approach may be misguided as these two entities may not be mutually exclusive and should be considered together to gain a true and transparent indication of the actual economic cost. Similar considerations need to be given to the environmental costs associated with a given system. In much the same way that trade-offs can exist between capital and operational expenditure for a given system in a given scenario, so too can trade-offs exist between environmental impact categories. Quite often attempts to reduce a system's contribution to impact in one category can result in an increased contribution to another. It is, therefore, necessary to evaluate the full environmental profile of a system to fully understand the environmental consequence associated with its implementation.

Economic life cycle cost analysis and environmental life cycle assessment provide a rational framework for the performance evaluation of wastewater treatment plants and systems. The strength of both analytical tools is the extent to which material and energy flows of a system are considered. This allows for the exposition of costs and environmental consequences that may not be immediately apparent with other assessment tools. Potential trade-offs that exist between a system's operational and capital costs can be identified, in much the same way as the trade-offs that exist within the environmental profile of a system.

An awareness of qualitative criteria such as social acceptance, ease of use, and reliability is important, but is difficult to include in an evaluation methodology with any significant degree of robust numerical traceability. In the MADM approach, quantitative and qualitative criteria are combined to produce a single weighted score. These are subjective, option-based weightings that can be difficult to interpret or justify. It is therefore, proposed that economic and environmental costs should be evaluated and presented individually to maintain transparency, rather than combining them in a single weighted score. Qualitative factors can then be considered where competing systems are producing similar economic and environmental profiles. Hence,

- wastewater treatment systems selection should be carried out on a scenario-specific basis because of the large variability that exists between locations in terms of scale, loading, discharge limits, and spatial restrictions;
- LCCA and LCA are appropriate tools with which to evaluate competing systems; and
- both methods of analysis should be conducted in parallel and results interpreted together without amalgamation.

3 Life cycle assessment (preliminary study)

3.1 Introduction

It has been determined that LCA is an appropriate assessment tool to evaluate the environmental performance of wastewater treatment systems. The environmental profiles of WWTSs are dominated by resource and emission flows from processes occurring upstream and downstream from the plant, and as such, require an assessment methodology that reaches beyond the immediate physical boundaries of the system. As discussed in the previous chapter, LCA provides a comprehensive and holistic mechanism for environmental cost accounting and analysis that is not achievable with other tools. The LCA component of this study was divided into two stages. The first stage is the preliminary LCA of a selection of WWTPs currently in operation in Ireland. The objectives of this stage are outlined in the goal and scope section. The findings of the study provided direction for the second stage by identifying relevant parameters, key performance indicators (KPI), and selecting suitable boundaries. The life cycle inventory (LCI) that was compiled was used in the development of a decision support tool (DST) LCA model for small wastewater treatment systems.

3.2 Methodology

Five CAS WWTPs were selected for assessment. The plants varied in scale, loading, discharge limits, and sludge disposal route. Plant characterisation, hydraulic and organic loading, discharge limits, sludge disposal details, and plant layouts are provided in Appendix B.1 – B.2. The LCA methodology presented here is applicable to both stages of the LCA component with some minor exceptions that will be discussed in LCA DST model section. The format of this assessment adhered to the framework set out by the ISO 14040 series of standards [146-149] (Figure 3-1), and references guidelines on the standards published by Guinée *et al*. [165]. The LCA software used in the project was *GaBi* 6.0. The *GaBi* database provided by *Thinkstep* (formally *PE International*) contains inventory data for upstream and downstream processes.

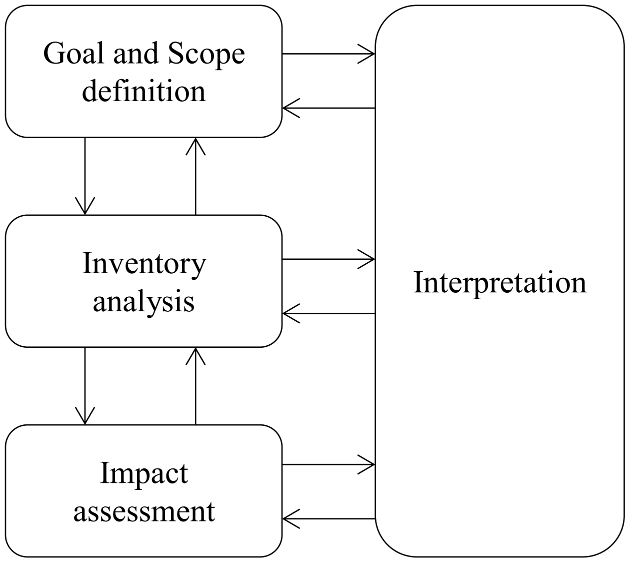

Figure 3-1: LCA methodological Framework as set out in the ISO series of standards

3.3 Goal and scope

The goals of the preliminary LCA study were

- to conduct energy audits of a selection of WWTPs in Ireland for the purpose of identifying the primary energy sinks within the systems and determining the extent to which energy consumption effects the overall environmental profile of a system;
- to determine the extent to which variations in scale, discharge limits and organic loading have on energy use, resource consumption, and environmental impact;
- to assess the environmental consequence of variations in the method of sludge treatment and disposal;
- to determine suitable boundary definitions, process flows, functional units, and impact assessment methodology for integration into a WWTS decision support tool; and
- evaluate LCA as an environmental assessment tool.

The scope of this phase of the study is presented in Table 3-1.

Table 3-1: Life cycle assessment scope definition

Parameter	*Description*
Data time line	1997 - 2015
Scale range	600 – 186,000 PE
System types	Activated sludge, pump flow bioreactors
Receiving water bodies	Coastal seawater, riverine

3.3.1 Functional unit

Baumann and Tillman [141] define the functional unit as corresponding to the reference flow to which all other flows of a system are related. There is some variance of opinion in the literature as to the most suitable functional unit for WWTS assessment. Suh and Rousseaux [171] have suggested that volume of treated wastewater per unit time is most appropriate as it is based on realistic quantifiable data. However, Corominas *et al*. [156] argue that this is not always representative, because it may not give a true indication of pollutant removal efficiency. Kelessidis [172] suggested volume of sludge produced, although it could be argued that this metric is secondary to a plant's primary function. Population equivalence (PE) and PE-year has been chosen as the function unit by several LCA practitioners [151, 157, 158], the rationale being that it allows comparisons between plants. There are a number of issues related to using PE or PE-year as a functional unit for WWTP analysis, most of which relate to a general lack of definition. Throughout much of the literature pertaining to WWTP LCA the quantity 'PE' is often ill-defined. In WWT, PE refers to two quantities: volume of wastewater, and mass of BOD loading. Henze et al. [4] define these quantities as: 1 PE = 0.2 m^3/d, and 1 PE = 60 g BOD/d, and state that '*these two definitions are based on fixed non-changeable values*'. However, the actual relationship between the hydraulic and organic loading values produced by one person can vary considerably, and the standard definition can be misrepresentative of the influent loading. The issues related to using PE, or PE-year as the functional unit, are of no relevance in stand-alone LCA audits of WWTPs; the problems arise during comparative assessments where systems are not being compared on an equal basis.

The solution proposed here was to use volume of influent as a 'base' functional unit as per the recommendations by Suh and Rousseaux [171]. Water quality analyses and energy audit results indicated whether or not there was any significant variance in influent composition between plants. Where it was determined that variance in composition was large enough to affect the results of the study, an additional impact assessment was conducted with a functional unit based on the substance of interest e.g. mass of BOD removed.

3.3.2 Boundaries

Boundary definition describes the extent to which system material and energy flows are considered. The initial boundary definition is directly related to the goal and scope of the study. If the goal of a study is to compare systems, the boundaries may be reduced to consider only the material and energy flows within the systems immediate technosphere, as in a 'gate to gate' boundary definition. This type of study is much less data-intensive because LCIs of upstream and downstream processes may not be required. If the objective of a study is a stand-alone audit, the system's material and energy flows over the entire life cycle from 'cradle to grave' are generally required. This includes the materials and processes involved in the acquisition of raw materials from the systems ecosphere, and the waste emissions returning back into the ecosphere. Alternatively, depending on the objectives, a LCA study may include a variety of boundary definitions such as gate to grave, and cradle to gate. In many cases boundary definition is a circular process, whereby the initial assumptions made during the goal and scope phase are assessed for sensitivity to boundary movement during impact assessment. Figure 3-2 represents the boundary definitions used in the current study. The following sections provide rationale for boundary selection.

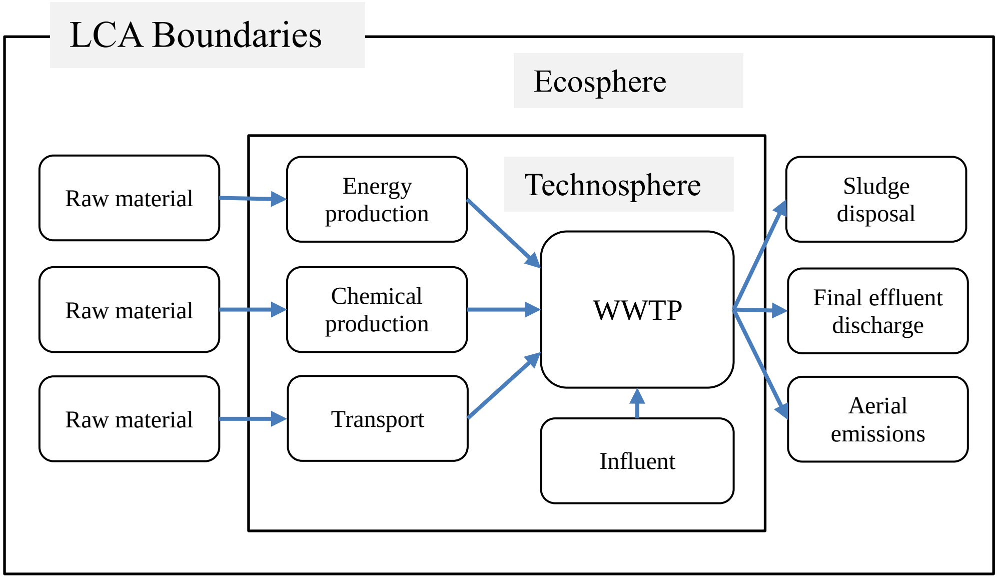

Figure 3-2: Life cycle assessment system boundaries

3.3.2.1 Upstream and downstream processes

The boundary definitions in this study extended to include many of the systems' upstream and downstream processes. There were some exceptions where LCI data for particular processes were unavailable. Comparative assessments can sometimes exclude the production of upstream inputs that are common to all systems. An example of this is the production of electricity, which, on a per kilowatt-hour basis results in the same environmental impact for all systems. However, because the quantity of energy used by each WWTS will vary, so too will the magnitude of impact from other upstream and downstream processes, because in many cases a reduction in one input can result in an increase in another. Therefore, to identify and understand the trade-offs that existed between impact categories it was necessary to include, as much as was practical, all competing inventories. Furthermore, in scenarios where a WWTP was using another source of energy such as natural gas, it produced different environmental consequences to that of electricity production, and affected the overall environmental profile of the system in question. The inclusion of chemical production LCIs were necessary to determine the effect of variations in discharge limits. Systems with total phosphorus (TP) reduction requirements generally use metal salts such as alum or ferric chloride for precipitation. Diesel production and transport inventories were linked to chemical use and downstream sludge disposal practice. The delivery of influent was not included in the LCI because the extent of sewer systems, topography, and pumping station energy requirements varied with location, and may have led to unfair comparisons of plant efficiency. Therefore, a 'gate-to-grave' boundary definition was adopted for the delivery of the influent, whereby the 'gate' was defined as the point where the influent physically enters the WWTP technosphere.

3.3.2.2 Construction

It has been reported that the impact from the construction phase of a WWTP's life cycle is negligible when compared to the operation and maintenance phase [157, 158]. In the WWTS LCA conducted by Tillman et al. [157] the construction phase was omitted from the LCI, not on the basis that the impact from construction was negligible, but rather that the magnitude of the difference in impact was negligible when compared with the use phase. However, Lundin et al. [158] state that

> *"In many long-lived installations, the construction phase is of less importance than the operation phase. However, the environment loads from the construction of smaller wastewater systems contribute a great deal to the total loads."*

There are two points in relation to including the construction phase. Firstly, the data acquisition exercise involved to compile construction phase LCIs for each system was beyond what was achievable from both a temporal and resource perspective for the current study. Secondly, the study conducted by Machado et al.[173] on CAS systems for small WWTSs found that the construction phase accounted for ~ 20% of the total life cycle impact (Figure 3-3). There is some uncertainty as to whether the construction phase percentage of attached growth systems differs significantly to that of the CAS systems. It is conceivable that there is some impact from the manufacture of the growth media but beyond that, there is very little variation in terms of the civil and structural work that occurs on site. It is known that the construction phase of the CW systems is more significant that electro-mechanical systems; however, the LCIs of each system type would be required to conduct a fair assessment, and referring back to the first point, the compilation of construction phase LCIs was beyond what was achievable in the timeframe of this study.

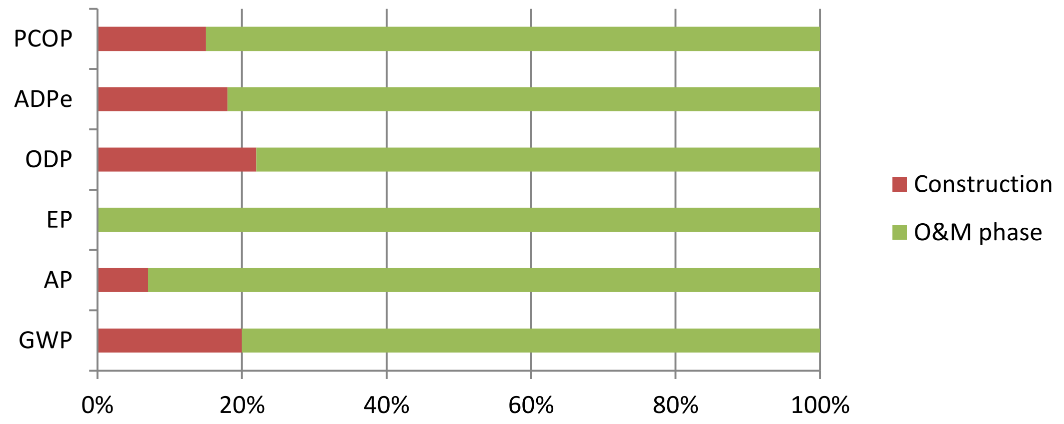

Figure 3-3: Percentage contribution of operation and construction phase to the total environmental impact of a 500 PE CAS plant (adapted from [173])

The impact of land-use was not included in the analyses. The issue of how to model land-use in terms of inventory and characterisation is an area of debate in the LCA community [174].

3.3.2.3 Avoided products

Several published LCA studies have extended the boundaries to include the production of mineral fertilizers so as to include nitrogen and phosphorus in the sludge applied to land as avoided products [164, 175]. However, in a study carried out by Renou *et al*. [176], it is stated that mineral fertilizers are spread on growing crops, and that due to safety concerns sludge is applied to the land before crop growth. Therefore, the sludge cannot be deemed to have the same fertilizing effect. However, there must be some net level of cost reduction in conditioning the soil with treated sludge or biosolids prior to crop growth, otherwise, it is unlikely that the practice would continue in such large numbers. Consequently, nitrogen and phosphorous in sludge outputs have not been included as avoided products.

3.3.2.4 Sludge disposal

Sludge disposal methods were limited to land spreading and composting. Boundary definitions for land spreading included depositions of heavy metals, nitrogen and phosphorus, atmospheric emissions of CH_4 and N_2O, and aquatic interventions resulting from leaching of nutrients into the surrounding watercourse. The boundary definition for composting was limited to the aerial emissions, and the subsequent land application emissions as described by Pradel et al. [177].

3.4 Life cycle inventory

3.4.1 Data Quality

The data quality of an LCA will ultimately determine the level of confidence that the commissioners of a study will have in the findings, and will shape the way in which the LCIA is interpreted. Direct collection and analysis of data is always preferred but not always the most practical or even possible. The type of emissions and resource data falls broadly into two categories: direct and indirect emissions and resource consumption. Indirect emissions are defined as those emissions that occur outside the wastewater treatment system technosphere. They are the residual products of all upstream and downstream processes within the wastewater treatment lifecycle (Table 3-2). In many cases, the collective indirect emissions within the WWTS life cycle account for the largest percentage of the total emissions. Direct emissions are defined as all emissions that occur within, or across the boundaries of the system's technosphere.

Table 3-2: Emissions characterisation

Direct emissions	*Indirect emissions*
Final effluent discharge	Energy production
Sludge discharge	Chemical production
Unit process aerial emissions	Transport

The direct site-specific data collected in this study included: water quality analysis data, energy use, quantities of chemicals use, sludge production and disposal method details. Indirect upstream data were aggregated datasets provided by *Thinkstep* and included LCIs for energy production in Ireland, chemical production, transport emissions and fuel refinement. Estimations were made where there were gaps in the data. These were based on a mixture of academic literature, engineering reports, manufacturers' specifications and first principles calculations. These related to areas such as unit process aerial emissions, sludge composition, and final effluent heavy metal concentrations.

3.4.2 Final effluent emissions

Final effluent water quality analysis was carried out at plants B through E. Sampling regimes and water quality analysis results are provided in Appendices B.3 and B.4 respectively. Ammonium-nitrogen (NH_4-N), total oxidised nitrogen (TON), nitrite-nitrogen (NO_2-N), and phosphate-phosphorus (PO_4-P) concentrations were determined using a Thermo Clinical Labsystems, Konelab 20 Nutrient Analyser (Fisher Scientific, Waltham, Massachusetts, United States). Suspended solids (SS) were measured in accordance with standard methods [178]. Total Nitrogen (TN), total phosphorous (TP), total organic carbon (TOC) and total inorganic carbon (TIC) were analysed using a BioTector TOC TN TP Analyser (BioTector Analytical Systems Limited, Cork, Ireland) in accordance with standard methods [178]. Biochemical oxygen demand (BOD) and chemical oxygen demand (COD) were measured in accordance with standard methods [178]. Water quality analysis data for Plant A were supplied by the plant operators and were limited to BOD, COD and TSS. Because there are no nutrient removal requirements at Plant A, operators do not record influent or effluent concentrations, and therefore, average values for effluent TN and TP were estimated based on 2012 data provided by the EPA (TN = 26 mg/l, TP = 6 mg/l, n = 63). Final effluent metal concentrations for all plants are based on national averages (Table 3-3).

Table 3-3: Irish national average final effluent heavy metal concentrations (personal communication, EPA, 2012)

Metals	*Concentration (mg/l)*
Cadmium	2.63×10^{-7}
Chromium	8.92×10^{-6}
Cobalt	4.79×10^{-7}
Lead	1.50×10^{-6}
Mercury	3.88×10^{-8}
Nickel	2.11×10^{-6}
Zinc	2.34×10^{-5}

3.4.3 Aerial emissions

The aerial emissions data used in the study were literature based. In the study conducted by Czepiel et al. [179] direct methane and carbon dioxide emissions from a CAS WWTP were estimated to be 39 kg CH_4/PE-year and 35,698 kg CO_2/PE-year. Based on a CO_2 equivalency factor of 21 for CH_4 for a time horizon of 100 years, the total CO_2 emissions are 36.5 kg CO_2/PE-year. This equates to 0.3 kg CO_2/m^3 of wastewater treated based on a hydraulic PE definition of 333 litres. The emissions data gathered in the study were taken from the inlet works, primary settling, aeration tanks, secondary settling and sludge holding. The most significant sources were found to be the grit removal, aeration, and sludge storage processes. However, in the study cited, the sludge holding tanks were also aerated, and further work is needed to determine whether the same emissions would occur in non- aerated sludge holding tanks. The aeration process accounted for over 51% of the total CH_4 emissions and for 92% of the total CO_2 emissions. It should be noted that in the study carried out by Czepiel et al. [179] the system did not include an AD process due to the small scale of the WWTP. In similar studies of larger plants equipped with the AD process, CH_4 emissions were reported to be almost ten times that of the non-AD system at 306 kg CH_4/PE-year [180]. The AD process was found to account for 75% of the total CH_4 emissions produced at the plant. However, it is unclear what percentage of the CH_4 emissions reported in this study was actually released into the atmosphere. The emission values that were recorded were taken from the plant's ventilation system that sends process off-gas to an ozone washer.

Nitrification and denitrification can act as both sources and sinks for GHG emissions. During denitrification, as nitrate (NO_3^-) is converted to N_2 gas, nitrous oxide (N_2O) is produced as an intermediary product, not all of which is converted to N_2. The CO_2 equivalency factor for N_2O is 310 kg CO_2 equiv./kg N_2O and therefore, small amounts of N_2O have significant impact on the system's GHG inventory. However, Czepiel et al. [181] reported an emission factor of 3.2 g N_2O/PE-year or 0.026 g N_2O/m^3 for an agglomeration size of 12,500 PE. This is a small quantity relative to the CO_2 and CH_4 emissions, and even with the large CO_2 equivalency factor the N_2O accounts for only 0.02 % of the total 0.3 kg CO_2 equiv./m^3 of treated wastewater. [145]

3.4.4 Sludge emissions

The method of sludge disposal at Plants B to E was through application to agricultural farmland. It has been reported that the most significant impact from this method of sludge disposal is caused by the concentrations of heavy metals being deposited in the soil [155]. The application of sludge to farmland provides a pathway to recycle nutrients in the form of nitrogen, phosphorus and potassium back into the ecosystem. However, it can also result in the deposition and accumulation of harmful metals in the soil which is characterised as toxicity potential and measured in units of kilograms of 1, 4 dichlorobenzene (DCB) equivalent in the CML LCIA methodology. Site-specific sludge composition data were unavailable. Estimations of metal concentrations used in the study are based on the report conducted by the EU commission (Table 3-4). Values of organic sludge pollutants were provided in the same report but are based on European averages. There were no specific organic pollutant data for Ireland included in the publication (Table 3-5).

Table 3-4: Average concentrations of metals in Irish sludge in 1997 [104]

Metals	*Concentrations (mg/kg DS)*
Cadmium	2.8
Chromium	165
Copper	641
Mercury	0.6
Nickel	54
Lead	150
Zinc	562

Table 3-5: European concentrations of organic contaminants in sludge [104]

Organic compounds	*Abbreviation*	*Concentrations (mg/kg DS)*
Absorbable organo-halogen compounds[4]	AOX	200
Polycyclic aromatic Hydrocarbons	PAH	14.15
Polychlorinated biphenyls	PCB	0.09
Polychlorinated dibenzo-dioxins and -furans[5]	PCDD/Fs	36

3.4.4.1 Composting

Sludge produced at Plant A was anaerobically digested before being exported to a composting company. Anaerobic digestion reduces sludge volume through decomposition of the volatile suspended solids (VSS) fraction of the total suspended solids. Concentrations of metals in sludge are reported as a percentage of the dry solids (DS) concentration, and therefore, it was assumed that there is no reduction in the quantity of metals leaving the plant as a result of anaerobic digestion. According to Ponsá et al. [182] the optimum volumetric ratio of bulking agent to dewatered sludge to reach satisfactory stability for application to agricultural land is 3:1. These values were determined with sludge and bulking agent moisture contents of 84% and 17% respectively. Sensitivity analysis was conducted to assess the effect of the bulking agent–sludge ratio using the sludge-based compost metal concentration values reported by Herity [183] as the benchmark. Metal concentration values reported in Table 1-5 were applied to the sludge dry solids concentration value from Plant A. Ratios of 1:1, 2:1 (reported by Ponsá as being the commonly adopted ratio), and 3:1 were assessed. The recommended volumetric bulking agent ratio reported by Ponsá was found to have the best agreement with the values reported by Herity (Figure 3-3). This result indicates that if the metal concentrations reported by the EU report are accurate, then the bulking agent ratios adopted by the Irish composting practitioners is in line with the recommended standards for wastewater sludge base composting.

[4] German data only.
[5] Units in ng/kg TEQ (toxicity equivalents)

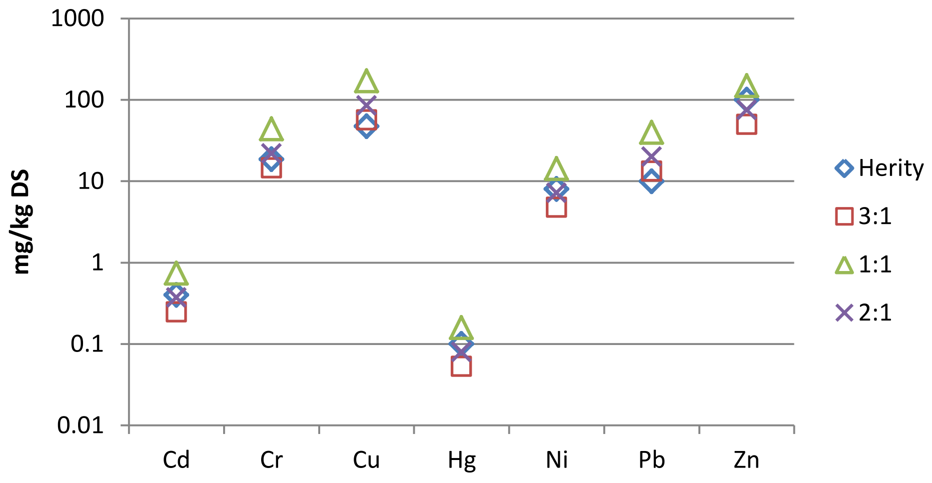

Figure 3-4: Sensitivity of metal concentration to variations of bulking agent volume to sludge volume

Hence, to facilitate variation in both sludge and bulking agent DS concentration, the concentration of an individual metal C_i, is given by Eq. 6.

$$C_i = C_{o,i}\left(\frac{B_{DS}}{S_{DS}}\right) \tag{Eq. 6}$$

Where,

C_i = concentration of metal *i* in compost (mg/kg DS)

$C_{o,i}$ = original concentration of metal in sludge (mg/kg DS)

B_{DS} = mass of bulking agent dry solids (kg) (Eq. 7)

S_{DS} = mass of sludge dry solids (kg) (Eq.8)

$$B_{DS} = \left(\frac{B_{vf}}{B_{DSc}(1 \times 10^{-3})}\right) \tag{Eq. 7}$$

Where,

B_{DSc} = bulking agent dry solids concentration (kg/m^3)

B_{vf} = bulking agent volumetric fraction (m^3)

$$S_{DS} = \left(\frac{S_{vf}}{S_{DSc}(1 \times 10^{-3})}\right) \tag{Eq. 8}$$

Where,

S_{DSc} = sludge dry solids concentration (kg/m^3)

S_{vf} = sludge volumetric fraction (m^3)

3.4.4.2 Sludge aerial emissions

The aerial emissions associated with sludge disposal were divided between on and off-site emissions and are presented in Table 3-6. The emissions from sludge storage were included in the aggregated unit process emissions from each WWTP and are therefore not included here in order to avoid double counting.

Table 3-6: Sludge treatment and disposal emissions presented in kg/tonne of dry solids

Process	*Emission*	*Quantity*	*Source*
Anaerobic digestion	CH_4	0.18	[177, 184]
	CO_2	1,291	[25]
	NO_2	0.85	[25]
	N_2O	0.02	
Composting	CH_4	2.9	[177]
	N_2O	0.4	[177]
Land application of limed sludge	N_2O	0.05	[177]
	CH_4	3.18	[25]
Land application of composted sludge	N_2O	0.05	[177]

As mentioned previously, there is some debate surrounding the inclusion of the production of synthetic fertilisers as an avoided product. However, the application of nitrogen or phosphorus to farmland does provide the potential for their transportation to a watercourse and ultimately contribute to eutrophication. This has particular relevance in countries with high levels of precipitation such as Ireland. Typical concentrations of nitrogen and phosphorus in wastewater sludge are presented below (Table 1-7). Eutrophication potential that results from the TP concentrations in the sludge outputs from the freshwater plants were based on the quantities of TP that were removed from the treated water line. The TP reductions at Plant A were based on average historical data. Total nitrogen content of sludge outputs could not be calculated in the same way as much of the nitrogen at the freshwater plants leave the system in the form of N_2 gas. Therefore, the concentration of TN in the sludge for the freshwater plants was based on the figures presented in Table 3-7.

Table 3-7: Typical nitrogen and phosphorus concentrations of primary and secondary sludge

	Dry solid concentration (%)	*Nitrogen concentration (% of DS)*	*Phosphorus concentration (% of DS)*	*Source*
Primary sludge	2 - 5	1.5 - 4	0.8 – 2.8	[114]
Secondary sludge	0.4 – 1.5	2.4 – 5	2.8 - 11	[114]

3.4.5 Energy

Energy audits were carried out at Plants B - E. Electricity and natural gas consumption data for Plant A was provided by the operators for November 2013 to coincide with final effluent and sludge production data. The electricity production LCI compiled by *Thinkstep* contains all upstream and downstream processes for the Irish electricity mix for the year 2011. Energy audit results are presented in Appendix B.5.

3.4.6 Chemicals

Quantities of chemicals used at Plants A, B and C were supplied by the plant operators. The quantities were based on monthly purchase orders. Plants D and E export untreated sludge to a larger parent plant. Plant A chemicals include ferric chloride, sodium hypochlorite, and sodium hydroxide. The chemical inventory at Plant A also includes two brand name sludge thickening and dewatering polymers (Envirofloc 166 and Dryfloc 909H). However, LCI datasets were not available for these two polymers. Generic polymer LCIs in published LCA literature were found to be aggregated into the larger system LCI, and therefore, could not be included in the study. Acrylic acid has been reported as the primary component of many flocculants, and as such has been included as the substitute for dewatering polymers. Chemical inventories for Plants B and C were limited to ferric chloride used for P precipitation, and sludge dewatering polymers (no chemicals used for gravity thickening). It was assumed that the sludge quantities exported by Plants D and E were thickened and dewatered at the parent plant; thus, estimated ferric chloride, calcium hydroxide, and acrylic acid quantities were included as part of a chemicals inventory for both of these plants.

3.4.7 Transport

Chemicals and sludge loads for plants B to E were assumed to be transported with a 7.5 tonne lorry at an average distance of 40 km. Plant A sludge was 175 km from the composting plant and 27 km from the chemical suppliers. The LCIs for transportation and diesel refinement are supplied within the GaBi database. The energy and material flow schematics for the freshwater (Figure 3-4) and seawater (Figure 3-5) systems are presented below. Unit process data sets with complete LCIs such as energy, chemicals and transport are represented by their own process block as these are data intensive processes. Single flows into and out of the WWTP such as the wastewater are accounted for within the WWTP block by the relative weight of their constituents e.g. mg BOD/l, mg NH_3/l. Although the sludge output, like the wastewater input is a single flow represented by the whole of its constituents, the sludge output is associated with several additional unit processes (polymer and lime addition, sludge transport), and warranted its own process block to differentiate its inputs and outputs separate from that of the treatment plant. The mass flows are represented for each unit process, but the energy flows are not presented in the schematics as the software is limited to one unit measurement type.

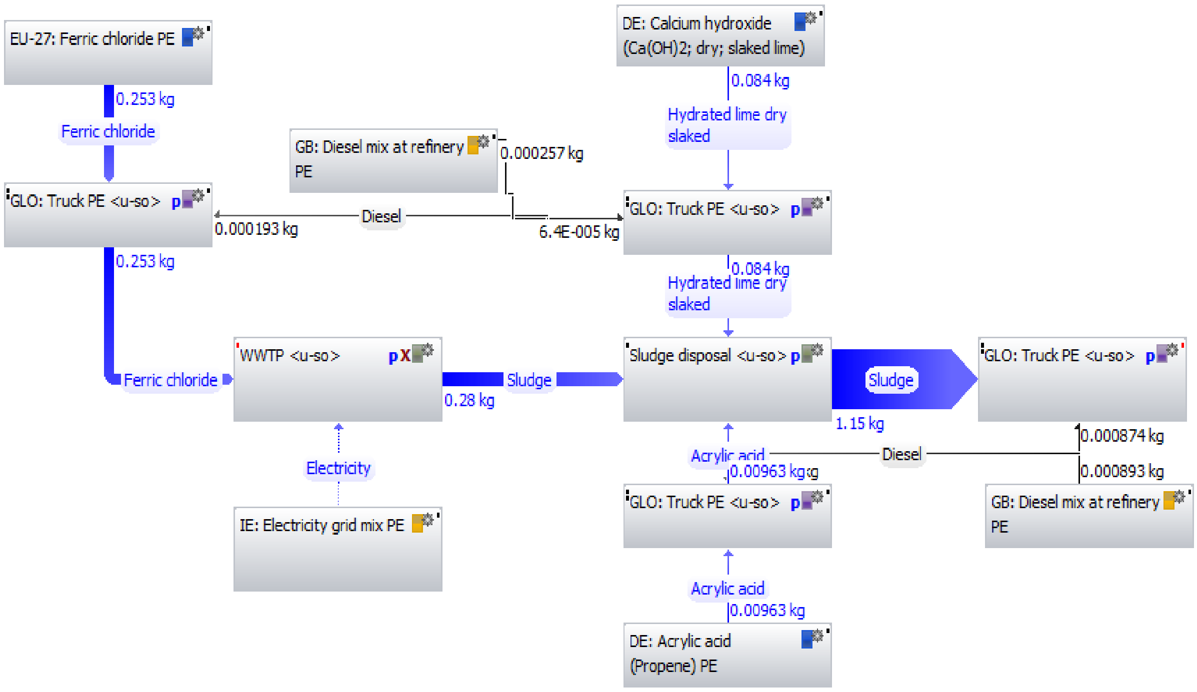

Figure 3-5: Freshwater WWTPs energy and material flow schematic

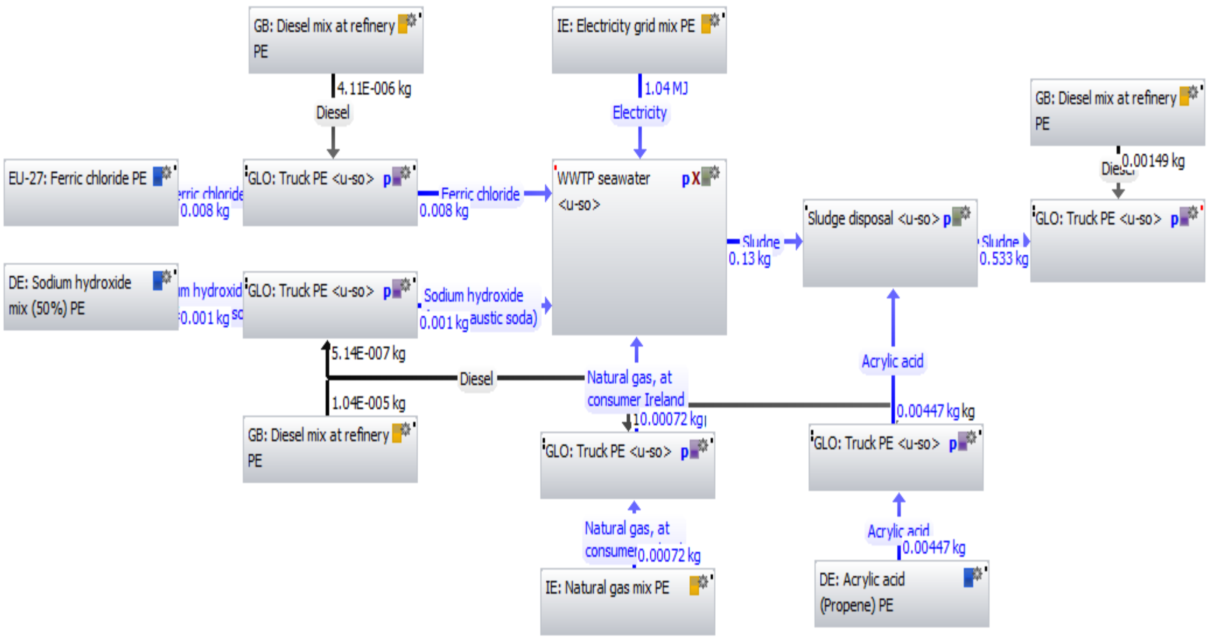

Figure 3-6: Seawater WWTP energy and material flow schematic

3.5 Energy auditing

Energy auditing was conducted at each plant with varying degrees of sub-system analysis. Plant A is an interesting case study from an energy management perspective. It is the only system large enough for AD to be economically feasible. The digester and the CHP plant accounted for 42% of the overall energy consumption (Figure 3-6) and 100% of the imported natural gas. However, the 14% energy flow to the CHP plant does not include the energy generated from the AD biogas. It can be seen from Figure 3-7 that the amount of energy generated from the CHP plant is less than the imported natural gas. Therefore, although the CHP plant was producing 31% of the overall plant energy, in actuality, the net energy benefit was only 10%. There are, however, other non-energy, economic and environmental cost savings associated with AD such as solids reduction, sludge stabilisation, and the removal of potential GHGs from outgoing sludge. Natural gas is also 3 – 4 times cheaper in terms of €/kWh than electricity from the mains grid. However, from an efficiency perspective, the energy recovery here was much less than the achievable 50% energy recovery value reported by Gude [113]. Proportionately, biological treatment (aeration) energy consumption at Plant A was relatively low in comparison with other activated sludge systems and with other unit process or groups of processes within this system such as the sludge treatment sub system, which consumed over twice the energy used for aeration. This illustrates one of the effects of variation in discharge limits. Because nutrient reduction is not required, the SRTs can be shortened. This reduces aeration energy and increases sludge volumes, which in effect, involves a trade-off within the system's energy distribution profile. The overall energy efficiency exhibited by Plant A could be attributed to scale, the less stringent limits, or to a combination of the two. However, it is worth noting that the effluent BOD, TSS, and COD at Plant A were lower than any of the other plants in this study (5.1, 10.1, and 34.8 mg/l respectively). A comparison with a similar size plant with more stringent discharge limits is necessary to make any definitive conclusions regarding this matter.

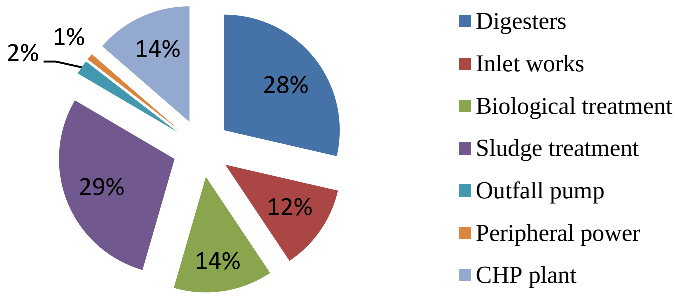

Figure 3-7: Plant A energy distribution profile

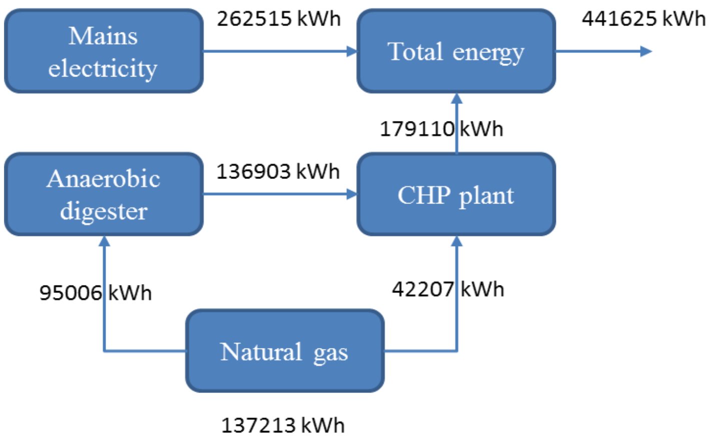

Figure 3-8: Plant A energy flow and recycle (November 2013)

The energy distribution for Plants B, C, D and E is presented below (Figure 3-8). Plants B and C are similar systems in size and configuration. Both plants have design capacities of 12,000 PE. Plant C is required to reduce TN and has a slightly lower TP limit (1 mg/l) than Plant B (2 mg/l). Plant B is not required to reduce TN, and also has slightly higher BOD and TSS limits (25 mg/l and 35 mg/l respectively). The overall specific energy consumption at both plants was high when compared with other reported specific energy consumption values for activated sludge systems (0.92 kWh/m^3 at Plant B, and 0.75 kWh/m^3 at Plant C). There was a 41% difference in the energy consumption attributed to aeration – 69% at Plant B, and 28 % at Plant C. Some of the difference can be attributed to variations in the other energy sinks in the respective systems. Plant C final effluent discharge was pumped 100 m uphill to its discharge point. Pumping accounts for 29% of the total energy consumption at Plant C, which is twice as much as any of the other plants.

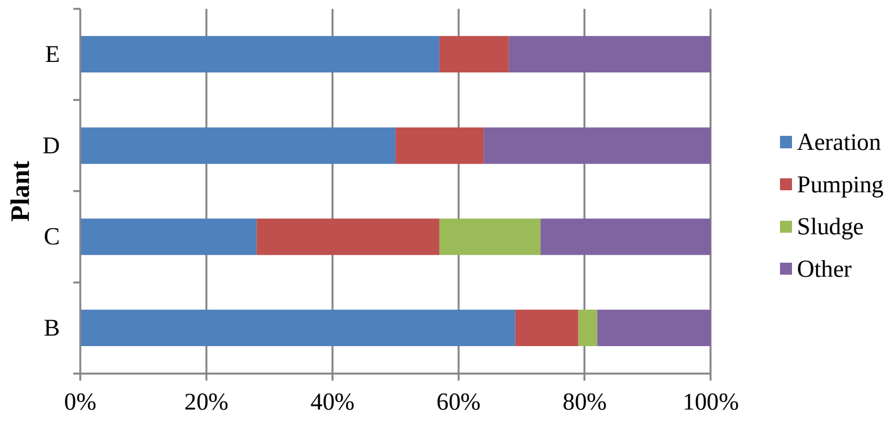

Figure 3-9: Energy distribution profile for Plants B, C, D, and E

The primary reason for the difference in energy can be attributed to organic loading. Figure 3-10 presents the relationship between the aeration energy percentage and influent BOD loading. During the testing period the hydraulic loading at Plant C was only 6% higher than that of Plant B, but the organic loading at Plant B was over twice that of Plant C. At plant B when the BOD loading was 200 mg O_2/l the percentage of the total energy attributed to aeration was 69%, while at plant C, where the BOD loading was 99 mg O_2/l the percentage of the total energy was only 28%. The variation in the aeration energy demand between the two plants demonstrates the direct relationship between energy consumption and organic loading. However, it should also be noted that aeration energy consumption is not limited to BOD loading. Both plants are subject to ammonia reduction, and as such will incur additional oxygen requirements beyond the oxidation of the organic substrate. The TN loading at plant B (71.5 mg TN/l) is over twice that of plant C (29.6 mg TN/l) which may account for the slightly higher aeration-percentage/ BOD-load ratio at plant B.

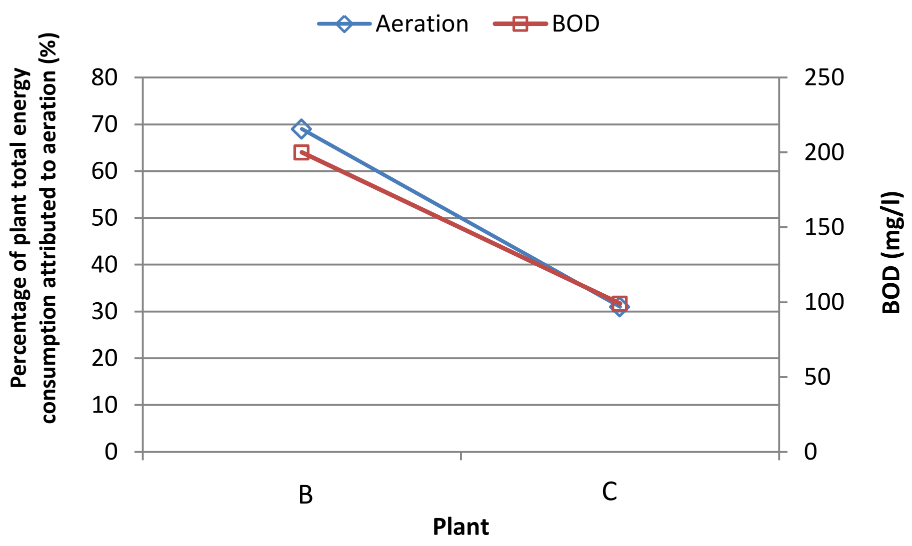

Figure 3-10: Plants B and C percentage of total energy consumption and influent organic loading

Plants D and E have design capacities of 820 PE and 600 PE respectively. The discharge limits at Plant E are more stringent than those at Plant D (Table 3-8). Except for TSS, the discharge limits at Plant E are less than half the value of those at Plant D. Despite this, the specific energy consumption at Plant E was lower than that of Plant D (0.68 kWh/m^3 and 0.60 kWh/m^3 respectively). It is difficult to assess the effect of the difference in ammonia limits without final effluent ammonia concentration data. Total nitrogen removal rates were almost three times higher at Plant E but this is not reflected in the energy consumption values. It is worth noting that during the period of testing Plant D hydraulic loading exceeded design capacity. Notwithstanding this, as with the Plants B and C there are correlations between the percentage of the total plant energy attributed to aeration and the levels of organic loading [Plant D expends 0.4% of aeration energy per mg of influent BOD and Plant E expends 0.42% of aeration energy per mg of influent BOD (Figure 3-10)].

Table 3-8: Plant D and E discharge limits

Discharge limits	*Plant D*	*Plant E*
cBOD	25 mg/l	10 mg/l
COD	125 mg/l	50 mg/l
Suspended solids	35 mg/l	25 mg/l
Total nitrogen	-	-
Total phosphorus	-	-
Ammonia	5 mg/l	1 mg/l
Orthophosphate	2 mg/l	0.5 mg/l

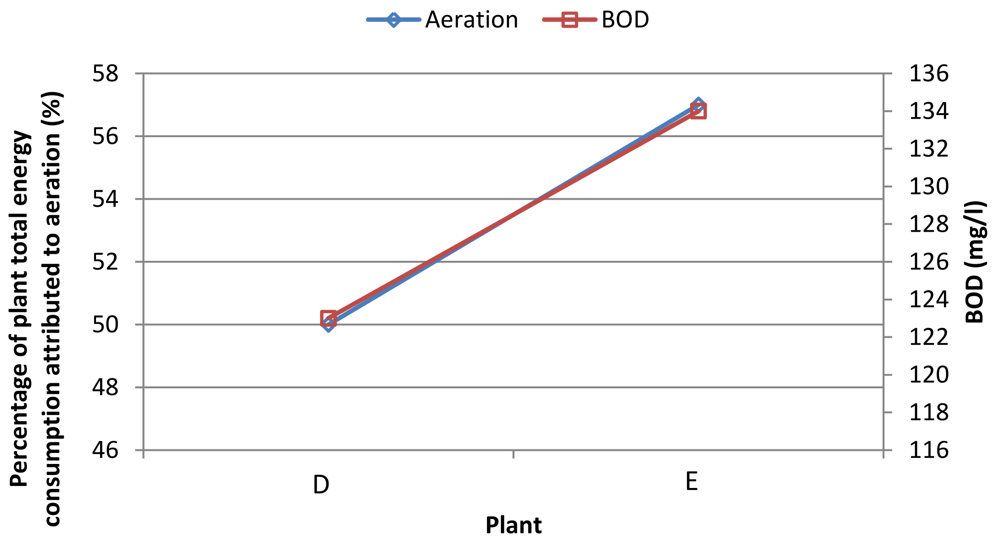

Figure 3-11: Plants C and D percentage of total energy consumption attributed to aeration and influent organic loading

Energy efficiency values are presented below in terms of hydraulic load and BOD removal (Figure 3-11 and Figure 3-12). There are significant variations in efficiencies depending on the chosen metric. The effect of scale is most prominent between the largest and smallest plants. The specific energy consumption at the two medium size plants in terms of hydraulic load seems higher than would have been anticipated when compared with the two smallest plants. The average specific energy use reported by Gallego et al. [151] for plants of a similar scale was 29.1 kWh/PE-year[6], which equates to 0.39 kWh/m^3 based on a hydraulic PE definition of 200 L.

[6] The value was reported as 29.1 kWh/PE. It is presumed that this is meant to be PE-year as the value reported per PE is unrealistic.

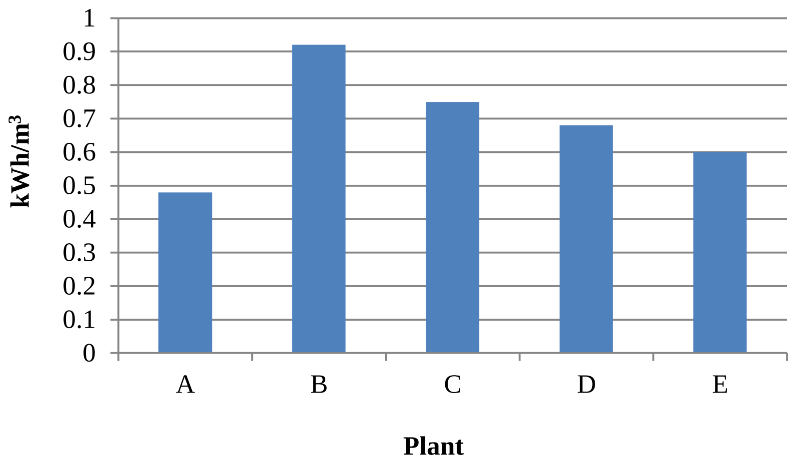

Figure 3-12: kWh/m^3

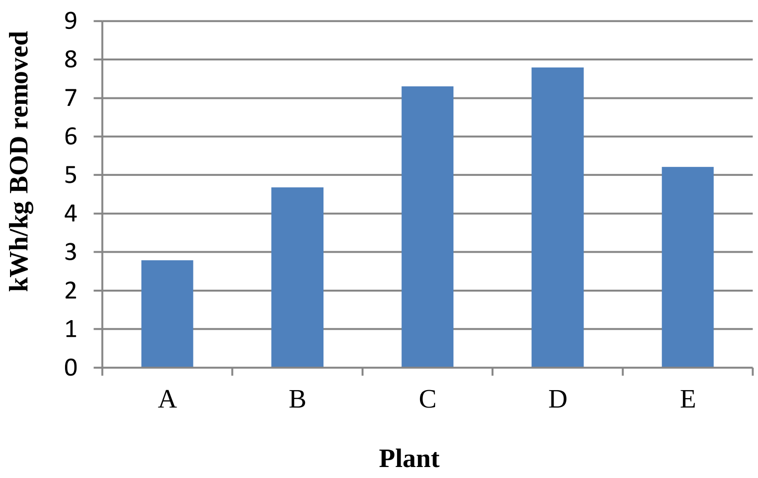

Figure 3-13: kWh/kg BOD removal

3.6 Life cycle impact assessment

3.6.1 Introduction

It is important to provide the rationale behind the choice of life cycle impact assessment (LCIA) methodology, and the limitations associated with its use. Firstly, it is necessary to understand the difference between *midpoint* and *endpoint* life cycle impact assessments. The LCIA methodology used in this study is the CML (Centre for Environmental Science) 2001 (Nov.10) which is compliant with the ISO 14040 series, and has been adopted by authors of similar studies [164]. This is a midpoint LCIA methodology, and as such, stops short in attempting to predict the actual effect of any environmental intervention as per endpoint LCIA methodologies. It instead presents the *potential* of a system's emissions to cause environmental harm. A brief overview of the methodology is presented here. The first stage in the process is to define the impact categories and their baseline units as presented here in Table 3-9.

Table 3-9: CML 2001 life cycle impact assessment categories

Impact Category	*Abbreviation*	*Units*
Global Warming Potential	GWP	kg CO_2 equiv.
Acidification Potential	AP	kg SO_2 equiv.
Eutrophication Potential	EP	kg PO_4^{3-} equiv.
Ozone Depletion Potential	ODP, steady state	kg R11 equiv.[7]
Photochemical Oxidation Potential	PCOP	kg C_2H_6 equiv.
Ecotoxicity		kg $C_6H_4Cl_2$ equiv.
• Freshwater Aquatic	FAETP inf.	
• Terrestrial	TETP inf.	
• Marine Aquatic	MAETP inf.	
Human Toxicity Potential	HTP inf.	kg $C_6H_4Cl_2$ equiv.
Abiotic Depletion elements	ADPe	kg Sb equiv.
Abiotic Depletion fossil	ADPf	MJ

Once the impact categories have been defined the next phase of the LCIA is *classification*, whereby system inputs and outputs that have been compiled in the LCI are assigned to one or more of the impact categories. Following this, the *characterisation* phase calculates the magnitude of a substance in an impact category based on an *equivalency factor* relative to a baseline substance for that category. For example, the mass of COD in the final effluent discharge is assigned to eutrophication. The baseline substance for eutrophication is PO_4^{3-} (phosphate) and has a value of 1. The equivalency factor value of COD is 0.022 [141], therefore, every 1 g of COD is equivalent to 0.022 g of phosphate in the CML EP impact category. This method allows aggregation of all substances assigned to a given category into a single score or *indicator result* (Eq. 9) [165], where i, is the type of substance, m_i is the magnitude and ef_i is the equivalency factor for that substance.

[7] The refrigerant R11 is a chlorofluorocarbon (CFC)

$$indicator\ result = \sum_i m_i \times ef_i \qquad \text{(Eq. 9)}$$

It is at this stage that midpoint methodologies move on to the interpretation stage of an LCA without any further levels of aggregation. Endpoint LCIA methodologies can have one or two further qualitative stages of aggregation, which in some cases results in a single indicator value. A source of uncertainty and an area of ongoing debate are the weighting factors or value judgements for endpoint impact assessment, whereby one impact category is compared or weighted against another [185]. These weightings are for the most part qualitative with only minor relative quantification and are based mainly on political, social or ethical values. Several weighting methods have been devised by different institutions; such as the *technology abatement approach*, whereby an impact value can be set based on the technology abatement method chosen, or *monetarisation*, whereby values are based on an aggregation of human preference and a willingness to pay [142]. A commonly adopted method is the *authoritative panel*, whereby a selection of societal groups, scientific experts, or other various international bodies join together to decide on weightings or values. No concrete methodology has so far been agreed upon by the scientific community. However, even before these further qualitative aggregations take place, there are several stages of the cause and effect chain that introduce varying degrees of uncertainty.

In general, the contributions from the inventory data to an impact category are governed by a single model that assumes one standard situation in each link of the cause effect chain (Figure 3-13). Potting et al. [186] postulate that for the impact categories of a global nature e.g. GWP, AP, the simplified linear model is sufficient in that the size of the impact can be adequately expressed in terms of an equivalent emission or reference compound. However, this assumption only takes into account the *potential* for global warming and not the effects. Tillman and Baumann [141] describe a range of effects by way of the following example. CO_2 emissions lead to a change in radiative forcing, which is the primary effect. The secondary effect is the change in radiative forcing which leads to a change in global temperature. At this stage a spatial differentiation is required due to the fact that temperature change will not be the same around the planet, nor will the tertiary effects i.e. melting polar caps, drought, and changes in biodiversity. Hence, while it is safe to assume that there is linearity between global warming emissions and potential, the relationship between emissions and effect is more complicated.

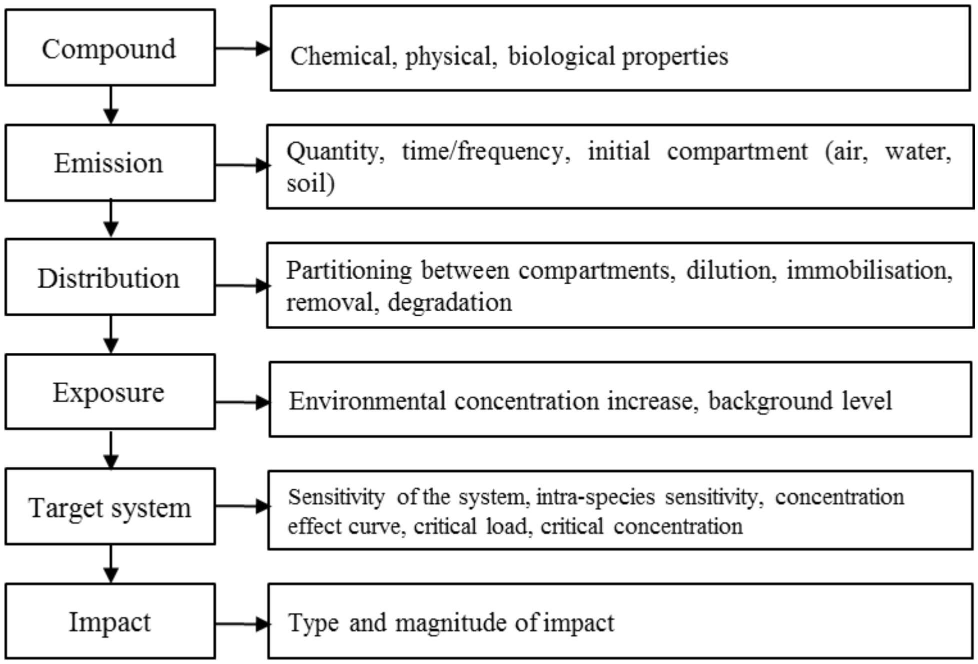

Figure 3-14: The cause effect chain in life cycle impact assessment. Adapted from [186]

Spatial variability is a significant contributor to the lack of accordance between predicted environmental impact and actual environmental impact [163]. Parameters such as existing background concentration of substances, physical, chemical and biological properties of the receiving environment, and human population densities have an effect on the actual environmental impact. These parameters however, are not accounted for in most of the current LCA models. Huijberg et al. [187] claim that spatial and temporal characteristics are lost by the aggregation of emissions in the inventory analysis.

Most current LCA models include some form of differentiation of receiving compartments (emissions to air, water and soil); however, another level of differentiation can be justified from the perspective that within a compartment there can also be significant variability in the rate of penetration of a compound e.g. sandy soils will leach compounds faster than clay soils. This awareness of the need for spatial differentiation has been realised by the EU in relation to acceptable levels of eutrophication caused by final effluent discharge. Annex 2 of the UWWTD outlines the '*Criteria for Identification of Sensitive and Less Sensitive Areas*'. This is in recognition of the fact that the ecologies of some waters are more sensitive to nutrient levels than others, thus underlining the need for some range of operational spatial variability in LCA. Much like spatial variability, there is limited accounting of temporal variability in LCA. Aerial based impact categories such as GWP, AP and POCP have a selection of time horizons (e.g. GWP 20, 50, 100) that give some degree of control over temporal variability [187]. The magnitude of the impact of each category varies depending on the time horizon chosen because of the varying residence times of the compounds contributing to these categories. The 100 year time horizon is often used arbitrarily as the default value by LCA practitioners. However, Smith and Wigley [188] have claimed that GWPs are only accurate for short time horizons.

Attempting to predict the actual environmental impact of system is subject to varying degrees of uncertainty, particularly at a regional or local level. This is a general limitation in most LCA methodologies that provides cause for scepticism [142]. Endpoint LCIA methodologies introduce further levels of uncertainty that may influence the willingness of an audience to accept the results being presented. Therefore, it is assumed that the use of a midpoint LCIA methodology may provide a more numerical traceability and transparency.

3.6.2 Discussion

The individual LCIA results of the eleven impact categories are presented in Appendix B.6. The discussion presented here is limited to some of the general findings in the study. Electrical energy use was found to dominate global impact categories such as GWP, AP, and ADPe. The environmental impact resulting from energy use was found to depend on two factors: the quantity of energy used, and the mode of energy generation. Accounting for over 80% of the total energy generated, the electrical grid mix in Ireland is heavily fossil fuel dependent (Figure 3-14). The GWP values (normalised with CML 2001 - 2013 Western Europe normalisation factors) ranged from 1.37 x 10^{-13} – 2.49 x 10^{-13}, which is higher than those reported by Pradip et al. [160] (2.15 x 10^{-14} – 5.09 x 10^{-14}). However, the scale factor of the Pradip study (200,000 PE) is the likely cause of the disparity here. In the study conducted by Gallego et al. [151] in which the plant scale range is closer to that of the current study, the GWP ranged from 6.85 x 10^{-14} to 2.19 x 10^{-13}. The AP ranged from a low of 2.44 x 10^{-14} to a high of 1.01 x 10^{-13}, and is more comparable with the Pradip study (3.00 x 10^{-14} – 1.70 x 10^{-13}). The Gallego study had the highest range of AP values (1.23 x 10^{-13} to 3.97 x 10^{-13}).

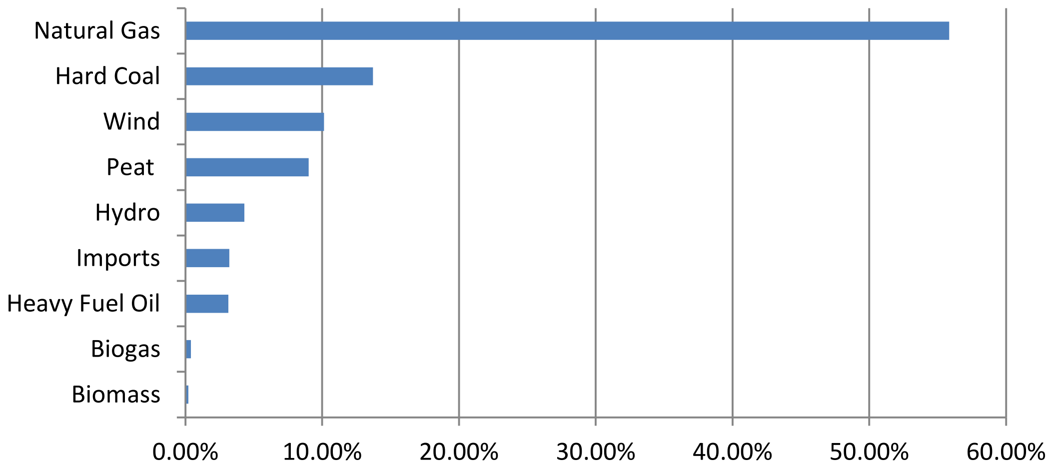

Figure 3-15: Ireland's electricity grid mix (2012)

The regional toxicity categories FAETP, HTP, and TETP were found to be highly sensitive to the heavy metal concentrations in the sludge. Sensitivity analysis found that freshwater toxicity is heavily influenced by the presence of nickel, of which, according to the 2001 report issued by the EU [104], Ireland has the third highest concentration in sewage sludge behind Greece and the UK. Despite this, the FAETP ranged from 1.19 x 10^{-14} to 2.82 x 10^{-13}, which is magnitudes lower than the Pradip study (1.68 x 10^{-12} – 4.19 x 10^{-12}); however, the TE range was higher (2.74 x 10^{-13} – 4.03 x 10^{-12}) compared with 1.45 x 10^{-14} – 2.60 x 10^{-14}. Polycyclic aromatic hydrocarbons (PAHs) and Polychlorinated biphenyls (PCBs) were considered for inclusion in the LCI but the CML methodology does not currently have characterisation factors for these substances. Phosphorus and nitrogen concentrations in the final effluent discharge and sludge outputs are the main contributors to eutrophication. The contributions from BOD and COD to EP are orders of magnitude lower. The ODP and ADPe categories represent the smallest and largest contributors respectively to the overall environmental profile of each plant (Figure 3-15). The ADPe impact category evaluates the depletion of natural elements such as minerals and ores. The large values presented in Figure 3-15 can be attributed almost exclusively to $FeCl_2$ used for phosphorus precipitation. The ODP category is also dominated by $FeCl_2$ use.

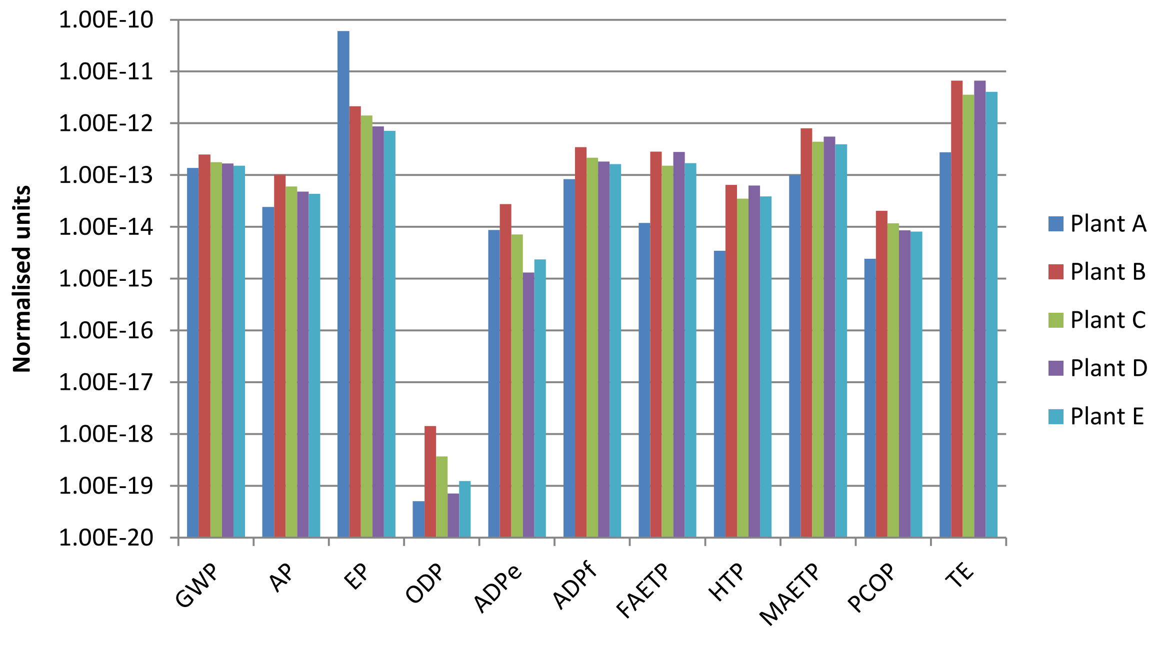

Figure 3-16: Lifecycle impact assessment results (volume of wastewater functional unit) normalised with CML 2001 - 2013 Western Europe normalisation factors [189]

Water quality analysis and energy auditing found that the variation in organic load had a direct influence on energy efficiencies, and produced varying efficiency levels depending on whether the metric used is kWh/m^3 or kWh/ kg BOD removed (Figure 3-11and Figure 3-12). It is, therefore, necessary to assess variations in functional unit. The base functional unit chosen for this study is 1 m^3 of wastewater treated (assuming influent flowrate = treated flowrate). To examine the effect of the variation in organic loading a second LCIA was conducted with an organic load functional unit of 200 g BOD removed (the BOD removal at Plant B was exactly 200 mg/l when the functional unit was 1 m^3, and for convenience was chosen as the baseline for comparison). It can be seen in Figure 1-15 when the functional unit is volume of treated wastewater that Plant B exhibits the largest output in most impact categories. Plant B EP is lower than that of Plant A because of less stringent discharge limits incurred by the coastal plant. When the functional unit is 200 g BOD removed the environmental profile of Plant B remains the same but appears more favourable because of the increase in impact category magnitudes exhibited by the other treatment plants (Figure 3-16). The exceptions are in the ADPe and ODP categories which are dominated by chemical production. Ferric chloride consumption at Plant B was significantly higher than recorded or estimated for other plants.

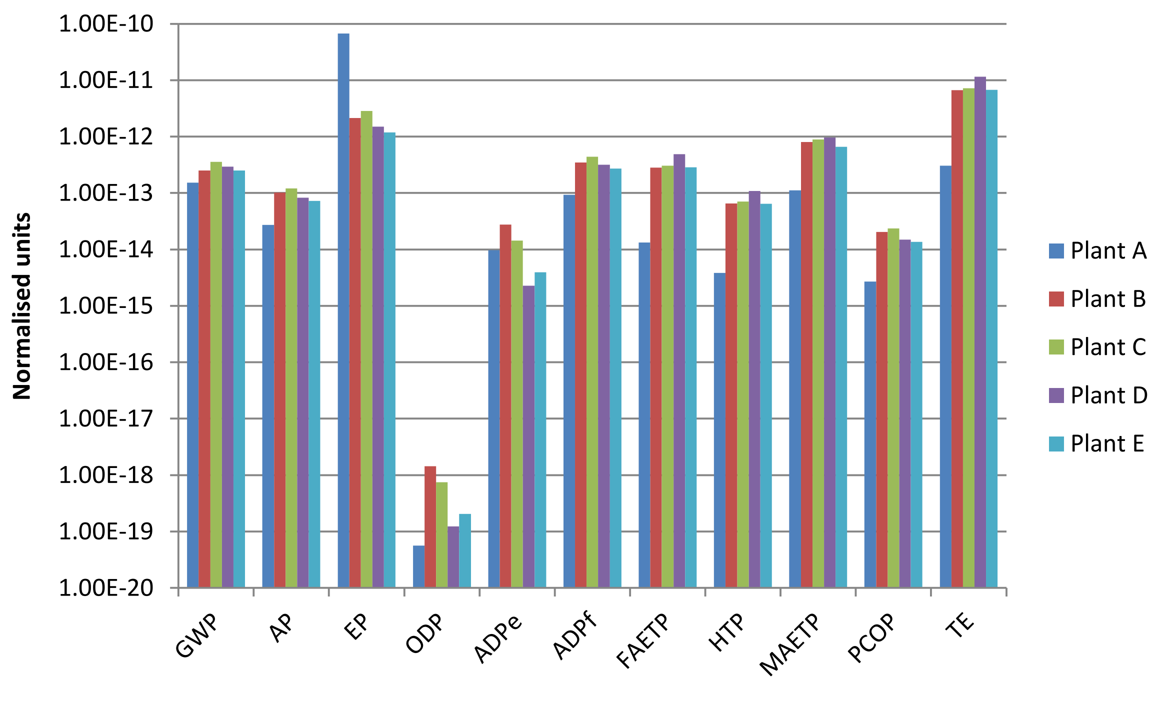

Figure 3-17: Lifecycle impact assessment results (200g BOD removed functional unit)

3.6.2.1 Global warming potential

Plant A had the lowest GWP at 0.65 kg $CO_{2,\ equiv.}/m^3$ treated wastewater, and Plant B had the highest (1.4 kg $CO_{2,\ equiv.}/m^3$) with the hydraulic functional unit (Figure 3-17). The lowest GWPs reported by Gallego et al. [151] (0.33 kg $CO_{2,\ equiv.}/m^3$)[8] were also recorded at two of the largest plants in the 13 plant study, while the highest GWP (1.07 kg $CO_{2,\ equiv.}/m^3$) was recorded at plants that were 6 times smaller in terms of agglomeration served. A significant increase in GWP was observed at Plants C, D and E when the functional unit was mass of BOD removed (Figure 3-18). Plant C then had the highest GWP (1.4 kg $CO_{2,\ equiv.}$/200g BOD removed), and Plant A GWP remained the lowest and relatively unchanged because of similar solids loading at Plants A and B. Similar variations were observed between specific energy efficiency metrics (Figure 3-11 and Figure 3-12), which indicates that impact categories that are sensitive to energy use will exhibit similar variations with changes in functional unit.

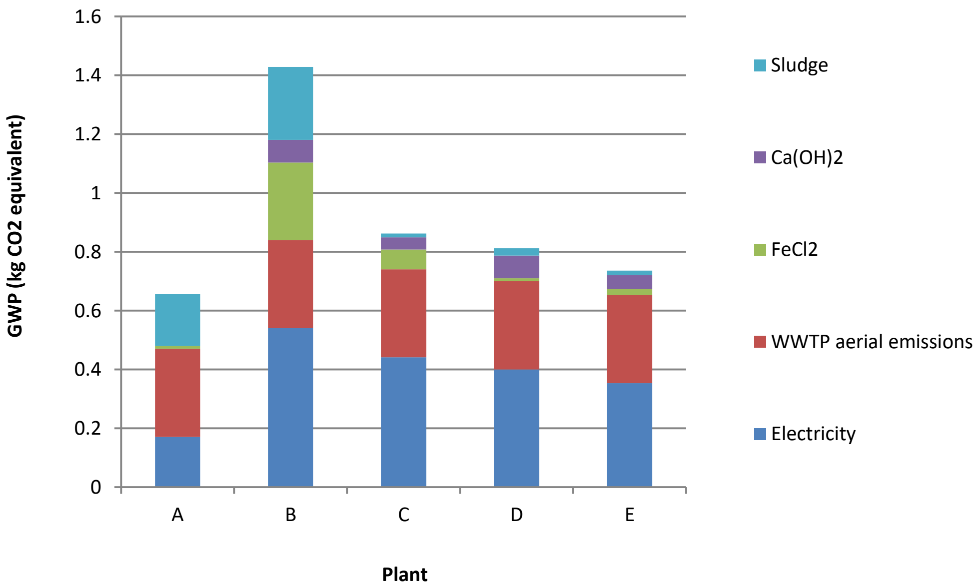

Figure 3-18: Global warming potential (hydraulic functional unit)

[8] The functional unit in the Gallego study has been converted from PE-year to m^3. A hydraulic definition of 1 PE = 200L has been assumed.

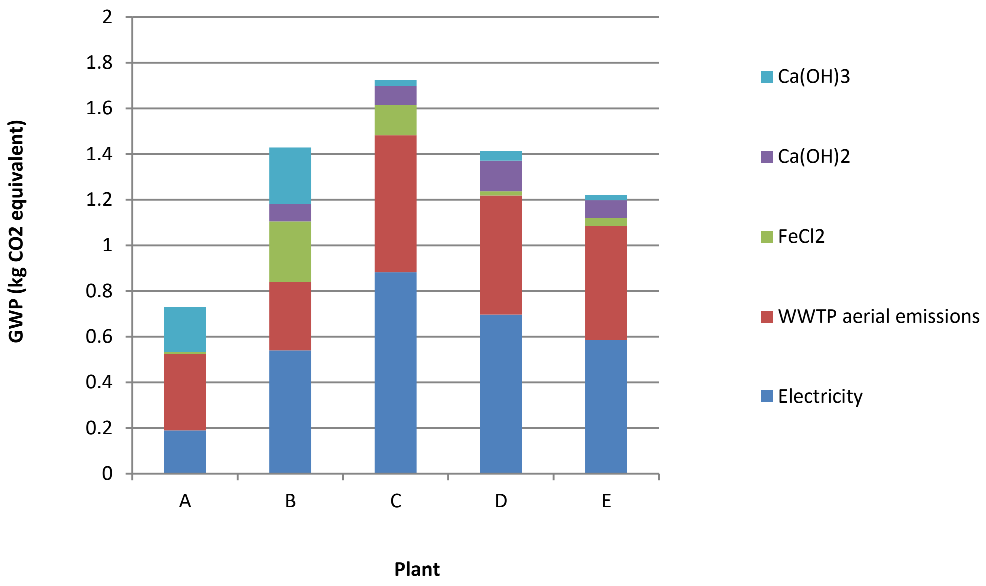

Figure 3-19: Global warming potential (organic functional unit)

Electricity generation is the dominant source of GWP at the freshwater Plants B – E, accounting on average for 50% of the total CO_2 equivalent emissions. Aerial emissions from the unit processes are responsible for most of the remaining GWP at Plants C, D, and E with minor contributions from lime and ferric chloride production, and sludge disposal. Chemicals and sludge disposal account for 42% of the GWP at Plant B. Excluding AD process emissions, aerial emissions at Plant A account for 46% of global warming potential. The remaining GWP at Plant A is divided equally between sludge treatment/disposal, and electricity production. Plant A has a natural gas input of 137 MWh/month. Economically, natural gas is less expensive on a per kilowatt-hour basis than mains electricity (electricity use ≥ 20 < 500 MWh/year = €0.18/kWh, natural gas use < 278 MWh/year = €0.06/kWh [190]). More significantly from an environmental perspective, natural gas production and transport yields less than 4 times the amount of CO_2 equivalent emissions than the mains supply (as per the Irish electrical grid-mix: mains electricity production = 0.59 kg $CO_{2,equiv.}$/kWh, and natural gas = 0.14kg $CO_{2,equiv.}$/kWh). However, the natural gas is used at Plant A to maintain AD temperature and to supplement the CHP plant. According to Hospido et al. [184], 1,291 kg of CO_2 is produced from the digestion of 1 tonne of sludge dry solids (DS). Therefore, the combined CO_2 production for the month

(Nov. 2013) from AD (1,291 kg CO_2/tonne DS x 36.7 tonne DS = 47,379 kg CO_2) and natural gas production (0.14 kg $CO_{2,equiv}$ /kWh x 137,213 kWh = 19,209 kg CO_2) is 66,589 kg CO_2. The additional energy gained from the AD process is 41,897 kWh/month, which equates to 1.59 kg $CO_{2,equiv}$/ kWh. When considering the global warming balance only, and comparing the CO_2 output from mains electricity production of 0.59 kg $CO_{2,equiv.}$/kWh, it could be concluded that the combination of AD and natural gas consumption does not produce a positive reduction in CO_2. However, considering the global warming balance in isolation does not give a true reflection of the overall benefits of anaerobic digestion. The reduction in sludge volume will reduce transport emissions and resource consumption, and AD stabilisation mitigates the impact of lime production.

The impact from $FeCl_2$ production is influenced more by the TP loading than the TP limit. Plants D and E have TP limits of 1 and 0.5 mg/l respectively but the TP loadings at the plants are low enough to consider the GWP impact from $FeCl_2$ production negligible. However, it should be noted that the TP limit at Plant E was being exceeded at the time of analysis. The impact of $Ca(OH)_2$ used for stabilisation is avoided by Plant A because sludge stabilisation is achieved by anaerobic digestion. It has been determined that although the energy reclamation from AD is little more than 10%, and the difference in the overall CO_2 emissions between stabilisation methods is minimal, there are significant reductions in the outputs of other impact categories by reducing the percentage of mains electrical power with the AD process. Polymers did not contribute to GWP, but it is worth noting again that the LCI for the polymers was limited to the production of acrylic acid, and it is probable that a more comprehensive LCI for the actual polymers used on the respective sites may have a bigger impact.

3.6.2.2 Eutrophication potential

Eutrophication potential is the aggregated measure of eutrophying substances in the final effluent and sludge discharges calculated through characterisation factors of phosphorus and nitrogen compounds. Plant E had the lowest EP at < 1 x 10^{-2} kg PO_4^{3-}/m^3 and Plant A had the highest at 1.98 x 10^{-1} kg PO_4^{3-}/m^3 (Figure 3-19). Plant A final effluent N and P data were not available during the research period. Average effluent N and P values had to be estimated from historical data sourced from the EPA for the year 2012, and may not accurately represent current levels at Plant A. Plants B – E had much different EP profiles than that of Plant A because of nitrogen limits at the freshwater plants. Plant B had the highest effluent TN at 50.1 mg/l but also the highest influent TN at 71.5 mg/l twice the value of the next highest influent TN at Plant C (29.6 mg/l). However, Plant B was not required to remove TN, only ammonia. Plant B also had the highest TP influent and effluent concentrations at 7.7 and 1 mg/l respectively. Plant E had the lowest TN effluent concentration at 8.7 mg/l, but also had the lowest influent concentration (13.6 mg/l). Plants D and E influent P concentrations were low (2.7 and 1.8 mg TP/l respectively), and the effluent P concentrations (0.2 and 0.9 mg/l respectively) did not contribute significantly to the EP profile of the plants.

The effect of the discharge limits is that most of the phosphorus in Plant A leaves the plant in the final effluent, while the greater percentage of the phosphorus removed from the freshwater plants' influent leaves in the sludge. There was negligible variation in EP with the change of functional unit.

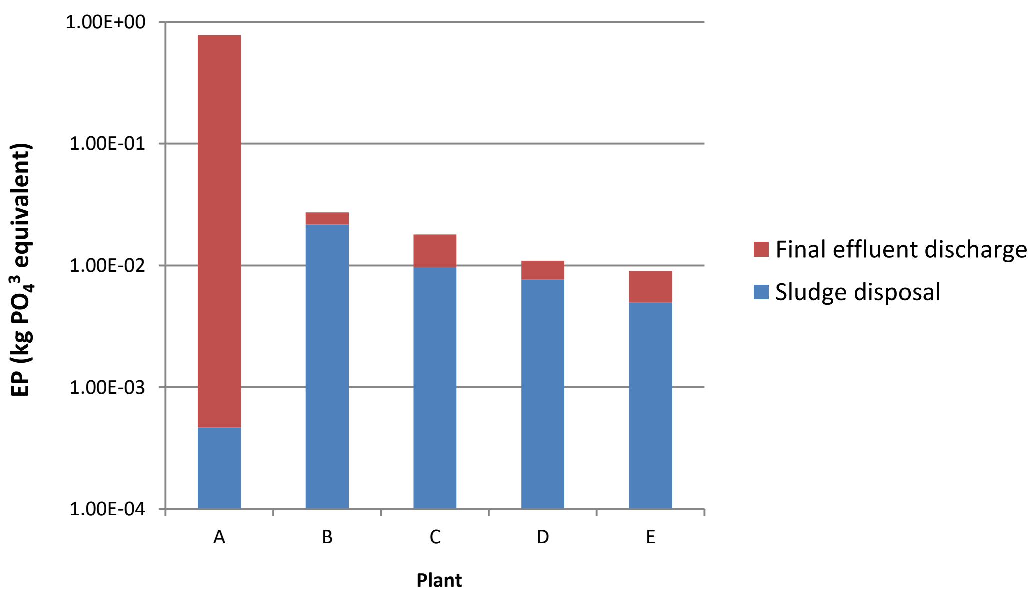

Figure 3-20: Eutrophication potential

There are some limitations in relation to how the EP is presented in the CML methodology. Firstly, the dominant contributor to this category is phosphorus, and the quantity of phosphorus coming into a plant in the influent is assumed to leave the plant either in the final effluent or the sludge outputs (assuming intermediary phosphorus losses are negligible). The EP characterisation factor for phosphorus applied to agricultural land in the CML methodology does not account for any soil, plant, or other biological uptake that may occur between emission point and eventual water body; and therefore, the phosphorus leaving the treatment plant in the sludge outputs presents the same EP regardless of the exit mode. Secondly, despite the different definition for the phosphorus emissions to freshwater and to seawater, the characterisation factors for both are the same.

3.6.2.3 Terrestrial ecotoxicity

The metal concentrations in sludge were found to be the primary source of terrestrial toxicity potential, in particular the concentrations of Cr, Ni, Hg and Zn, which is consistent with the findings from several similar studies [3, 151, 160, 175]. Organic sources of toxicity such as AOX, PAH, PCB, and others (

Table 3-5) were obtained from an EU commission report [104]. However, characterisation factors for these compounds have yet to be developed for the CML LCIA methodology. None of the aforementioned studies make any reference to organic compounds when discussing terrestrial ecotoxicity. This may be because of similar characterisation factor issues, or it may have been determined that their contribution to toxicity potential was deemed negligible when compared with the contribution from the heavy metals. Li et al. [191] concluded that there is a significant relationship between the proportion of industrial wastewater entering the WWTP and the levels of organic compounds in the wasted sludge. In their study of 12 WWTPs in China it was found that the levels of PAHs in the sludge (13.87 – 82.58 mg/kg DS) far exceeded the recommended limit set by the European Union (6 mg/kg DS). This may be due to the extensive use of coal as a source of energy generation in China which is responsible for atmospheric depositions of PAHs. It was reported that the concentrations of organic compounds in the wastewaters are so high that they had overtaken heavy metals as the primary pollution source in sludge in one particular province. Conversely, the EU report [104]stated that increasing scientific investigation has shown that there are no significant environmental consequences associated with PAHs, PCBs, or PCDD/Fs. It may be prudent to carry out further investigation into the impact of organic pollutants in the wasted sewage sludge when the appropriate characterisation factors are developed.

Similar sludge DSCs were reported for Plants A, B, and C (18 – 20%), and it is assumed that the DSCs from Plants D and E are also within this range. Therefore, because the metal concentrations are based on the sludge DSC, the magnitude of TEP is largely a function of sludge volume (Figure 3-20). The sludge produced at Plants B, C, D, and E is lime stabilised before land spreading. It is assumed that this does not affect the metal concentration. The sludge produced at Plant A is sent to a compost company that mixes the sludge with a bulking agent before being spread on land. Assuming that the metal concentrations in the bulking agent are negligible, this produces a dilution effect and acts to reduce the metal concentrations, which results in a significant reduction in the Plant A loading in this impact category.

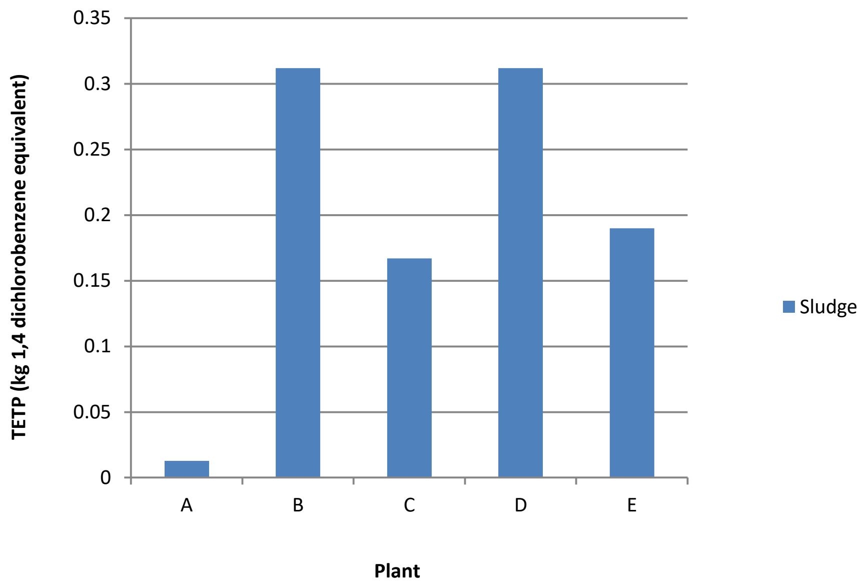

Figure 3-21: Terrestrial ecotoxicity potential

3.6.2.4 Photochemical oxidation potential

Energy and $FeCl_2$ production are the primary contributors to photochemical oxidation potential. However, an anomaly occurs in relation to transport emissions with the PCOP model in the CML methodology (Figure 3-21). The NO (nitric oxide) that is emitted from the vehicle's exhaust reacts with O_3, producing NO_2 and O_2. Therefore, the NO oxidation and the ozone reduction cause a reduction of O_3. This aspect of the PCOP output appears counter intuitive i.e. transport emissions are good for the environment. However, this is a particular situation where regional conditions need to be considered when interpreting this data. Photochemical oxidation occurs most commonly in locations where there are high concentrations of nitrogen oxides and volatile organic carbons (VOCs), and in atmospheres of high sunlight, stagnant air, and low precipitation. None of which are commonplace in Ireland.

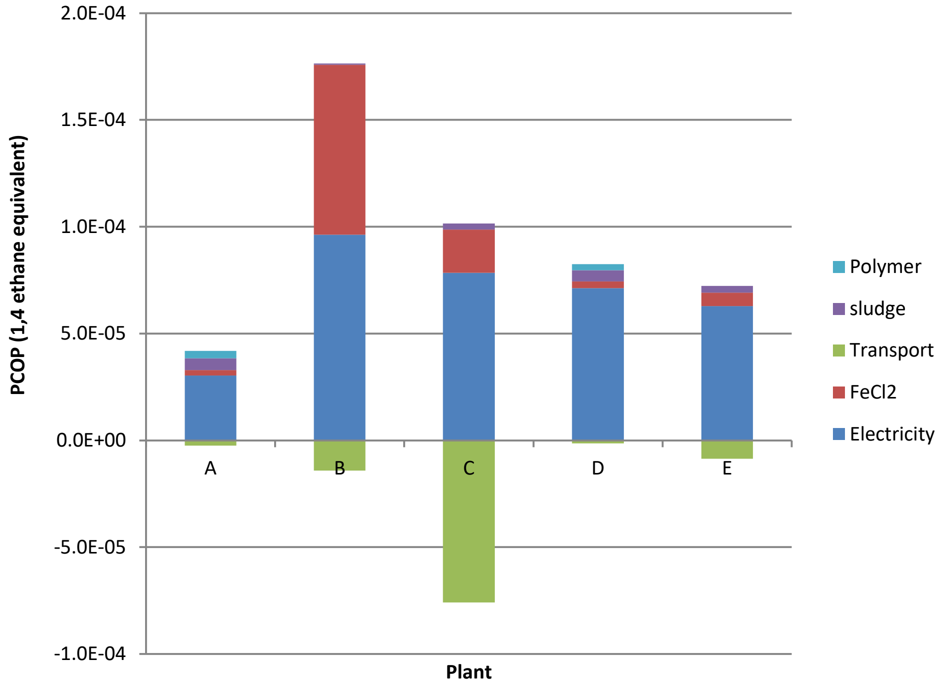

Figure 3-22: Photochemical oxidation potential

3.7 Conclusions

The energy and resource efficiency of the WWTPs were influenced by several variable and fixed interdependent parameters. In general, energy economies of scale were evident across the range of plant sizes depending on the energy metric used. There were exceptions to the trend where two medium sized plants exhibited very high specific energy use. However, this could be attributed in most part to the effect of loading and to a lesser extent the effect of variations in discharge limits. It is difficult to draw any firm conclusions on the effect of variations in discharge limits. The energy efficiency exhibited by Plant A could be attributed to either scale or to the less stringent limits. Determining the exact cause would require a comparative analysis with a plant of similar scale. However, it is worth noting that the effluent BOD, TSS, and COD at Plant A were lower than any of the other plants (5.1, 10.1, and 34.8 mg/l respectively), which means that the plant could operate at shorter SRTs and still achieve compliance. This would reduce energy consumption even further, and therefore, it could be concluded that less stringent discharge limits equates to a reduction in energy requirements. The variation in the TP limits between Plants D and E had very little effect on material use. The phosphorus loading was so low that very little precipitant was required to reach the TP discharge limits, albeit that one of the plants was exceeding the limits during testing. There was little difference in energy consumption between the small plants in terms of kWh/m^3. The difference in terms of kWh/BOD removed was more significant.

It was found that the choice of functional unit was critical in this type of assessment. It could be seen that using the mass of BOD removed as the functional unit produced a different environmental profile than when the functional unit was the volume of influent treated. The effect of the variation in functional unit was most evident in impact categories with a significant electricity input.

The potential environmental cost associated with upstream and downstream processes such as sludge disposal, energy, and chemical production were elucidated, and trade-offs between impact categories within a system's environmental profile were identified. It was observed that actions to reduce regional impacts such as eutrophication and terrestrial ecotoxicity resulted in an increase in global impacts such as global warming and acidification. This places the interests of the local environment in conflict with the interests of the global environment. Efforts to reduce global impact should focus on improving a plant's energy efficiency and minimising chemical inputs. Additionally, at a national level, the impact of energy use can also be reduced by improving the electrical grid-mix through the introduction of more sustainable sources of energy. The impact from chemical use is more difficult to reduce. While biological phosphorus removal is effective in reducing TP to levels of just below 2mg/l, TP limits for freshwater are generally lower and require some level of chemical precipitant.

The use of LCA as a decision support tool has both advantages and limitations. Within the scope of this study, several key system inputs and outputs that contribute to environmental impact were identified. However, at a local level the methodology suffers from a lack of site-specific parameterisation in areas such as pedology, topography, and other geographic and aquatic variances that affect the cause-effect chain of environmental interventions. However, further parameterisation requires knowledge of pre-existing concentrations of background substances and other sensitivities related to the receiving system. Life cycle assessment data acquisition is already an expensive and exhaustive process, and the addition of another level of data collection may perhaps render the entire process excessive and cumbersome. Therefore, a compromise needs to be reached between what could be considered reasonable in terms of accuracy, transparency and value, and the time, resources and overall cost associated with an assessment. Midpoint assessment methodologies are considerably closer to these aims in the sense that accuracy and transparency are maintained, but perhaps at the cost of a reduced value in terms of predicting actual environmental impact at a local level.

4 Methodology

4.1 Introduction

This chapter begins with the rationale for the systems that were included in the study. Details of the LCA and LCCA methods and procedures adopted for the study are then provided.

4.2 Systems

The system assessment methodology and framework presented in this study has universal application. However, for the purposes of demonstration, data availability and acquisition, it was determined that the systems selected for the study should be based on the systems most commonly found on the island of Ireland. A survey was conducted of the 538 registered WWTPs in Ireland (sourced: EPA, 2015). Table 4-1 lists the treatment systems and the percentage of the total that they represent. Suspended growth systems are the most common system type found in Ireland, accounting for almost 60% of all systems. Of this percentage, CAS systems account for over 36%, with EA[9], SBR, IFAS, and MBBR making up the remainder. Attached growth systems (excluding hybrid IFAS and MBBR systems) account for less than 10%. Biofilter, PFBR, and MBR systems collectively account for just over 1%.

Table 4-1: Treatment systems as a percentage of total treatment systems in Ireland

Treatment system	*Percentage*
Biofilter	0.63%
Conventional activated sludge	36.27%
Extended aeration	7.97%
Integrated constructed wetlands	0.42%
Integrated fixed film activated sludge	0.21%
Membrane bioreactor	0.21%
Moving bed biofilm reactor	0.21%
Pump flow bioreactors	0.21%
Rotating biological contactor	5.87%
Sequence batch reactor	13.84%
Trickling filter	2.94%

[9] The percentage of EA systems also includes oxidation ditches. The terms were used interchangeably throughout the survey of plants.

4.2.1 Rationale for system selection

4.2.1.1 Data availability

One of the most significant challenges faced when conducting a LCCA or LCA is the quantity of data that is required. Where sufficient inventory data could not be acquired, or it was felt that the quality of data was such that it compromised the fairness of comparison, or the overall quality of assessment, the system was omitted.

4.2.1.2 Modelling practicality

Modelling practicality applies specifically to ICW systems. The value and objectives of ICW systems are not limited to wastewater pollutant removal. There are several significant qualitative properties that are difficult to include in numerical steady-state system models such as the provision of diverse ecological habitat, or public amenities. These are properties that are better captured with CBA or WLCC models that include externalities and indirect costs. The implementation of an ICW can depend on the potential of the surrounding landscape to provide a platform to achieve these objectives. Furthermore, it is difficult to put a 'per capita' area on such a location specific treatment system. However, it was concluded that a natural system should be included in the analysis to demonstrate the associated economic and environmental benefits. It was considered that a HF-VF hybrid CW would provide a system that could be practically modelled based on the review of the literature.

4.2.1.3 Level of expertise

Level of expertise required, is a significant issue when choosing the most appropriate WWTS for small agglomerations. It is often misrepresented through other qualitative criteria such as robustness, reliability, or ease of use. The term 'ease of use' can be ambiguous as it can refer to systems with minimum control parameters, or highly automated systems. In either case the term would suggest minimum human input. While this is desirable for WWTPs serving small rural agglomerations, it should not be considered as a criterion, but rather as a system component that has to be fit for purpose in much the same way a pump or motor must be sized correctly in order for a system to function efficiently. The level of expertise required can be an immediate deciding factor in cases where the expertise is simply not available, and this is understandable. However, it should not be a deciding factor because of the perceived additional labour cost. The higher labour costs should be included within the cost analysis in the same way that a higher energy or material cost would. The LCCA will then make recommendations on whether or not the additional costs are justified. Notwithstanding the expertise issues, probability of selection is a consideration that emerged from consultation with professionals and local authorities, with particular reference to MBR systems. It was concluded that because of a range of O&M issues with MBRs, the probability of their selection would be very low. It was therefore, decided that they would be omitted from the study. It should be noted however, that MBR systems are operated successfully in many parts of the world. Efficient operation of MBR systems depends on a strict O&M regime and a higher level of expertise that may not always be available. Table 4-2 presents the list of systems selected for the study and provides a brief description of the reasoning for inclusion.

Table 4-2: Selected systems and rationale

Treatment system	*Selection Reasoning*
Single stage CMAS	Provides most basic level of treatment for the lowest capital cost. Provides a good opportunity to demonstrate the effect of high discharge limits on cost distribution
Anoxic oxic (AO)	Illustrates the effect of a TN discharge limit on cost
Anaerobic anoxic oxic (AAO)	Demonstrates the material cost reduction potential of EBPR
Extended aeration (EA)	Elucidates the trade-off that exists between increased aeration costs and reduced sludge management costs
Oxidation ditch (OD)	Demonstrates the effect the type of aeration delivery systems can have on energy costs.
Constructed wetlands (CW)	Illustrates a myriad of economic, energetic and environmental advantages of natural systems implementation in locations where land availability is not an issue
Integrated fixed film activated sludge (IFAS)	Illustrates the benefits of hybrid systems where nutrient removal is required and space restrictions are an issue
Rotating biological contactor (RBC)	Elucidates the difference in cost distribution between suspended growth and attached growth systems
Sequence batch reactor (SBR)	Illustrates the cost benefits of an all-in-one system
Trickling filter (TF)	Elucidates the energy distribution trade-offs that exist between attached growth and suspended growth in an alternative way to the RBC system

4.3 Life cycle assessment model

The preliminary LCA study provided the basis for the DST LCA model. The findings of the study have identified the relevant resource and emissions inventory that is responsible for the greater percentage of the environmental impact. The methodology and LCA framework remains the same, as do the majority of the upstream and downstream inventories. Many of the differences encountered relate to the systems that are included in the model. The preliminary LCA study was limited to the evaluation of activated sludge based systems, whereas the DST model includes natural, attached growth, and hybrid systems that have different forms of energy input, oxygen transfer mechanisms, aerial emission factors, sludge quantities and concentrations. However, the key difference between the interpretation of the LCA results provided by the preliminary study, and those that are produced by the DST is that the estimations of energy and resources in the DST model are based on first principle calculations that may not capture all of the efficiency losses experienced in a real life. In essence, the results that are provided by the DST model represent the impact from the operation of an ideal wastewater treatment system.

4.3.1 Goal and scope

4.3.1.1 Goal

- To provide an LCA model as part of a DST for the selection of small wastewater treatment systems.

4.3.1.2 Scope

Table 4-3 presents details of the program scope.

Table 4-3: Life cycle assessment model scope

Item	*Details*
Number of system types	10
Types of systems	Table 4-2
System design scale range	500 – 2,000 PE
Intended region of DST application	Ireland – rural and urban environments

4.3.1.3 Boundaries

The boundaries of the model are as defined in the preliminary LCA study (Section 3.3.2). Only the use-phase of the systems' life cycles is considered. Life cycle inventories were not available for the production of the growth media used in the TF, RBC and IFAS systems.

4.3.1.4 Functional unit

The functional unit is '1 day of system operation'. The problems surrounding the functional unit in the previous systems' analyses will not be an issue in the DST model. The previous analyses were conducted on existing plants with variable flow rates and composition. The objective of the DST is to evaluate the performance of different systems under similar conditions with similar flows.

4.3.2 Life cycle inventory

The inventory for the DST LCA model is presented in Table 4-4. The LCI includes only use-phase inventory and does not consider the construction phase. The reasons for which are discussed in the following section.

Table 4-4: Life cycle inventory

Parameter	*Quantity*
Inputs	
Influent composition	
BOD (mg/l)	User defined
COD (mg/l)	User defined
TSS (mg/l)	User defined
TN (mg/l)	User defined
TP (mg/l)	User defined
NH_3 (mg/l)	User defined
PO_4^3 (mg/l)	User defined
Electricity	Calculated based on loading and limits
$FeCl_2$	Calculated based on loading and limits
$Ca(OH)_2$	Calculated based on loading and limits
$Ca(ClO)_2$	Calculated based on loading and limits
Polymer (acrylic acid)	Calculated based on loading and limits
Outputs	
Effluent composition	
BOD (mg/l)	Calculated based on loading and limits
COD (mg/l)	Calculated based on loading and limits
TSS (mg/l)	Calculated based on loading and limits
TN (mg/l)	Calculated based on loading and limits
TP (mg/l)	Calculated based on loading and limits
NH_3 (mg/l)	Calculated based on loading and limits
PO_4^3 (mg/l)	Calculated based on loading and limits
Effluent metals	As per Table 1-3
Sludge (kg DS)	Calculated
Sludge metal concentrations	As per Table 1-4
Sludge nutrient concentration	Calculated
Treatment process aerial emissions	As per Table 1-5
Transport emissions	Calculated

4.3.2.1 Additional notes in relation to the emissions inventory

Lundin et al. [158] reported that the impact from the construction phase of the WWTP life cycle becomes more significant at small scales. The study conducted by Machado et al. [173] on a small activated sludge plant (500 PE) found that for most of the considered impact categories the construction phase accounted for around 20% of the total impact. While it is conceivable that the differences in the magnitude of the construction phase contribution to the overall impact could be considered negligible for electro-mechanical systems, the same study found that the construction phase of CW systems accounted for as much as 80% in some categories. However, without a detailed LCI for each system, any estimations of the percentage contribution of the construction phase to the entire life cycle are prone to uncertainty. Therefore, only the impact from the use-phase of each system is considered in the life cycle impact assessment.

Estimations of GHG emissions produced by constructed wetlands are based on the study conducted by Søvik et al. [192]. The study determined the net CO_2 and CH_4 emissions for HF-CW (3.8 g CO_2/m^2, 0.17 g CH_4/m^2) and VF-CW (8.4 g CO_2/m^2, 0.055 g CH_4/m^2) systems. As a simplification, the CO_2 output the HF-VF hybrid CW system is the aggregation of the two emission factors which yields 0.23 kg $CO_{2,equiv.}/m^3$ of treated wastewater based on an average active surface area of 7.44 m^2/PE. The CW CO_2 emissions are 24% lower than those of the electro-mechanical systems at 0.3 kg $CO_{2,equiv.}/m^3$.

As a simplification for quantifying GHG emissions, Monteith et al. [193] considered CAS and attached growth systems to have similar GHG emission rates. It is difficult to determine without further investigation whether similar levels of emissions would occur during the biological process in attached growth and suspended growth systems, or at least be within a small enough range to be considered negligible for the purpose of LCA system comparison. It is conceivable that microbial activity (cell lysis and synthesis) would be similar in both system types, and this would produce relatively similar GHG emissions. Until further data becomes available it is assumed in this study that GHG emissions are similar for both system types.

4.4 Life cycle cost analysis model

4.4.1 Introduction

An overview of the LCCA procedure developed for this study is presented in Figure 4-1.

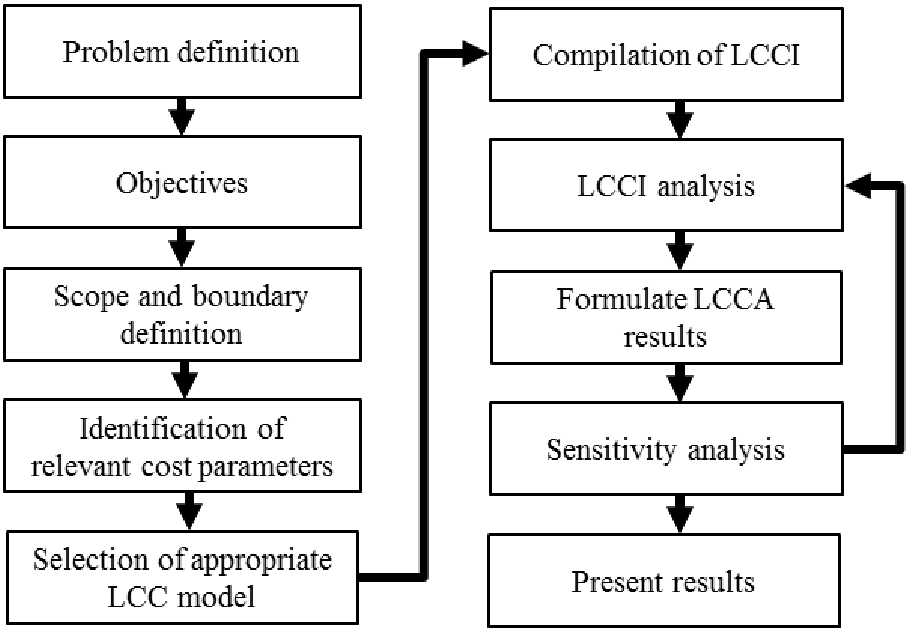

Figure 4-1: Life cycle cost analysis procedure

4.4.2 Procedure

4.4.2.1 Problem definition

Wastewater treatment systems currently in operation will exhibit variable economic performance depending on several systems-specific and site-specific conditions that have been discussed in previous chapters. It is therefore, necessary to develop a methodology to assess the economic performance of these systems in varying conditions. A review of the literature has identified system scale, loading, discharge limits, method of sludge treatment and disposal as the parameters that have the greatest influence on the operational economic profile of a WWTS, and as such, are the primary focus during compilation of the life cycle inventory.

4.4.2.2 Objectives

The objective of this element of the research is to develop a LCCA model for small WWTSs to be included as part of a multi-criteria decision support tool.

4.4.2.3 Scope

The scope of the LCCA is defined in Table 4-5.

Table 4-5: Life cycle cost analysis scope

Parameter	*Description*
System types	(Table 4-2, Chapter 4)
System lifetime	24 years
System scale range	500 – 2,000 PE
Region	Ireland
Audience	Semi technical/technical

4.4.2.4 Identification of relevant cost parameters

The preliminary LCA study identified the environmental cost parameters. Many of these inputs are common to the economic cost inventory. The relevant LCC parameters are distributed as shown below (Figure 4-2). The operational costs are divided between labour, energy, chemicals, and sludge disposal. Capital expenditure includes the aggregated cost of engineering, civil works, construction, electrical and mechanical components, managerial costs, and contingency percentage. The cost of replacement parts applies to large unit replacements such as a blower or RBC motor. Smaller replacement costs generally fall under the operation and maintenance (O&M) category.

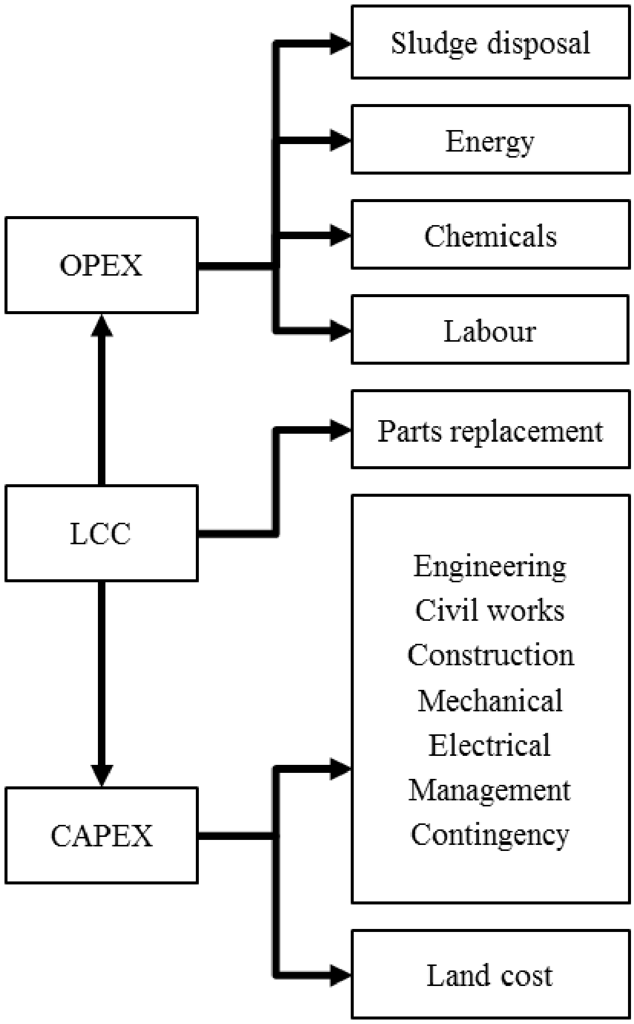

Figure 4-2: Life cycle cost distribution

4.4.2.5 Selection of appropriate LCC model

There are three types of temporal LCC variations that have to be considered in the analysis of wastewater treatment systems: initial capital expenditure (CAPEX), recurring costs i.e. operation and maintenance expenditure (OPEX), and one-off replacement costs. The CAPEX is assumed to be the total cost of the project from the start the of the procurement process, through pre-engineering, design, and construction, to the first day of operation. Depending on the scale, and anticipated duration of a project, a contractor may choose to include an inflation rate in a tender application. Considering the plant scale range involved in this study it is assumed that a plant can be constructed in one year and that the project cost estimation provided by the contractor does not include an inflationary cost factor. Therefore, a discount rate needs to be applied to the CAPEX to account for depreciation that occurs between the time of initial project cost estimation to the time of operations; assumed here to be one year. This value is calculated using the single present value (SPV) method (Eq.10). The SPV method applies to a one-off payment that occurs sometime in the future. This method is also used to account for large unit replacement parts that occur within the lifetime of the system.

$$SPV = \frac{C_o}{(1+d)^n} \qquad \text{(Eq. 10)}$$

Where;

C_o = original cost at the base year

n = number of years from the base year

d = applied discount rate

Annually recurring O&M costs are calculated with the uniform present value (UPV) formula (Eq. 11)

$$UPV_{OM} = \sum A_{O,i}\left(\frac{(1+d)^n - 1}{d(1+d)^n}\right) \qquad \text{(Eq. 11)}$$

Where; $A_{O,i}$ is the annual recurring cost of the O&M element i, at base year 0. In the study conducted by Rawal and Duggal [89], recurring energy costs were treated separately from other O&M costs. This relates to the volatility in the cost of energy. In recent years, changes in the cost of energy has not aligned with construction cost indices (CCIs), and a separate discount rate for energy should be used (Eq.12).

$$UPV_E = A_O \frac{(1+d)^n - 1}{d(1+d)^n} \qquad \text{(Eq. 12)}$$

Hence, the total LCC of a WWTS is given by Eq. 13.

$$LCC = \sum (SPV + UPV_{OM} + UPV_E) \qquad \text{(Eq. 13)}$$

4.4.2.5.1 Discount rate

The test discount rate (real discount rate[10]) for OPEX is 3.5%. This is in accordance with the requirements of The Public Spending Code [194]. As stated previously, energy has been assigned a separate discount rate because of the volatility of energy prices. Energy discount rates vary significantly depending on mode of energy generation, and energy consumer or sector. The EU Commission recommend an energy discount rate of 12% (Table 4-6). However, a 12% discount rate creates a large gap between the LCCs of energy intensive and non-energy intensive systems. Hence, a 5% discount rate has been adopted here as per the studies conducted by Rawal and Duggal [89], and Pretel [169]. In practical terms, the selected discount rate is an important and determining factor during an actual assessment process; however, it is not critical for the purpose of demonstrating the assessment methodology proposed in this study. It is recommended that an actual life cycle cost analysis should include sensitivity analyses to assess the effect of variations in the energy discount rates.

Table 4-6: Current and projected energy discount rates differentiated by sector or consumer. Adapted from [195]

Consume/sector	*Year 2015*	*Year 2020 - 2050*
Power generation	9%	9%
Industry sector	12%	12%
Tertiary sector	11%	10%
Public transport	8%	8%
Truck/inland transport	12%	12%
Private cars	17.5%	17.5%
Household	17.5%	12%

4.4.2.5.2 System lifetime

Wastewater treatment system lifetimes in the literature vary from 21 [89] to 40 [88] years. Systems with large initial capital investment and low operational costs will suffer from short nominal lifetimes in a LCCA because it takes longer to realise the benefits of the large initial capital investment. Conversely, systems with larger operational costs will suffer with longer

[10] As opposed to a nominal discount rate that includes the effect of inflation.

lifetimes. There are two possible approaches to this problem. The first approach is to determine the system with the longest lifetime and use this as the base lifetime for all systems. The cost of maintaining the other systems to reach this lifetime is then included in the replacement parts component of the LCC model. However, this requires detailed knowledge of the replacement regimes of many different system components, and may include having to make estimations of future capital investment in large structural components that have reached their end of life phase and require replacement. Furthermore, the rate of technological development in the field of wastewater treatment coupled with increasing water quality requirements may suggest redundancy in long lifetime systems as new technologies introduce improvements in efficiency. A less speculative approach is to decide upon a relatively short lifetime e.g. 20 years, estimate a depreciation factor to assess the residual value of the plant at the end of the nominal lifetime, and then calculate the residual SPV based on the estimated depreciated value. This appears to be a more rational approach, but is still subject to uncertainty regarding the depreciation factor estimations. The depreciation factors are system dependant. Large surface area systems such as CWs will not experience the same rate of depreciation as an electro-mechanical system because the greater percentage of CW CAPEX is the cost of land, which does not depreciate because it is considered to have an unlimited useful lifetime. Rawal and Duggal [89], determined depreciation values for large [12 million litres per day (MLD)] suspended growth, and attached growth (trickling filter) systems of 7% and 6.2%/year respectively. The waste stabilisation pond (WSP system) depreciation value was estimated as 1.2%/year. Further investigation is required to assess how the depreciation values are affected by scale. Notwithstanding the uncertainty in depreciation rates, it is felt that the latter approach is more practical and less prone to uncertainty. Hence, a system lifetime of 20 years has been chosen for this study.

4.4.2.6 Capital expenditure

Capital expenditure inventory is limited to aggregated project cost data sourced from academic literature and engineering reports (Table 4-7). The reported cost data includes the cost of engineering, civil works, electro-mechanical equipment for inlet works, primary and secondary treatment, sludge dewatering, chlorination and the inclusion of a 15% contingency for unforeseen costs.

Table 4-7: Sources of CAPEX data

System	*Year of publication*	*Region*	*Source*
Single stage CMAS, AO, AAO, RBC, SBR	1998	United States	Foes et al. [87] (Appendix C.1)
TF, OD	2006	Greece	Gratziou et al. [88]
IFAS	2003	United States	Johnson [6]
EA	2002	Greece	Tsagarakis et al. [196]
HF –VF CW	2014	Greece	Gkika et al. [25]

The CAPEX values were normalised as much as possible. Where variations existed between different sources regarding elements included in the aggregated CAPEX totals, adjustments were made accordingly e.g. a system's CAPEX may not have included sludge dewatering, in which case estimations were made for dewatering based on a percentage of the total capital expenditure. Deductions were made where the cost of land was included in reported CAPEX for each system. This was done to facilitate the inclusion of the cost of land in Ireland based on the calculated surface area requirements for each system. Temporal and locational normalisation to the Irish context was carried out with Eq. 14.

$$\mathrm{C}_c = \left(\frac{I_c C_t K_l}{I_t}\right) \times ER_l \qquad \text{(Eq. 14)}$$

Where C_c is the current cost of the system, I_c is the current construction cost index (CCI), I_t is the construction cost index at time *t* of plant construction, C_t is the cost of construction at time *t*, K_l is the location factor (Ireland – U.S. location factor 2015 = 1.3 [197], Ireland – Greece location factor was unavailable, assumed factor of 1)[11], ER_l is the currency exchange rate (€ - US$, 2015 ≅ 0.9).

4.4.2.6.1 Construction cost indices

The CCI monitors changes in cost of construction projects, materials and labour over time. It is reported relative to a nominal base number in a previous year e.g. Jan 2001, index = 100; Jan 2015, index = 356. Figure 4-3 presents the U.S. average CCI from 2001 to 2011 [198].

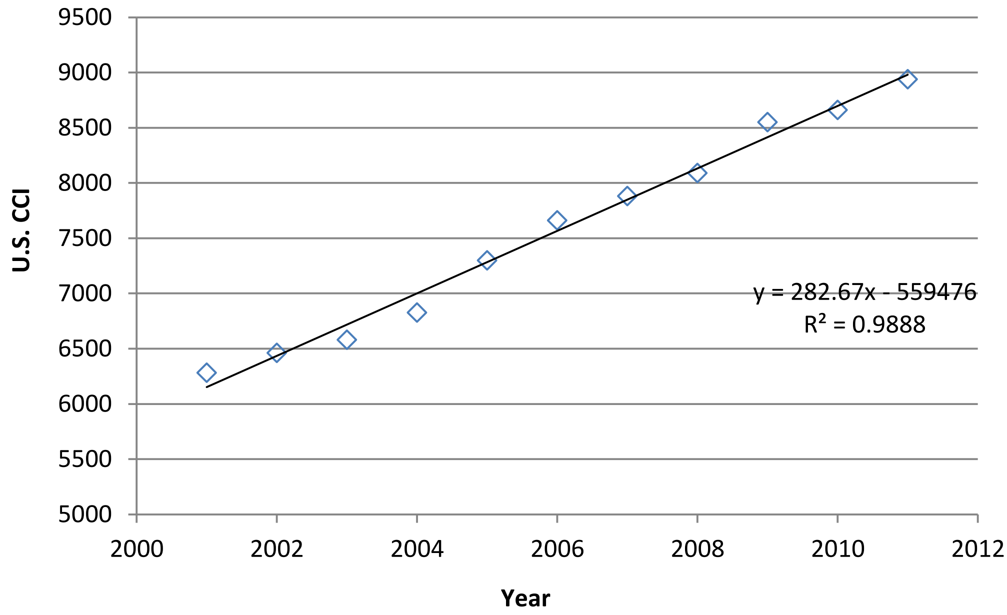

Figure 4-3: Average United States construction cost index history from 2001 to 2011

In the European Union (EU) the CCI is sometimes referred to as the *construction factor price index* and is an EU business cycle indicator giving temporal construction cost indices for each member State and an average CCI for across the 28 EU states [199] (Figure 4-4).

[11] The location factor normalises the differences in cost of construction between countries

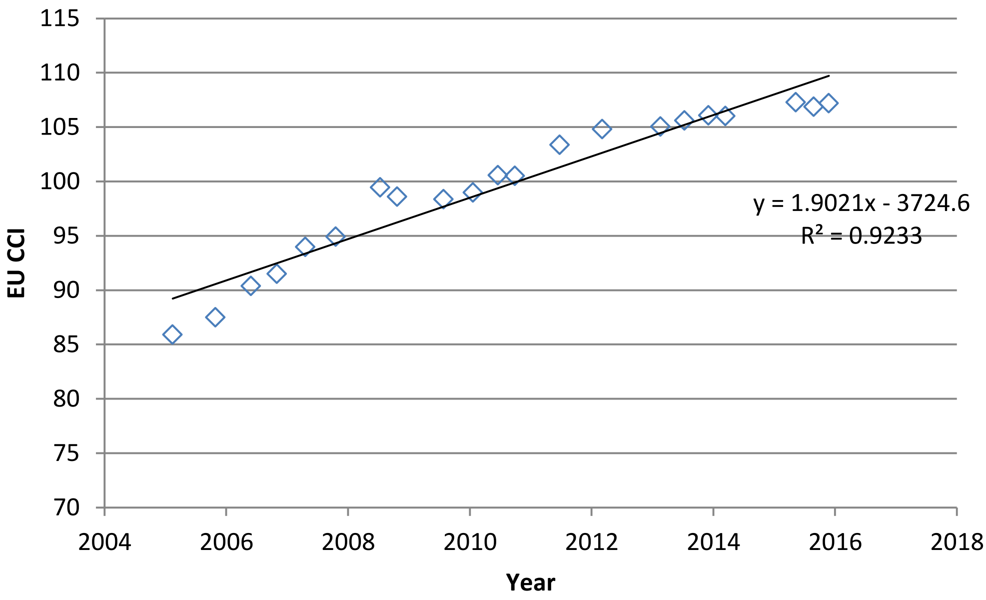

Figure 4-4: European Union (28 member state) CCI. 2005 – 2016. Adapted from [199]

4.4.2.6.2 Land cost

The specific cost of land in Ireland varies between rural and urban locations, and between farmland sites and development land. A value of €20,000/acre (~ €5/m^2) has been assumed for the study. In the case of greenfield sites, it is assumed that land earmarked for development may incur less cost implementing support-utility infrastructure such as roads, water and power, but may encounter greater legal resistance depending on future plans for neighbouring development spaces. Specific surface area calculations are provided in Appendix C2.

4.4.2.6.3 Cost curves

Power law regression CAPEX cost curves were developed from the normalised cost data. The expressions calculate the cost as €/PE (Table 4-8).

Table 4-8: Normalised capital expenditure cost curves (x = PE)

System	*Cost curve (€/PE)*	*R^2*
AO	$124589x^{-0.624}$	0.9825
AAO	$143261x^{-0.641}$	0.9812
CMAS (single stage)	$72800x^{-0.594}$	0.9771
CW[12]	$470.54x + 26700$	0.9291
EA	$72329x^{-0.888}$	0.9981
IFAS	(Eq.15)	
OD	$(5 \times 10^6)x^{-0.852}$	0.99
RBC	$86781x^{-0.534}$	0.9794
SBR	$185602x^{-0.534}$	0.9819
TF	$(1 \times 10^6)x^{-0.741}$	0.9977

Total project CAPEX data for IFAS systems were unavailable. However, in the study conducted by Johnson [200], the additional cost of upgrading a CAS system to an IFAS system was determined and presented as a function of the aeration tank volume. The additional cost elements included the plastic attached growth media, adjustment of the aeration system, and the media restraining sieves.

$$\mathrm{IFAS}_{CAPEX} = [72800(PE)^{-0.594}] + \frac{[2556(V_{tank})(Ff_{media})]}{PE} \qquad \text{(Eq. 15)}$$

Where,

V_{tank} = aeration tank volume

Ff_{media} = media fill fraction

PE = population equivalence

4.4.2.7 Validation

Benchmark data for electro-mechanical systems CAPEX validation was provided by the WWTS project costs published by Response Group [201]. The systems included in the publications

[12] Constructed wetlands cost curve presents total project cost

were limited to CMAS, EA, OD, SBR, AO and AAO. Project costs for TF, RBC, IFAS and MBBR systems could not be obtained. Good correlation was observed for systems greater than 2,000 PE (Figure 4-5) (average error < ± 5%, PE > 2,500), however, from 2,000 - 500 PE, error percentage ranged from 5 – 25%.

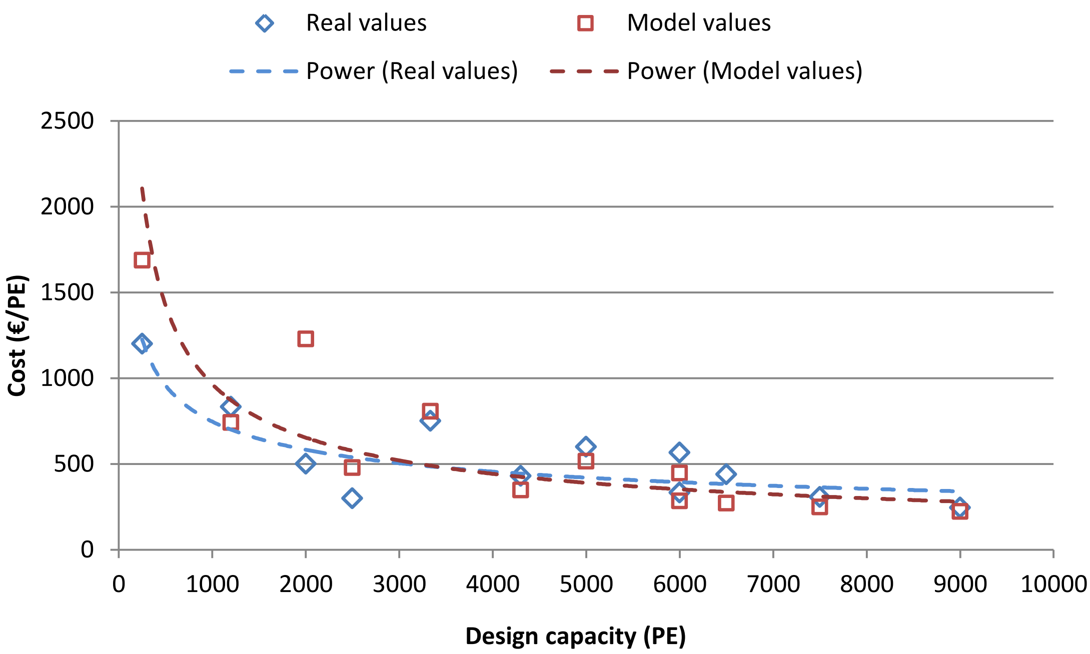

Figure 4-5: Capital expenditure validation. Specific cost per capita as a function of design capacity

4.4.2.8 Operational costs

Operational cost is distributed between energy, labour, sludge management, and chemicals. Details of sludge and chemical quantity calculations are provided in Appendices C.3 and C.4 respectively. Energy is discussed in Chapter 5. The cost of labour is unique to LCCA and is discussed here.

4.4.2.9 Labour

Labour is a significant cost element for small wastewater treatment systems. Estimations of labour percentages range from 35 - 57% of the total operational cost [95]. The type of labour required includes operators, engineers, lab technicians, and helpers/yard workers. The magnitude of estimated labour cost is influenced by

- system type;
- level of expertise required;
- location;
- scale; and
- specific salary scales (also location dependant).

Empirical labour cost data were unavailable for systems in Ireland, either in terms of hours, level of expertise, or specific salary. The specific salaries (€/hour) are not a critical issue as values can be user-defined for regional variation. The values that are in included in the study have been gathered from various career and job websites. Expertise level is difficult to quantify with any direct numerical traceability, and is often weighted simply as low, medium, or high. However, it is difficult to relate these types of indicators to an exact level of profession, and associated cost. The approach adopted for this study is based on the report published by the *New England Interstate Water Pollution Control Commission* (NEIWPCC) [98]. The data in the report were gathered from a survey of 50 WWTPs of varying system and scale. The report details the hours spent per year on individual components and unit processes for a given system (Appendix C.2). The hours are given as a function of discrete plant scales: 0.25 MGD (1136 m^3/d), 0.5 MGD (2273 m^3/d) and 1 MGD (4546 m^3/d). Values for hours spent on certain unit processes that are used in the report are constant with scale and may not reflect the hours required for very small systems (< 90 m^3/d). However, it should also be noted that not all O&M tasks will vary with scale. For example, the time spent maintaining a foul pump for a 1,000 PE plant may not be much less than the time spent maintaining a foul pump in a 2,000 PE plant. Therefore, it is reasonable to assume that some tasks will remain close to constant with plant scale. For O&M elements that vary with scale, linear regression models for annual labour hours as a function of flowrate were developed from the data provided in the report and extrapolated to cover the plant sizes considered in the current study.

Labour type is divided into four categories: operator, maintenance, laboratory, and helper/yard-hand. It is assumed that maintenance on motors, pumps and other electro-mechanical equipment are carried out by an engineer. There are inherent difficulties in assigning specific salaries to each labour category. Salaries will vary between the public and private sector, location, and with different levels of experience. The values used in the study are based on a survey of a number of different employment and salary scale websites (Table 4-9).

Table 4-9: Labour categorisation, description and assumed cost

Labour type	*Description*	*Cost per hour (€/h)*
Operator	General operation	18
Engineer	Technical maintenance, operation and trouble shooting	24
Lab technician	Carries out water quality analysis	18
Yard hand	Carries out low level tasks such as grass mowing, painting, rust removal	10

4.4.2.10 Replacement parts

The frequency and cost of parts replacement is system specific. The cost and frequency of parts replacement included in this study is based on the values reported by Rawal and Duggal [89]. The system types are limited to a suspended growth (CAS), attached growth (TF), and natural (WSP). A simplification has been made here that assumes similar replacement frequencies and associated costs based on the system classification. The author is aware that this is a broad assumption; however without a detailed inventory of the components of each system it is impossible to make a more accurate estimation. Details of the assumed parts replacement are presented in Table 4-10.

Table 4-10: Parts replacement details

System	*CAPEX (%)*	*Replacement frequency (years)*
Suspended growth	5.5	8
Attached growth	9.25	8
Natural	3	8

4.5 Summary

A methodology to assess the economic and environmental costs of small WWTS implementation has been presented here. It was determined through a review of the literature that the best approach was to evaluate these costs from a life cycle perspective, because this is the most effective way of understanding the true cost of system ownership. Environmental LCA and economic LCCA were determined to be the most appropriate assessment tools to achieve this objective. Many of the specific cost and emission factor data used in the methodology are specific to Ireland; however, the framework has universal application and any Ireland-specific data can be replaced with data specific to any given region.

Ten of the most commonly found system types in Ireland were selected for inclusion in the DST to demonstrate the application of the methodology. Their selection was based on several factors including data availability, modelling practicality, and diversity in function and configuration. The included systems provide representation for the four main categories of WWTS, namely; suspended growth, attached growth, hybrid, and natural. The variation in system types provides a good platform to illustrate variation in system performance under different site-specific conditions.

The dual assessment methodology presented here is based on the assertion that much of the environmental and economic cost can be attributed to process flows common to both cost types, namely; energy, chemicals, and sludge disposal. Therefore, quantifying these flows along with the other flows unique to LCCA and LCA such as labour and process emissions respectively provides a solid basis for system assessment and comparison.

The adopted procedure and findings from the preliminary LCA study provided the basis for the DST LCA model by determining suitable boundaries, assessing potential functional units, identifying the key system inputs and outputs, and determining some of the Ireland-specific emission factors such as the average heavy metal concentrations in final effluent discharge and sludge. The critical difference between the preliminary LCA study and the DST LCA model is the processes flow data (energy, chemicals, and sludge), from the perspective that the first study is empirically based whilst the second is mostly theoretical. The most significant aspect of this is that the theoretical specific energy consumption values are lower than those recorded at the various plants. A more detailed discussion on energy is presented in the next chapter. Additional emissions, not relative to the preliminary study, were included in the DST LCA model. The aerial emissions from CW systems were estimated from literature sources, and the aerial emissions from attached growth and hybrid systems were assumed to be similar to those of suspended growth systems, but with the assertion that further investigation is required to assess the accuracy of this assumption. The LCIA methodology selected for the preliminary study was considered to be the most suited to the DST LCA model, but there is a general acknowledgement that in order to improve the value of the LCIA, a greater level of site-specific parameterisation is required.

The LCCA model described in the methodology follows tried and tested procedures that facilitate variations in energy and OPEX discount rates, system depreciation and lifetime. One limitation of the study pertains to the lack of a detailed CAPEX inventory. This effects estimations of the replacement parts cost and maintenance regimes. This is not a weakness of the methodology, but rather a constraint due to a lack of available site-specific data. The data limitation also has consequences for the LCIA as it is known that the construction phase of the environmental life cycle is of greater significance for small wastewater treatment systems. While this does not affect the demonstration of the methodology, it should be considered during the interpretation of both sets of results.

5 Energy

5.1 Introduction

Wastewater treatment accounts for over 1% of the total energy consumed in most of the developed world, and as much as 3% in the U.S. [202]. It is expected that the increase in global population combined with more stringent discharge regulations will see these figures increase in the coming years. Energy use is a central theme in both the economic and environmental cost assessments of most types of wastewater treatment system. Specific energy use can vary significantly depending on scale, system type, desired effluent quality, and site-specific conditions. From an economic perspective, energy consumption can account for a significant percentage of the overall cost of operating a treatment plant. In Europe, values of WWTP energy consumption can vary widely from state to state, and within a state. In Central and Eastern Europe, the cost of water and wastewater management attributed to energy use can be as high as 70% of total operating cost (Appendix D.1) [203]. The cost of energy becomes more significant as WWTP sizes decrease and the specific energy use per volume of wastewater treated or mass of substrate removed increases. Results from energy auditing of the electro-mechanical systems in the preliminary LCA study indicated that plants below agglomeration sizes of 2,000 PE tend to exhibit an exponential increase in specific energy use as the agglomeration size decreases. Constructed wetlands are an exception to this as the relationship between agglomeration size and specific energy use generally tends to remain linear. From an environmental perspective, the preliminary LCA conducted in this thesis determined that energy consumption is one of the main contributors to the overall environmental profile of a treatment plant. This finding is consistent with similar published studies [151, 156, 175, 204]. It should be stated, however, that the magnitude of environmental impact from energy consumption is as much a function of how the energy is produced, as quantity consumed. European Union member states with strong renewable energy programs such Norway, Iceland and Austria will generally have a much lower environmental impact because of energy consumption than states such as Hungary, Luxembourg and Malta (Figure 5-1).

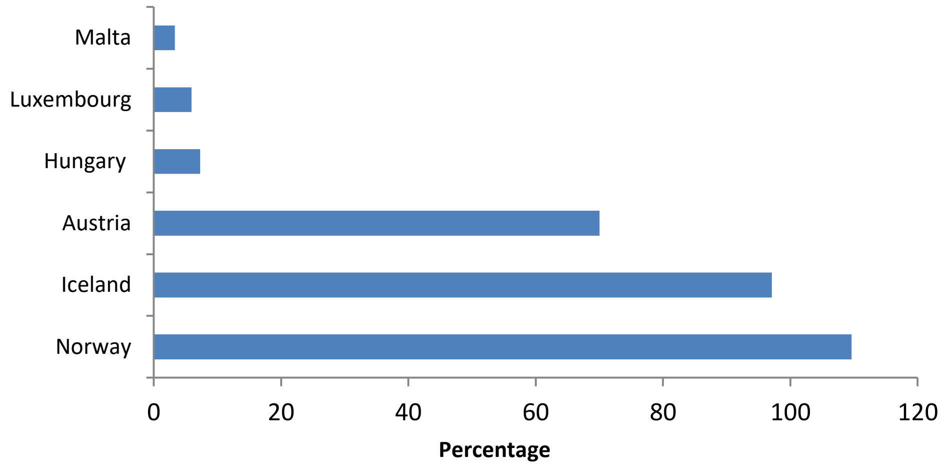

Figure 5-1: Percentage of electrical grid mix sourced from renewable energy [205]

5.2 Factors influencing energy consumption

It is difficult to suggest an average energy value that represents wastewater treatment as a whole because of the multitude of wastewater treatment parameters that influence energy consumption. System type, scale, climate, geography, topography, hydraulic load, organic and inorganic load, discharge limits, expertise availability, sludge management options, and plant design can affect the quantity of energy use. The following sections provide a brief overview of some of the key parameters that influence energy consumption.

5.2.1 System

The type of treatment system will have varying degrees of influence on the amount of energy consumed, and how the energy is distributed across a system. The preliminary LCA study identified aeration as being the primary energy sink in suspended growth (activated sludge) systems. The energy use attributed to aeration can range from 30 – 75% of the total energy consumed at a treatment plant depending on the desired final effluent quality [206]. Oxygen transfer efficiencies (OTEs) of many of the submerged diffused aeration systems are generally quite low due to the large percentage of oxygen that is lost into the atmosphere. Surface aerators such as those used in Orbal ODs have even lower OTEs, but these types of aeration systems are slowly being phased out and replaced with submerged diffusers as the emphasis on energy efficient systems increases.

Unlike suspended growth systems where oxygen is delivered to the microbial population; attached growth systems such as TFs and RBCs deliver or expose the microbes to atmospheric air. The primary energy sink in trickling filter (TF) systems is the pumps that are used to elevate and distribute the wastewater over the growth media. The motors that drive rotating RBCs are responsible for most of the energy consumption in these systems. Proponents of attached growth systems will often refer to the reduced energy consumption when compared with suspended growth systems. However, savings in energy costs are often achieved at the expense of some other aspect of plant performance, or other cost components elsewhere in the system. For example, trickling filter systems are reported to have lower specific energy consumption than activated sludge systems [28]. However, trickling filters in isolation are limited in the level of effluent quality that can be achieved [207], specifically when nutrient removal is required. Similarly, RBC systems can achieve high BOD removal efficiencies with minimal energy input, but to achieve nitrification several more stages are necessary in the RBC train, which requires additional motor power.

Natural treatment systems such as constructed wetlands (CW), reed beds and waste stabilisation ponds are low energy input systems. Some of the smaller natural systems can be considered 'zero energy' systems[13]. However, depending on topography, some degree of pumping may be required to elevate influent. Some constructed wetland systems may incorporate preliminary treatment and some degree of sludge pumping that would require low levels of energy. Aerated lagoons can be energy intensive but there is a reduction in CAPEX because aerated lagoons can be built deeper than other natural systems, which reduces the surface area requirements.

[13] The term "zero energy" here refers to day-to-day energy inputs and does not account for energy used for sludge pumping and transport. These energy inputs are deemed infrequent enough to be considered negligible.

5.2.2 Scale

It has been reported that there are energy economies of scale to be achieved with wastewater treatment systems. There are several identifiable reasons for this such as pump and motor efficiencies, larger pipe diameters incur less frictional headloss, and small systems are subject to relatively greater magnitudes of influent flow variation. There are some suggestions the aeration tank depth can influence standard oxygen transfer efficiency (SOTE), because increasing diffuser depth will increase substrate-bubble contact time, thus, reducing the required airflow and the loss of oxygen to atmospheric air. However, Pöpel and Warton [208] argue that this does not necessarily reduce energy requirements. Aeration tank designs for conventional activated sludge systems generally specify tank depths of between 4 and 6 meters, and there is a developing trend when seeking to increase plant capacity to increase the depth of the aeration tank, in some cases to between 8 and 10 meters [209]. However, an increase in submergence would also increase hydrostatic pressure at the liquid – diffuser interface. According to Casey [123], SOTE as a function of diffuser depth can be approximated with Eq. 16.

$$SOTE_{D_s} = SOTE_4 \left(\frac{D_s}{4}\right)^{0.75} \qquad \text{(Eq. 16)}$$

Where D_s is the diffuser depth, and 4 is the reference depth of 4 m. This relationship is illustrated graphically below (Figure 5-2). This would suggest significant achievable increases in SOTE and thus, a reduction in energy consumption. Small WWTSs cannot take advantage of this energy reducing potential, because aeration tank volumes are determined by hydraulic and organic loading, and for very small loads it may not be practical to have large tank depths.

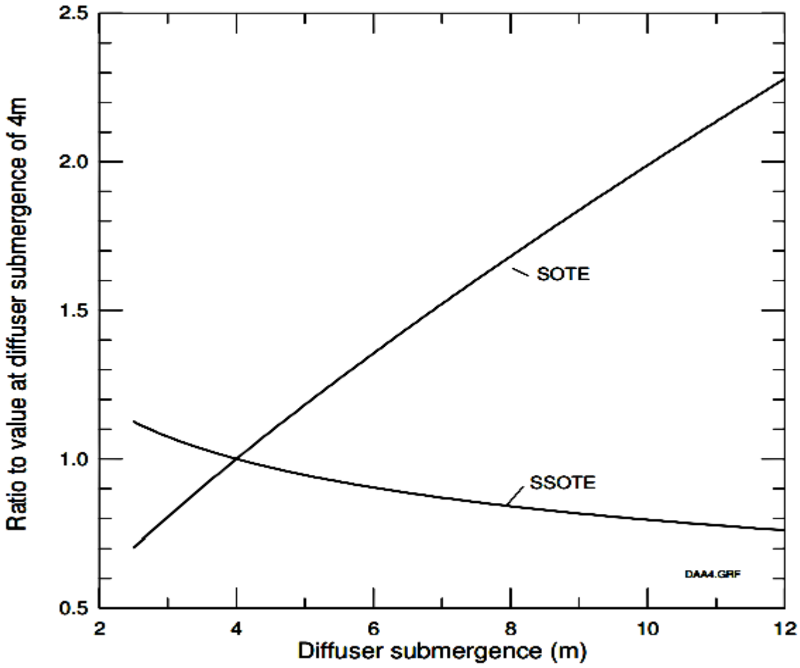

Figure 5-2: SOTE ratio of value to submergence at 4m as a function of submergence depth [123]. Standard specific oxygen transfer efficiency (SSOTE, % oxygen absorbed /m)

5.2.3 Loading

There is theoretical and empirical evidence that demonstrates the relationship between energy consumption and organic loading (g BOD/m^3). The preliminary LCA study illustrated direct correlations between organic load and the percentage of total plant energy use attributed to aeration. It was also observed that the variation in hydraulic loading had the lesser effect on energy consumption. Of the two smaller plants assessed in the study, the hydraulic load at Plant E was almost 3.5 times that of Plant D, but was consuming 11% less energy (0.8 kWh/m^3). Attached growth systems exhibit similar correlations between organic load and energy consumption. Trickling filter media bed volumes will increase with an increase in organic load, thus, increasing pipe lengths and headloss. The required RBC disc media surface area is determined by organic loading. Energy consumption of natural systems is the least affected by loading.

5.2.4 Discharge limits

Discharge limits will affect energy consumption in different ways depending on the type of system being employed. In general, lower limits equate to an increase in energy for conventional suspended and attached growth systems. Each additional substrate type that is required to be removed introduces the potential for an increase in energy consumption. It has been reported that ammonia removal can be responsible for up to 50% of a plant's total energy consumption [210]. A denitrification limit will require additional energy for mixing and nitrate return pumping; and depending on the type of system, a phosphorus limit may require chemical dosing pumps, and produce over 30% more sludge that has to be pumped, thickened and dewatered. In addition, lowering the limits of BOD removal will increase energy consumption. The specific energy (kWh/kg BOD removed) required to remove 90 – 99% of BOD is much greater than the energy required to remove 0 – 90% (Figure 5-3). As mentioned previously, the addition of a nitrification limit in an RBC system will require additional stages in the train. As the ammonia limit is reduced more stages are required and thus, more energy is consumed by the motors. In general, an increase in a substrate removal requirement equates to an increase in energy for most types of treatment system.

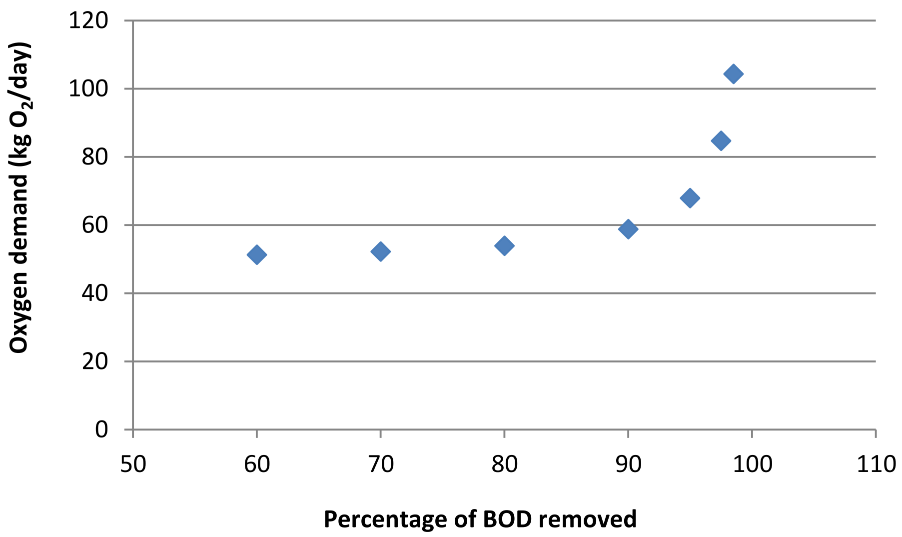

Figure 5-3: Oxygen demand as a function of required BOD removal (CMAS system, influent flowrate = 400m^3/d, primary effluent BOD concentration = 200 mg BOD /l)

5.2.4.1 Problem definition

Energy use accounts for a significant percentage of both the economic and environmental life cycle inventories of the electro-mechanical wastewater treatment systems. It is therefore, important for both the LCIA and LCCA to provide an accurate representation of the quantity of energy used by each system in a variety of site-specific conditions. There are two approaches that have been considered for this problem. The first was the empirical approach. Gathering systems-specific and site-specific empirical energy data from existing plants will provide realistic data that captures the multitude of energy losses that occur from operational inefficiencies, motor, blower, and pumping efficiencies, effects of flow variations, piping and plant wear, and other losses that are difficult to identify or predict. However, the practicality of the data collection and normalisation exercise involved cannot be underestimated. In order to provide the specific energy use of 10 different treatment system with variations in 4 discrete scales, 4 variations of discharge limits, and 3 sludge treatment/disposal options would require a survey of 480 plants, and this figure provides a sample size of one energy datum per scenario. This level of differentiation may seem excessive, but without knowledge of how each system will perform in each of these conditions it is impossible to identify the benefits, limitations and trade-offs that exist between each system, both economically and environmentally. The second approach is to estimate the energy use based on a combination of first principle calculations and empirical data. This approach is limited in the sense that it cannot capture the range of inefficiencies that have been mentioned, but it does provide more robust numerical traceability. It is also thought that this method will provide a better platform for the comparison of energetic performance, and may help identify the energy losses within a system. Therefore, the approach adopted here is to develop system-specific energy models to quantify the energy components of the respective economic and environmental life cycle inventories.

5.3 Energy modelling

Systems energy use has been calculated with varying degrees of complexity. At the most basic level, energy values for some of common unit processes (inlet screens, mixers, sludge dewatering units) will vary with flowrate, or organic loading only. These units are low energy consumers and in some cases account for less than 1% of total plant energy-use. There is some evidence of economies of scale with certain unit processes; however, the values only become significant over a larger scale-range than the one used in the current study. Energy sinks such as pumping and aeration account for a greater percentage of a plant's total energy use, and require a higher degree of parameterisation for calculation. Methods for calculating energy use were adopted from a number of sources [25, 28, 114, 211] to account for a range of site-specific variability. Considering the plants scale range adopted for the study, it is assumed in all scenarios that anaerobic digestion is not economically feasible. Table 5-1 outlines the energy sinks included for each system in the study.

Table 5-1: Wastewater treatment system energy sinks

	AO	AAO	CMAS	CW	EA	IFAS	OD	RBC	SBR	TF
Screening	•	•	•	•		•		•	•	•
Drum screen					•		•			
Primary settling	•	•	•	•		•		•	•	•
Sub-surface aeration	•	•	•		•	•			•	
Surface aeration							•			
RBC motors								•		
Secondary settling	•	•	•		•	•	•	•		•
Volute	•	•	•		•	•	•	•	•	•
Anoxic mixing	•	•				•				
Anaerobic mixing		•				•				
SBR mixing									•	
<u>Pumping units</u>										
Influent	•	•	•	•	•	•	•	•	•	•
TF pumping										•
RAS	•	•	•		•	•	•			
Nitrate recycle	•	•				•				
P.sludge	•	•	•		•	•	•	•		•
W.sludge	•	•	•		•	•	•	•	•	•

5.3.1 Aeration energy

As stated previously, aeration is the primary energy sink in most suspended growth systems, and is one of the most complex energy sinks to model due to the number of parameters involved. Several assumptions and simplifications have been made where there are gaps in the literature, or where it has been determined that further levels of accuracy would be rendered redundant because of broader assumptions that have been made. The diffused aeration energy model development is presented in Figure 5-4. The specific details and calculations are provided in Appendix D.2. Horizontal surface aeration is unique to the oxidation ditch model; the details of which are included in the OD system model (Appendix E.4). Some of the key parameters that influence aeration energy are discussed here.

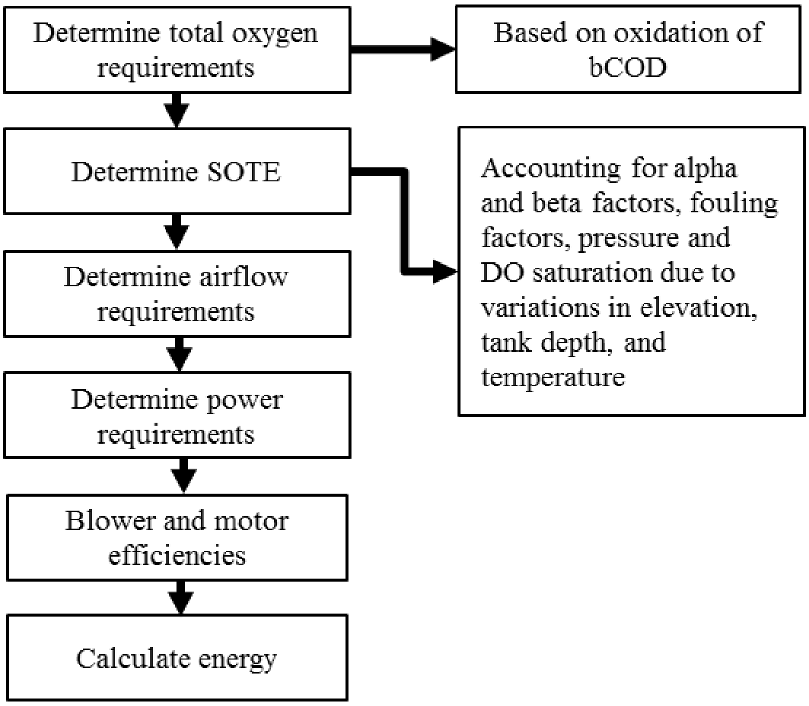

Figure 5-4: Schematic of diffused aeration energy model development

5.3.1.1 Aeration parameters

Oxygen demand is the primary aeration parameter and is determined from the bCOD (biodegradable chemical oxygen demand) oxidised per day. It is a function of influent and desired effluent substrate concentration, biomass production, and oxidised nitrogen. Values for oxygen transfer efficiency (OTE) are primarily dependent on the oxygen delivery mechanism and diffuser type. The oxygen delivery systems and diffuser types used in the study are fine bubble diffusers and mechanical surface aerators. The surface aerators are used only for racetrack type oxidation ditch. Aeration tank volumes are determined by the organic loading. Considering the plant scale range in question it is conceivable that tank volumes can be relatively small. In theory, increasing the aeration tank depth will increase oxygen transfer efficiency [212]. Therefore, it is preferable to have the tank as deep as possible. However, for practical reasons there are recommended minimum tank depth-width (3:1), or depth-diameter (1.2:1) ratios. The alpha correction factor α is the ratio of the mass transfer of oxygen in wastewater to that of clean water given by Eq. 17, where $K_L a$ is the volumetric mass transfer coefficient with units of s^{-1}.

$$\alpha = \frac{K_L a \; wastewater}{K_L a \; clean \; water} \qquad \text{(Eq. 17)}$$

The alpha factor is presented here as a function of the calculated SRT (Figure 5-5).

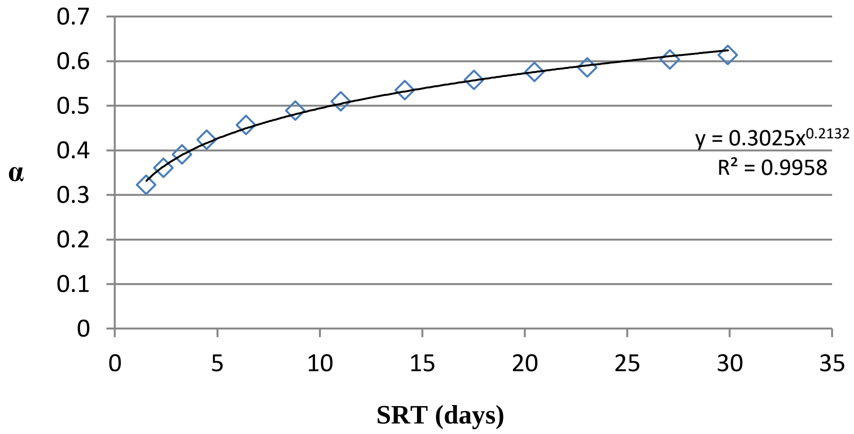

Figure 5-5: Alpha factor as a function of solid retention time

5.3.1.2 Aeration blowers

Compressed air systems are highly energy inefficient with only 10-20% of the energy reaching the end-point of use [213]. Most of the energy consumed by blower systems is converted to unusable heat with the remainder being lost through friction and noise. The choice of blower can be dependent on scale, and this can be a contributing factor to reported energy scale economies. Large scale WWTPs with airflow capacity requirement greater than 425 m^3/min generally operate multi-stage centrifugal blowers with efficiencies ranging from 60 – 70% [28]. Rotary-lobe positive-displacement blowers are often chosen for small WWTSs with airflow requirements less than 425 m^3/min [28]. These are the simplest type of blower in terms of operation and control, and also required the lowest capital investment. Throttling is not possible with these blowers and capacity change is generally achieved with variable frequency drives (VFDs). Their efficiencies range from 45-65% depending on the level of maintenance [28]. Detail of the parameters included in aeration modelling, as well as the value ranges, assumed values and sources are presented in Table 5-2.

Table 5-2: Aeration system parameters, reported value ranges and assumed values

Parameter	*Variation/range*	*Assumed values*	*Source*
Aerator system	Submerged diffuser Horizontal surface (rotary type)		
Diffuser types	Fine bubble diffusers Coarse bubble diffusers		
Oxygen transfer efficiency • Surface aerator • Fine bubble diffusers	Range (kg O_2/kWh) 1.5 – 2.1 3.0 – 4.8	 1.8 3.5	[8, 214]
Alpha factor (α) • Surface aerator • Fine bubble diffusers	 0.85 Variable	 Function of SRT (Figure 7-5)	[215]
Beta factor (β)	0.97 – 0.99	0.9	[216]
Fouling factor	0.4 - 1	0.9	[217]
Tank depth (m)	4 - 6	Variable based on tank surface area to depth ratio	
Tank shape	Rectangular, round		
Blower efficiency	0.45 – 0.65	0.60	[28]
Motor efficiency	0.85 – 0.95	0.90	[28]
Temperature (°C)	Variable	10	
Elevation (meters above sea level)	Variable	118	

5.3.2 Pumping energy

Wastewater pumping can account for up to 15% of total WWTP energy use [28]. Energy consumption values for pumping can vary depending on a number of factors such as sludge characteristics, pump and motor efficiencies, plant size, age, design and layout, topography, and type of secondary treatment. Certain pumping functions such as influent pumping will have similar energy use values across all systems in a comparative analysis, and there is an argument that system boundaries should be adjusted to exclude them. However, their inclusion allows for the compilation of a complete energy distribution profile in the case of a stand-alone system audit.

The type of secondary treatment in particular will dictate to a large degree, the percentage of energy consumption attributed to pumping. Pumping is the primary energy sink in TF systems. The TF process requires a minimum amount of wetting in order to maintain microbial population and avoid insect and odour problems on the surface of the growth media. This means that the process must be continuous, and even with minimum wetting rates the dynamic head required to maintain distributor arm motion in hydrostatic systems can be significant. In CAS plants, RAS pumping energy can account for 1% of total plant energy consumed [28], which equates to 15% of total pumping energy. Extended aeration systems produce less WAS than the CAS systems with shorter solid retention times, which reduces both pumping in the WAS lines. However, these values are generally low (~ 0.3 % of total pumping energy) when compared with other unit process pumps in conventional systems.

The size of a WWTP can be linked to pump efficiency. Firstly, there are energy economies of scale to be achieved with increased flowrate as frictional headloss decreases with increases in pipe diameters. Secondly, small wastewater treatment systems can experience much greater variations in flowrate compared with larger systems (Figure 5-6) [28]. The magnitude of these variations is amplified during storm events. Maximum efficiency on the pump performance curve falls within a narrow band on the flowrate axis. When flowrate experiences large fluctuations the pump spends more time away from its maximum efficiency value. Variable frequency drives can act to counter this effect, but from personal communication with WWTP operators, management and other professionals in the field, the uptake of this practice is often overlooked due to capital restraints. Maintaining high wet-well levels is a control strategy that can be used to maximise pump efficiency at a plant. However, this process can lower fluid velocities and result in unwanted solids deposition, and can also reduce the reserve capacity of the system.

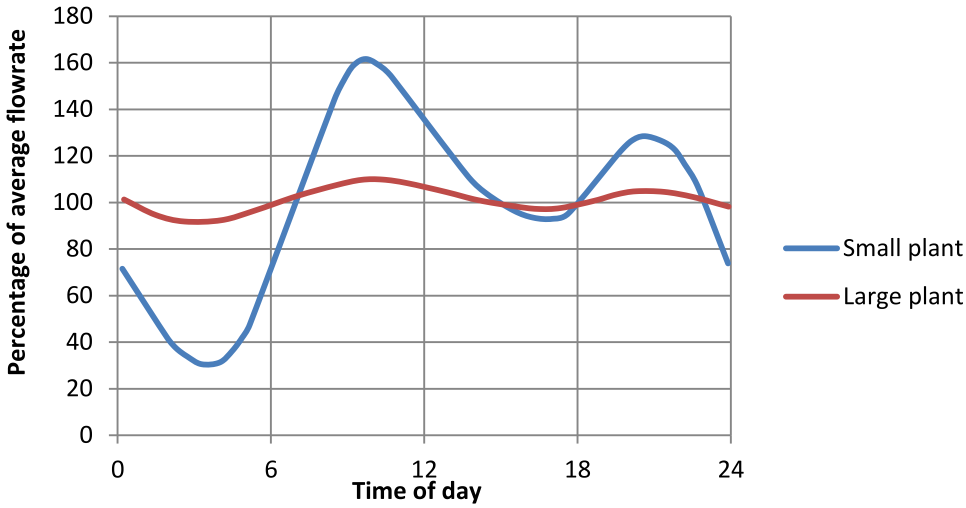

Figure 5-6: Average percentage variations of normal flowrate during 24 hour cycle for large (> 400,000 m3/day) and small (4,000 to 40,000 m3/day) plants. Adapted from [3]

5.3.3 Pumping models

Details of the pumping units, parameters, assumed values and sources are presented in Table 5-3. An overview of the rationale behind the assumptions and formulations of these values presented are provided in Appendix D.3. Foladori et al. [124] conducted a detailed energy audit of a several small scale wastewater treatment plants in Italy. The results of the study are used for validation of pumping models developed here.

Table 5-3: Pumping model parameters and assumed values

Variable	**Influent**	**Primary sludge**	**WAS**	**RAS**	**Nitrate recycle**	**Trickling Filter**	Source
ΔH (m)	3	7	7	3	0	Variable	
L_{pipe} (m)	8	Variable	Variable	Variable	Variable	Variable	
D_{pipe} (m)	0.1 – 0.15	0.1 – 0.15	0.1 – 0.15	0.1 – 0.15	0.1 – 0.15	0.1 – 0.15	[218]
Minimum Fluid velocity, v, (m/s)	1.83	1.83	1.83	1.83	1.83	1.83	[219]
Fluid density, ρ, (kg/m^3)	1010	1030	1010	1010	1010	1010	
Solids concentration (%)	0.1	4.3	1.3	0.8	0.35	0.8	[114]
Viscosity [μ] of water (Ns/m^2)	1.25 x 10^{-3}						
Sum of the minor headloss coefficients (Σk)	12.5	9.6	9.6	8	8	12.5	[122] [218]
Motor efficiency η_m	0.8	0.8	0.8	0.8	0.8	0.8	[218]
Pump efficiency η_p	0.55	0.55	0.55	0.55	0.55	0.55	[28]
Mulbarger friction factor, m_f	N/A	1.75	N/A	N/A	N/A	N/A	[218]

Flow variations of 100 – 1,000 m^3/d were input into the influent pumping model. The pumping h/day were adjusted to maintain minimum velocities of 1.83 m/s as per the recommendations of Poloski et al. [219]. Upon reaching a 24 hour/day pumping regime for a given flow rate the pipe diameter was increased from 0.1 – 0.15 m to maintain minimum velocity. This resulted in a constant influent pumping energy value of 0.042 kWh/m^3 based on the assumed parameter values presented in Table 7-3. The influent pumping energy values reported by Foladori et al.

[124] ranged from 0.032 to 0.076 kWh/m^3, with an average of 0.54 kWh/m^3. The most significant parameter in the influent pumping model was found to be the static head. Model sensitivity analysis was conducted to assess the effect of variations in static head (Figure 5-7). The average value reported by Foladori et al. [124] coincides with the model static head height of 8 m. However, it was felt that this height was excessive considering the scale range in question and therefore, the model value remains at 6 m.

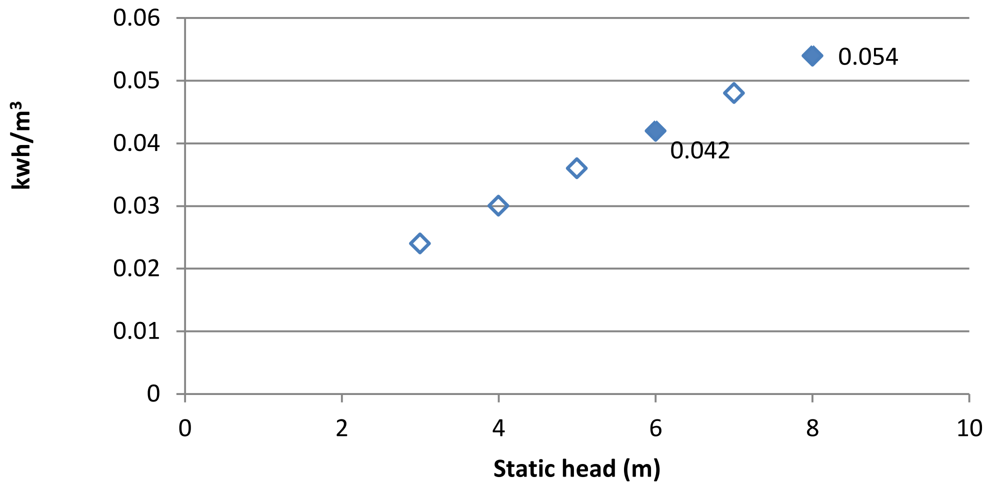

Figure 5-7: Influent pumping energy as a function static head height

The combined primary and secondary sludge pumping energy varied from 0.0013 – 0.0017 kWh/m^3 for influent flowrates of 100 m^3/d and 1,000 m^3/d respectively. These values are low when compared with the values reported by Foladori et al. [124] that ranged from 0.002 – 0.017 kWh/m^3, with an average value of 0.009 kWh/m^3. However, the design capacities of the plants in the study were larger (1,050 – 20,000 PE). When the model pipe lengths were adjusted to reflect similar design capacities, values of 0.0104 kWh/m^3 were observed. It should be noted that the piping configuration included in the model assumes the optimum layout to achieve minimal minor headlosses. The RAS line model energy values ranged 0.038 to 0.044 kWh/m^3, based on a MLSS concentration of 3,500 mg/l and a return concentration of 8,000 mg/l. An average value of 0.014 kWh/m^3 was reported by Foladori; however, it is unclear if this value was based on the influent or RAS flowrate. Nonetheless, the RAS model values are very high and comparable with the influent pumping values that have significantly higher static head. However, the RAS line energy recorded at Plant D and E in the preliminary study ranged from 0.002 – 0.041 kWh/m^3. It is possible that the velocities of the RAS lines at the Italian plants may not have been maintained at the recommended minimum velocities reported by Poloski et al. [219]. Very low flowrates at small plants require much reduced pumping times that could have an adverse effect on MLSS concentrations, and, therefore, pumping velocities may be reduced at the risk of solids deposition. Model nitrate recycle energy values ranged from 0.032 – 0.033 kWh/m^3 (nitrate recycle flowrate). An average TF pumping model energy value of 0.0905 kWh/m^3 (trickling filter pumping flowrate) was observed. Values of additional headloss in TF distribution arms are significant and reported to range from 0.6 to 1.5 m [40]. Sensitivity analysis was conducted to assess the effect of variations in distributor arm head loss (Figure 5-8). The variations in distributor arm headloss from 0.6 -1.5 m resulted in a 4.5% increase in total pumping energy. Assuming the medium value of 1.05 m yields an error of ± 2.5% of total pumping energy. There were limited TF pumping data available for comparison. Values reported in Metcalf and Eddy [28] ranged from 0.061 – 0.096 kWh/m^3, however, the plant scale range that these values are taken from is unclear.

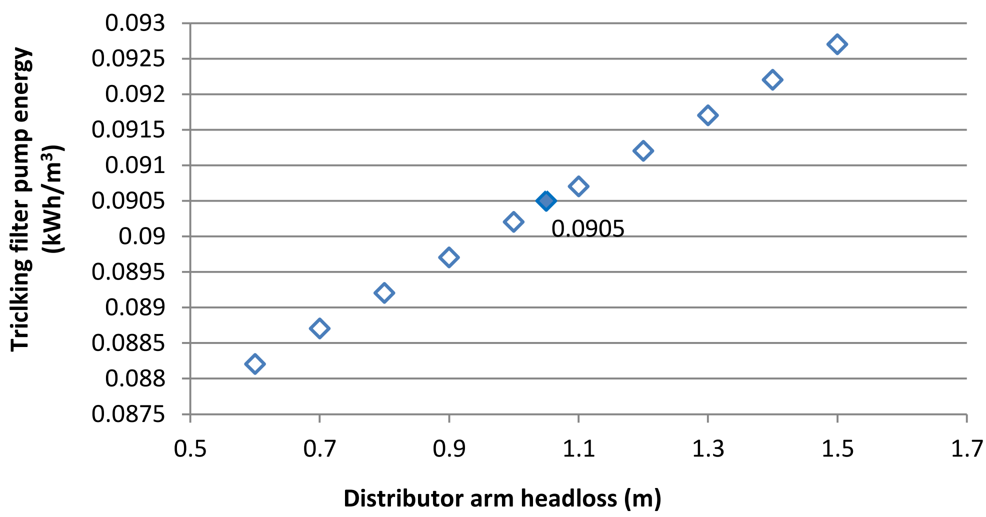

Figure 5-8: Trickling filter pumping energy with variations in distributor arm headloss

5.3.3.1 Drum screen

The energy consumption of the rotary drum fine screen used in the EA and OD systems varies depending on flowrate. The motor power of the smallest model reported by [220] is 0.244 kW for a maximum capacity of 502 m^3/d; above this flowrate, the power increases to 0.56 kW for a capacity of 1794 m^3/d. With sufficient wet-well capacity and control of inlet flow, the energy demand can be maintained between 0.01- 0.03 kWh/m^3.

5.3.3.2 Mixing

It is assumed that mechanical mixing is required for systems that employ anoxic or anaerobic zones. It is assumed that all mixing is carried out by mechanical means. Sludge thickening mixing energy is included in the average values used for the individual process units. Power values for anoxic and anaerobic zone mixing are calculated as a function of liquid volume (5 $kW/10^3\ m^3$) [28].

5.3.3.3 RBC energy

RBC system energy requirements are dominated by the power required for shaft rotation. Shaft-rotation energy demand is a calculated as a function of the required disc surface area. A linear regression model was developed based on the study carried out by Gilbert et al. [221], and is given by Eq.18.

$$\mathrm{E}_{RBC} = (184.382 \times 10^{-6})A_{disc,req.} \qquad \text{(Eq. 18)}$$

Where;

E_{RBC} = specific energy required (kWh/m^2)

$A_{disc,req.}$ = disc area required (m^2)

5.4 Additional energy sinks

Many of the conventional treatment systems have common unit processes (inlet works, primary sedimentation, sludge dewatering, etc.). The plant scale range in question is sufficiently small that anticipated economies of scale in terms of energy use of many of these unit processes are considered negligible. Hence, the energy values provided are a function of flowrate only. Details of these processes are presented below (Table 5-4).

Table 5-4: Energy use assumptions for common unit processes

Unit process	*Value*	*Details*	*References*
Mechanical inlet screens (kWh/m^3)	0.01	Continuous belt type	[124]
Primary sedimentation tanks (kWh/m^3)	0.012	Circular tank	[124]
Secondary sedimentation tanks (kWh/m^3)	0.012	Circular tank	[124]
Thickening and dewatering (kWh/kg DS)	0.05	Volute	[222]
Municipal energy (kWh/m^3)	0.012	Plant lighting, control and automation, administration buildings	[124]

5.4.1 Total energy use

Model validation for activated sludge system energy use of was carried out with energy data collected during energy auditing in the preliminary LCA study. The system type, discharge limits, and design loads were matched accordingly. Good correlations were observed for agglomeration values greater than 2,000 PE (Figure 5-9). A significant increase in error between model and empirical values was observed for plants below 2,000 PE, which ranged from 3% at 2,000 PE to 25% at 500 PE (Figure 5-10). This indicates that the models are not reproducing the negative scale economies observed with the empirical energy values. The steady state assumption made for the models does not capture the energy losses that occur due to the variation in flow rates, which can be significantly larger for small systems. The model does not assume that VFDs are employed to mitigate the effect of variation in flowrate, and so higher values could have been expected, particularly with the low pump and motor efficiency values that were used. In reality, VFDs may be overlooked because of CAPEX restraints, with operators prepared to accept some given level of energy loss to reallocate capital for issues that are considered to be of greater priority.

Unit process start-up and shut-down energy losses were assumed to be negligible for small plants, but this assumption may require further investigation. Small unmanned systems may lack adequate monitoring and control, and as a result may be operated at elevated DO levels as an additional safety precaution to avoid discharge limit breaches. The plants that have been used to validate energy estimations are old systems nearing the end of their lifetime and may suffer from overloading and inefficient plant design and configuration. The accuracy of the DST energy models for systems below 2,000 PE needs to be determined with a) more modern state of the art systems, and b) a much greater sample size of systems to compare against. Reliable data were not available to carry out validation of attached growth total system energy use. However, the only energy sink unique to the RBC system is the disc motor energy which is based on empirical data, and, therefore, deemed to be an appropriate representation of actual RBC energy demand. Similarly, primary effluent pumping and distribution over growth media is the only energy sink unique to the TF system, and the estimated values are considered to be within an acceptable range. Natural systems energy use was limited to influent rising.

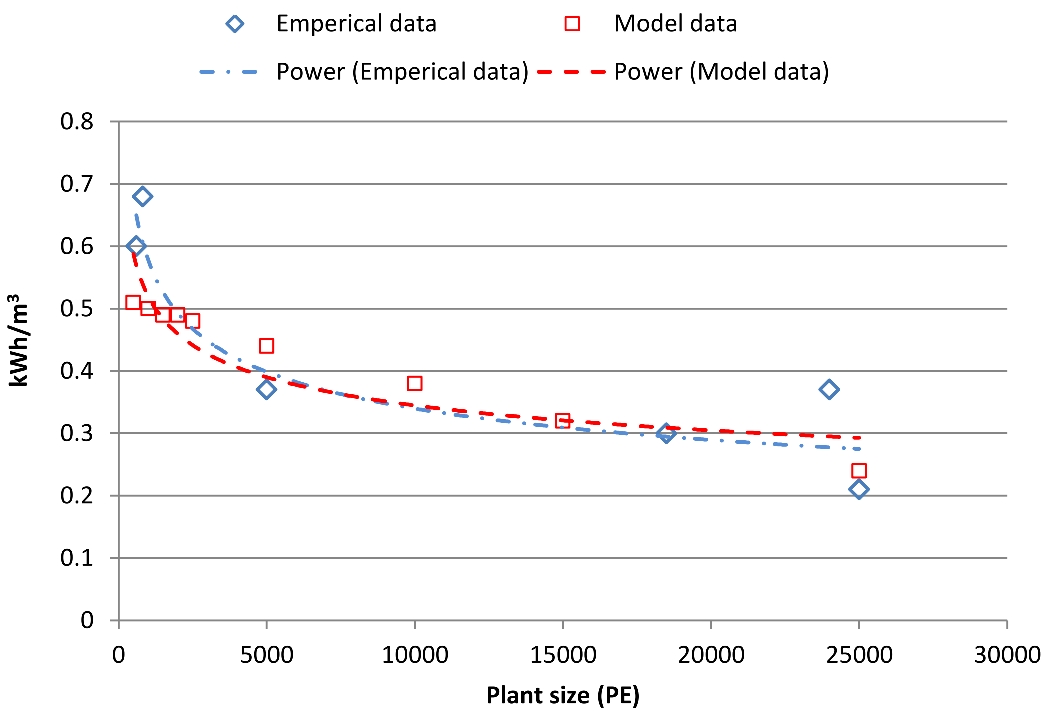

Figure 5-9: Energy use as a function of plant scale (range 500 – 25,000 PE)

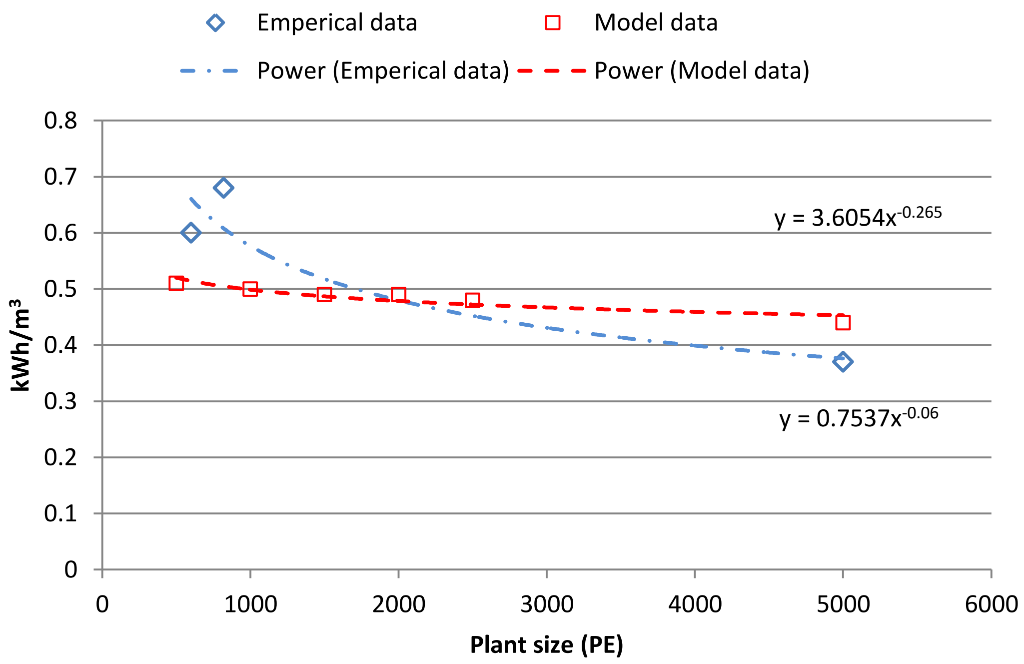

Figure 5-10: Energy use as a function of plant scale (500 – 5,000 PE)

5.5 Conclusions

Wastewater treatment energy consumption estimations are generally based on empirical data collected from existing systems. These data are often presented in the literature in terms of average energy consumption values of a selected cohort of plants without consideration or qualification of variation in site-specific conditions. This can lead to a misrepresentation of the actual energy efficiency of a given system. One solution to this problem is to gather data from plants with variations in system type, scale, loading, and discharge limits. The method of sludge disposal must also be considered because of the difference in energy use between natural and mechanical sludge treatment systems. However, even a small number of discretions in each of these parameters, would require auditing a very substantial cohort of plants, which may not be practical or even achievable. To overcome this problem, and the approach adopted here is to calculate energy consumption based on first principle modelling.

The energy sinks in each of the WWTSs were identified and modelled to allow for site-specific variability. Energy consumption estimations are based on defined scale, loading, discharge limits, and method of sludge disposal. Aeration and pumping models are for many systems the primary energy sinks and have the largest degree of parameterisation. Other energy sinks common to many of the systems have been modelled with less complexity and in some cases their values are a function of a single variable such as scale, flowrate, or organic loading. Good correlation was observed during model validation for suspended growth systems over a large scale range; however from 500 to 2,000 PE there was a significant increase in error. Very little data were available for validation of the attached growth or hybrid systems; however, the additional energy sinks for these systems are limited to TF pumping and RBC motor power, the latter of which has been compared with energy values sourced from personal communication with the Irish water utility and found to be within ± 5%.

The energy models do not capture all of the energy inefficiencies that can occur within a system, and it is debateable whether they should. It could be argued that in order to attain realistic estimations of operational cost, energy consumption values should be representative of empirical energy data because low energy values will produce higher percentage contributions from the other operational cost elements. However, it is questionable whether there is any benefit in reproducing energy consumption values of an inefficient system. The position adopted here is that the estimated energy values represent the best case scenario for each system and provide an acceptable basis for comparison and compilation of overall energy cost.

6 Decision Support Tool

6.1 Methodology

The decision support tool (DST) was developed on the Microsoft Excel 2010 platform with *Visual Basic for Applications* (VBA) coding. The program is intended to support the WWTS selection process by providing economic and environmental system-specific information for a range of user-defined, site-specific scenarios. The program has been designed for both technical and non-technical users. Default values for loading, discharge limits and specific costs are provided for the non-technical user. For technical users these parameters have been soft coded into the system for site-specific variation. In addition, aeration parameters such as oxygen transfer efficiencies, beta values, and diffuser fouling coefficients have also been soft coded. The program has been designed for a plant scale range of 500 – 2,000 PE, and while it will accept data for large systems, assumptions and simplifications that have been made for small scale systems may not be applicable. For example, the program assumes a single primary and secondary settling tank, which would not be practical for large scale systems that would normally employ multiple settling tanks. Power requirements for unit processes such as inlet works and dewatering are based on single units that can respectively accommodate influent flow and sludge production for the defined scale range. The DST program overview is presented below (Figure 8-1).

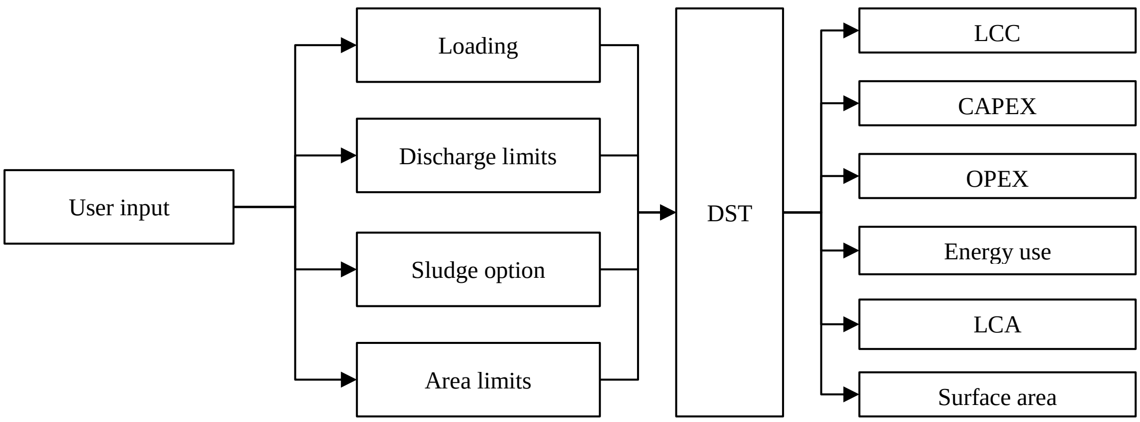

Figure 6-1: Decision support tool program overview

6.2 User input

The program receives several site-specific user-input data: loading, discharge limits, sludge option and area limits (Figure 8-2). Average influent organic and inorganic loading values are provided for situations where site-specific loading values are unknown. Hydraulic loading can be defined in terms of estimated hydraulic load or by agglomeration size. The default relationship between hydraulic load and agglomeration size is 1 PE = 200 litres of influent wastewater. This relationship is soft coded for user definition. The DST discharge limits included in the program are based on a survey of limits found in Ireland. There are three sludge treatment options included in the program: 1) no treatment 2) volute sludge thickening and dewatering, and 3) drying beds. There are also three sludge disposal options included: 1) land spreading 2) transport to a larger parent plant, and 3) disposal by an external contractor. The costs associated with each method of sludge disposal can be user defined, but for convenience default values sourced from personal communication have been provided. A surface area restriction input has been included in the support tool. The program estimates the area associated with each system and eliminates those from the analyses that exceed the user-defined area. The original motivation for the inclusion of a surface area restriction was that in many cases the CW system was the optimum choice, but the large CW surface area requirements meant that their implementation may not be always be feasible. Finally, a filter option has been included that allows the user to define an output of interest. The filter menu includes: LCC, CAPEX, OPEX, energy and footprint. Upon selection of the output of interest the program sorts the systems in order of magnitude i.e. the system with the lowest output for a given filter is presented first, and then the second lowest, and so forth.

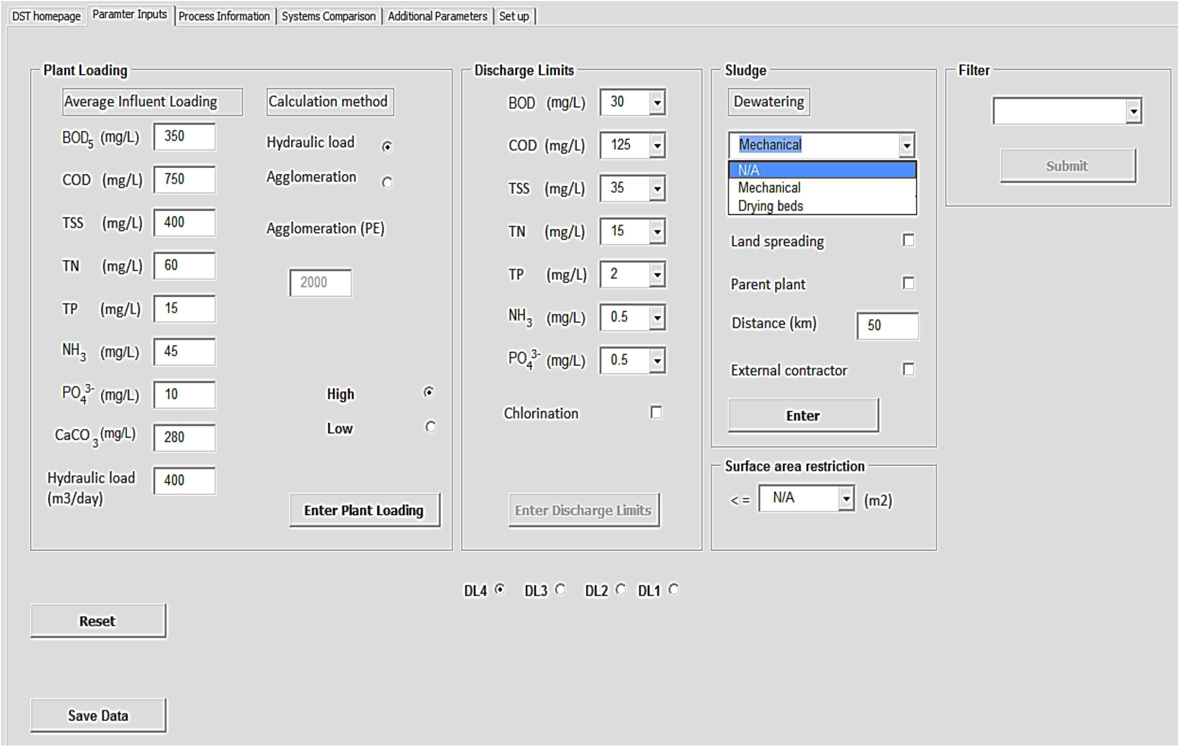

Figure 6-2: Parameter input user interface

6.2.1 Process information output

Information regarding the economic, environmental and energetic performance of each system is presented on the *Process Information* page (Figure 8-3). Energy efficiency is presented in terms of treated wastewater, BOD, TSS, NH_3, and PO_4^{3}. Energy distribution is presented as the percentage that each energy sink contributes to total energy consumption. Cost information includes: CAPEX total, CAPEX per capita, OPEX per PE-year, OPEX per volume of treated wastewater, and net present value. Operational cost distribution is presented in terms of the percentage of energy, labour, sludge disposal and chemicals. Chemical cost distribution is also provided. The system's environmental profile is presented giving the percentage that each of the considered input and output flows contributes to each of the impact categories.

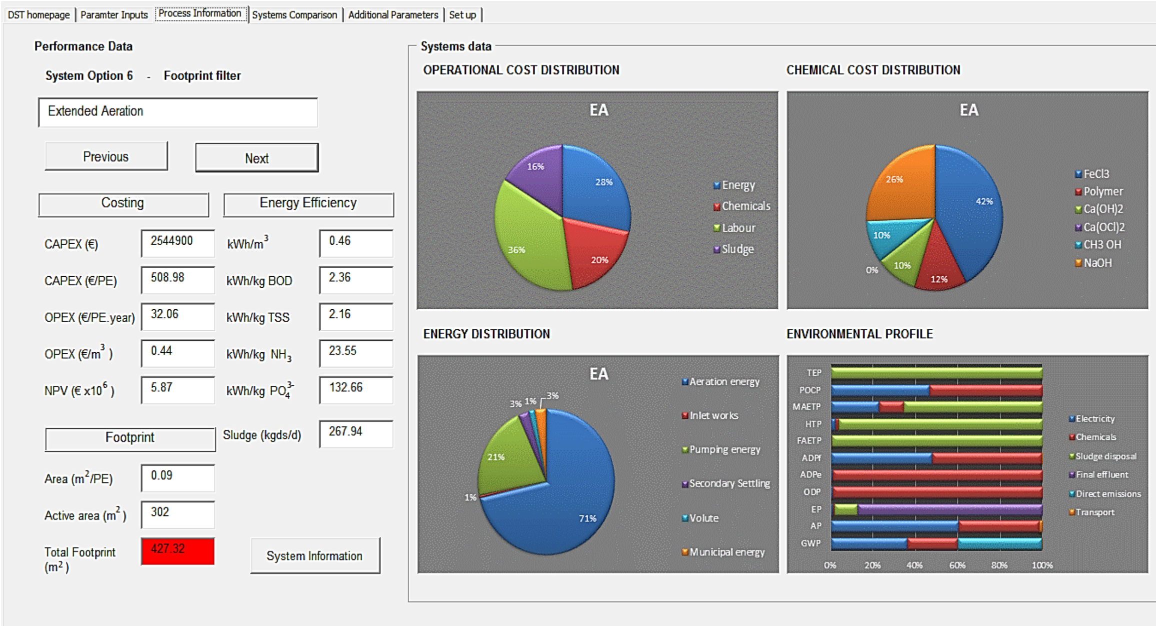

Figure 6-3: Decision support tool process information page

6.2.2 System comparison

To facilitate system comparison a *Systems Comparison* page has been included that presents energy, cost, surface area, and environmental life cycle data for a limited selection of impact categories (Figure 8-4).

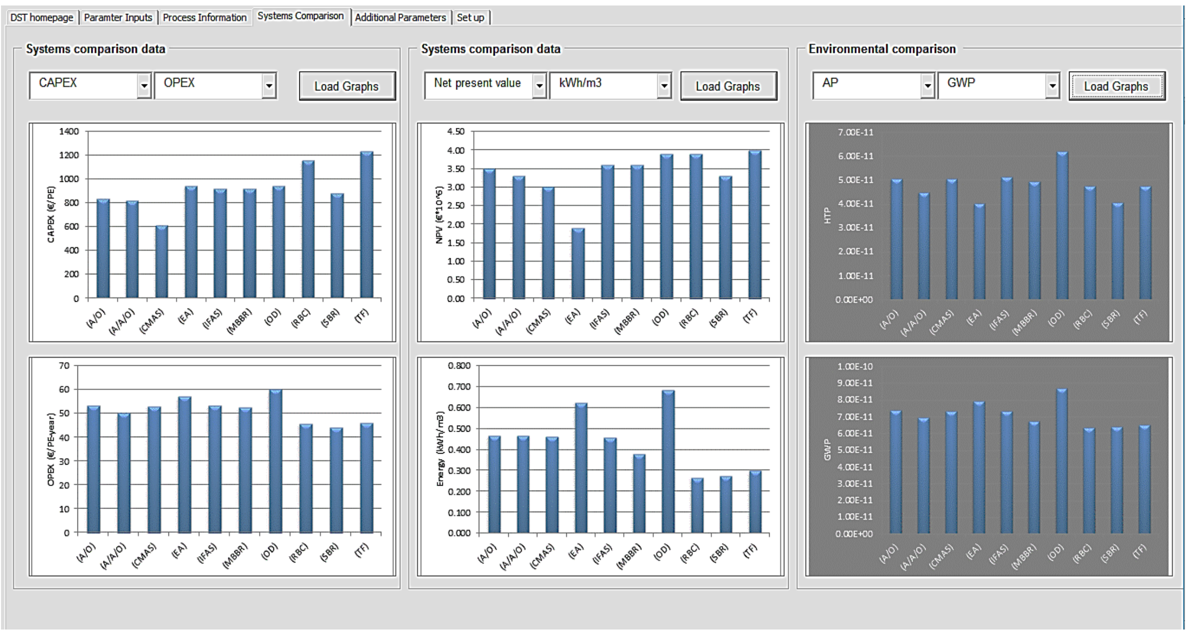

Figure 6-4: Decision support tool systems comparison

6.2.3 Additional parameters

To allow for regional specific cost variation an *Additional Parameters* page includes all of the specific operational cost information for electricity, labour, chemicals, and sludge disposal that is soft coded into the program (Figure 8-5). The LCC model lifetime and discount rates for OPEX and energy are included for user definition, as well as the specific cost of land and the value for the offset buffer (see Appendix C.1 for offset buffer details). Also included on this page are several aeration related parameters that have been assumed for the suspended growth models.

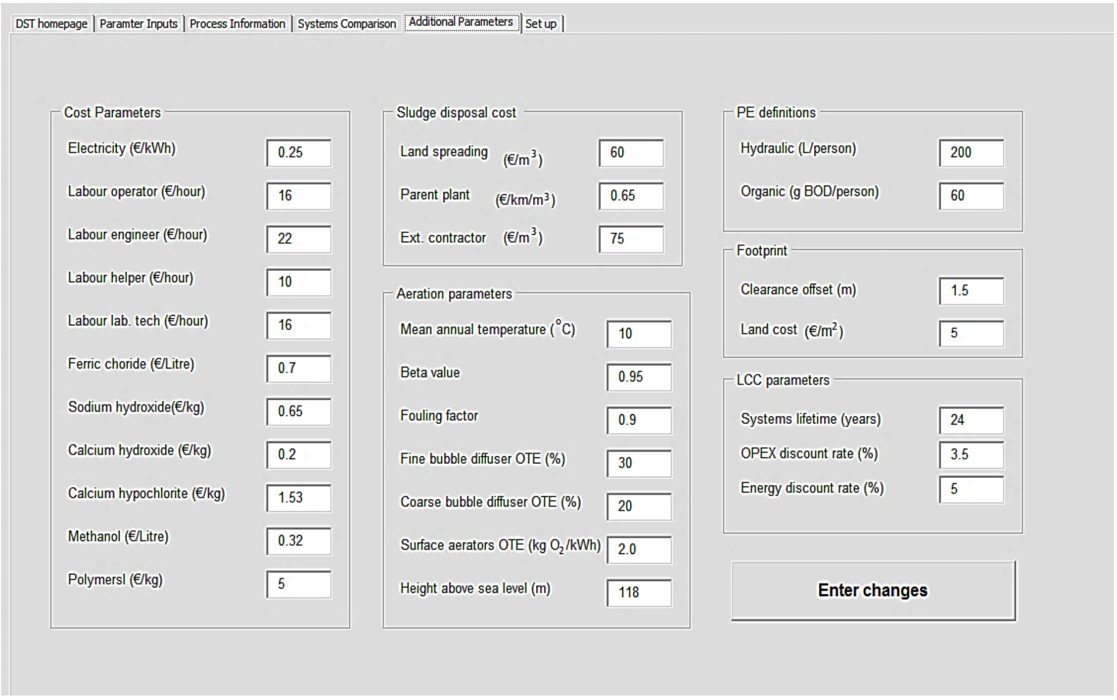

Figure 6-5: Decision support tool additional parameters page

6.3 Program architecture

6.3.1 Introduction

Details of the calculation methods for all systems are provided in Appendices E.1 to E10. The following sections provide a general overview of the program architecture.

The program is divided into two domains, the first of which, handles the quantity calculations that are carried out on the individual spreadsheets for each system. The second domain is the VBA code where the user interface is managed. There are some other functions within the VBA domain that are used to carry out spreadsheet calculations where iterations are required or there are multiple levels of conditions involved. The calculation methods used to determine quantities for BOD removal only, and BOD with nitrification were different enough in some cases to justify creating separate models for some systems. Therefore, the first stage of the calculations involves determining the governing substrate so as to select the appropriate model (Figure 8-6).

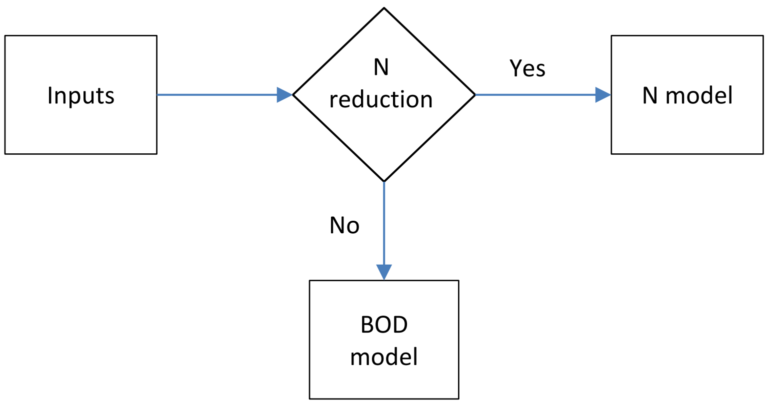

Figure 6-6: Model selection

6.3.2 Final effluent control

For BOD removal only, effluent BOD concentrations in suspended growth systems are controlled by the solid retention times (Figure 8-7). For nitrification, the SRT is determined by the AOB substrate utilisation rate (μ_{AOB}) (Figure 8-8). In attached growth systems, final effluent BOD and ammonia concentrations are controlled by the organic loading rate (Figure 8-9).

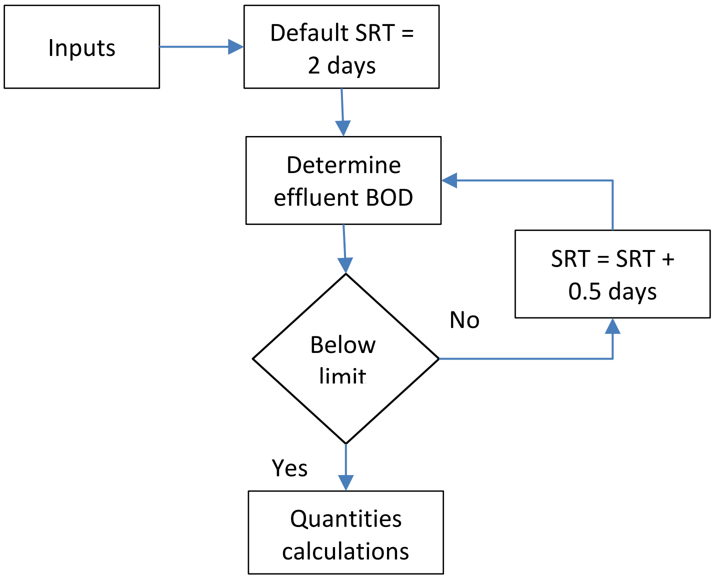

Figure 6-7: SRT determination for BOD removal only in suspended growth systems

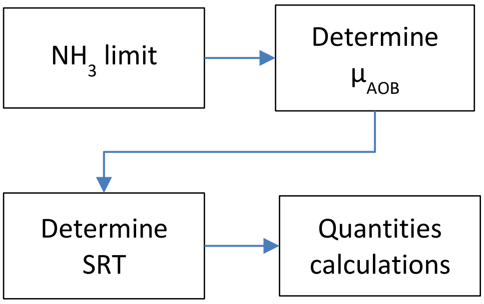

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Figure 6-8: Nitrification control sequence for suspended growth systems

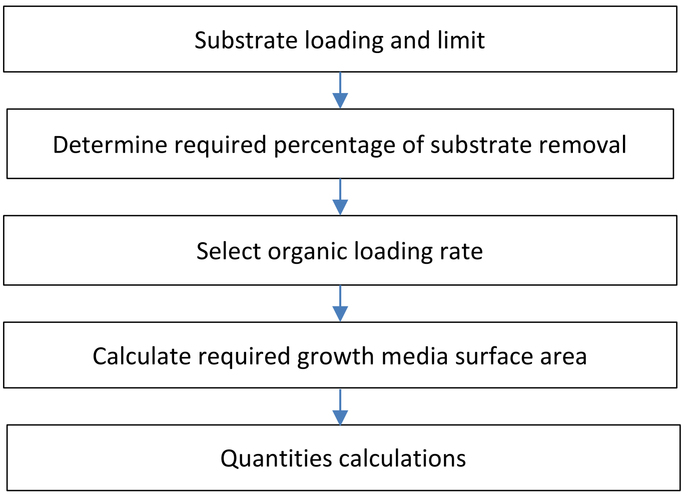


Figure 6-9: Attached growth final effluent control

Pre-anoxic denitrification in suspended growth systems is controlled by the rbCOD/bCOD ratio and the anoxic HRT (Figure 8-11). The CMAS, TF, and RBC systems achieve denitrification in a post-anoxic tank with ethanol addition, the algorithm for which is presented in the chemicals section of this chapter (Figure 8-17). The SBR denitrification is a pre-anoxic process that is controlled by the fill time and fill volume fraction (Figure 8-12). The EA and OD systems achieve denitrification through a cyclical aeration process as described in Appendix E.11.

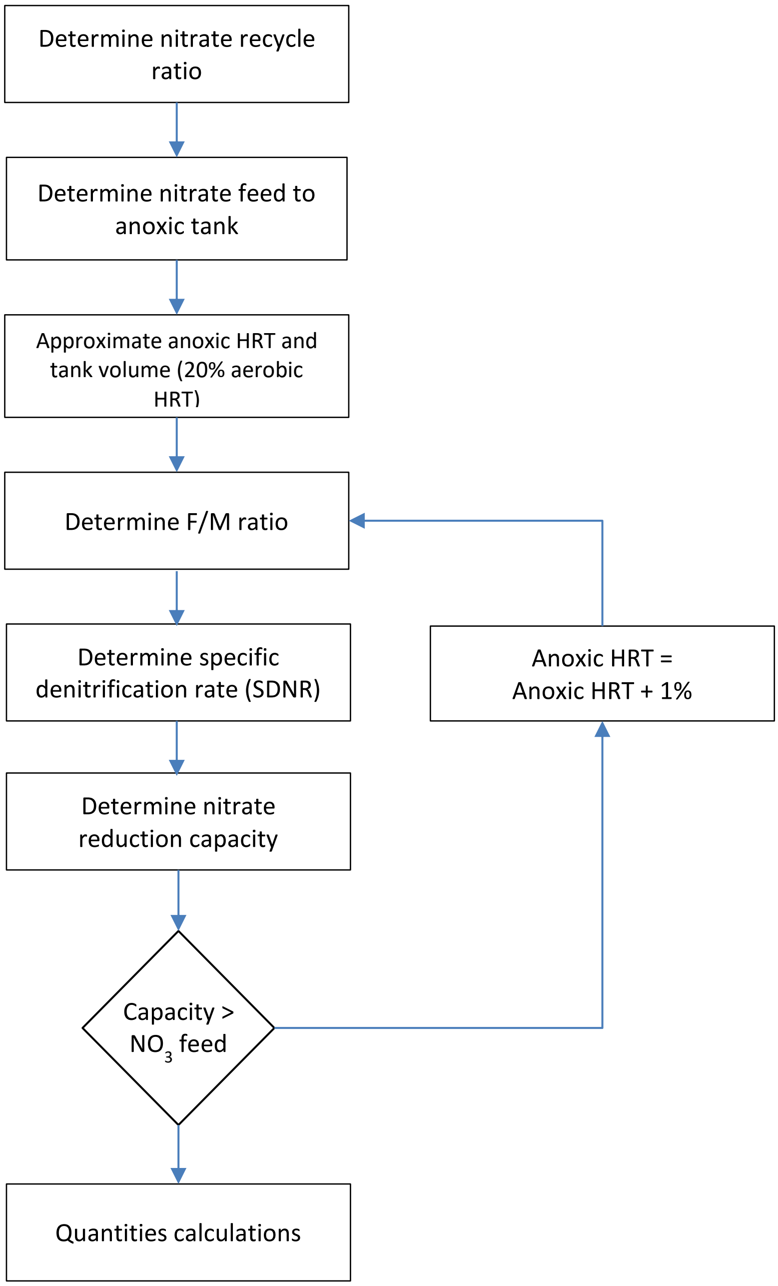


Figure 6-10: Pre - anoxic denitrification logic

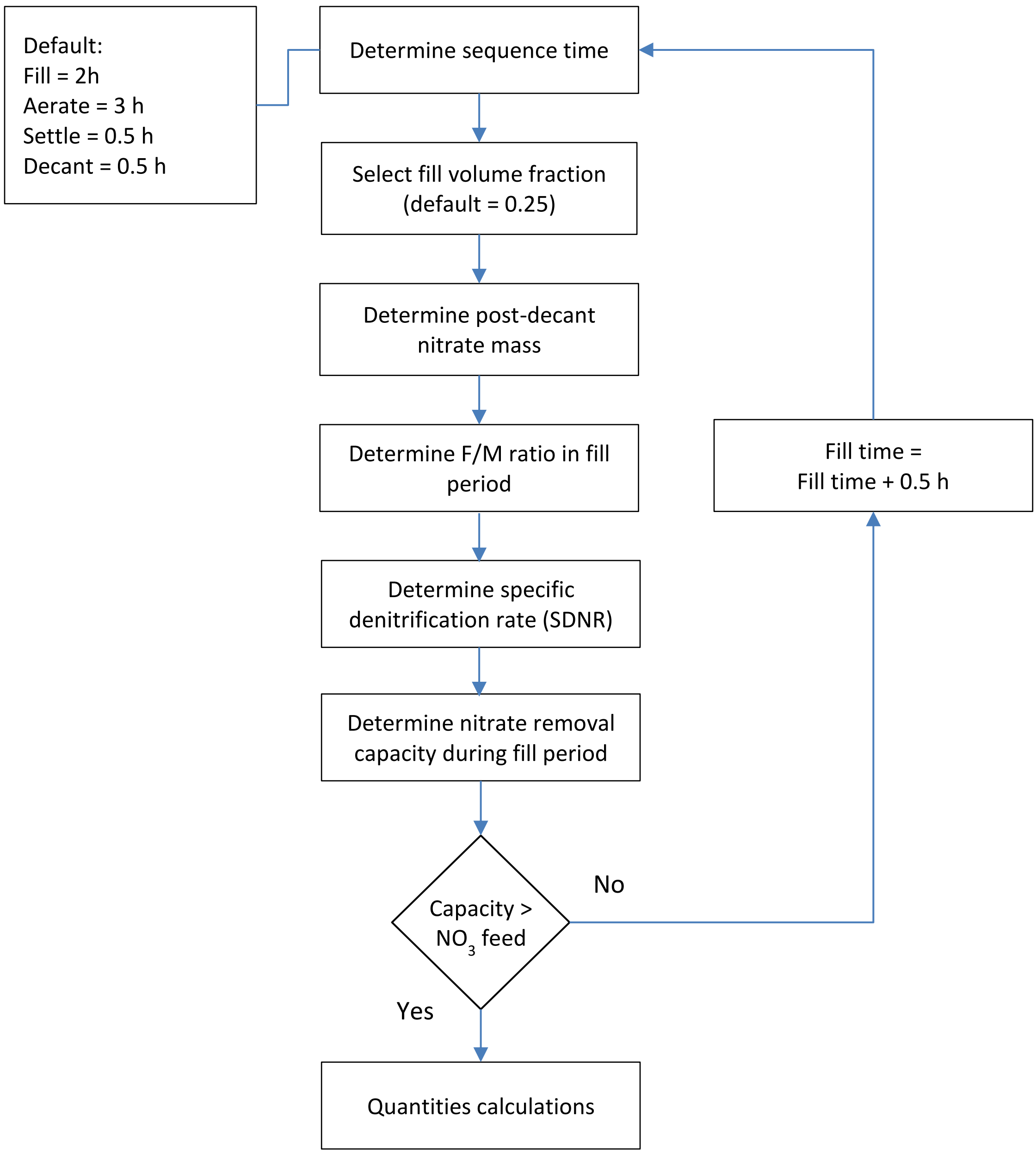


Figure 6-12: Sequence batch reactor pre-anoxic denitrification

6.4 Quantities calculations

The three quantities that are of primary interest to both the economic and environmental costs are: sludge, energy, and chemicals. An overview of the process flow and logic for the quantities calculations are presented here. As stated, details of calculation methods are provided in Appendices E.1 – E.10. The energy quantity determination flow diagram includes all system types and indicates the primary variables used in the calculations. Labour hours calculations are relevant to the LCC only. The logic for compiling the amount of hours begins with the base hours specific to a given system calculated as a function of agglomeration size. This is followed checklist of system requirements (Figure 8-13).

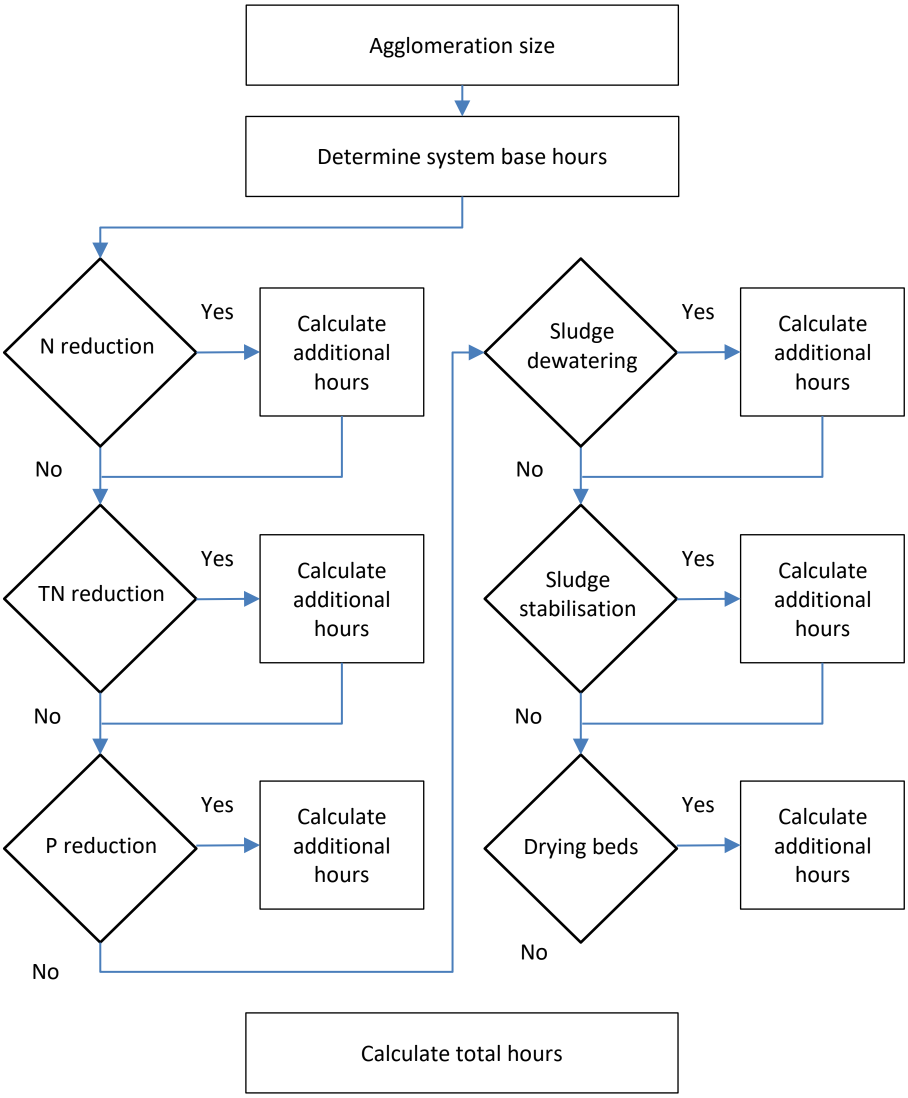
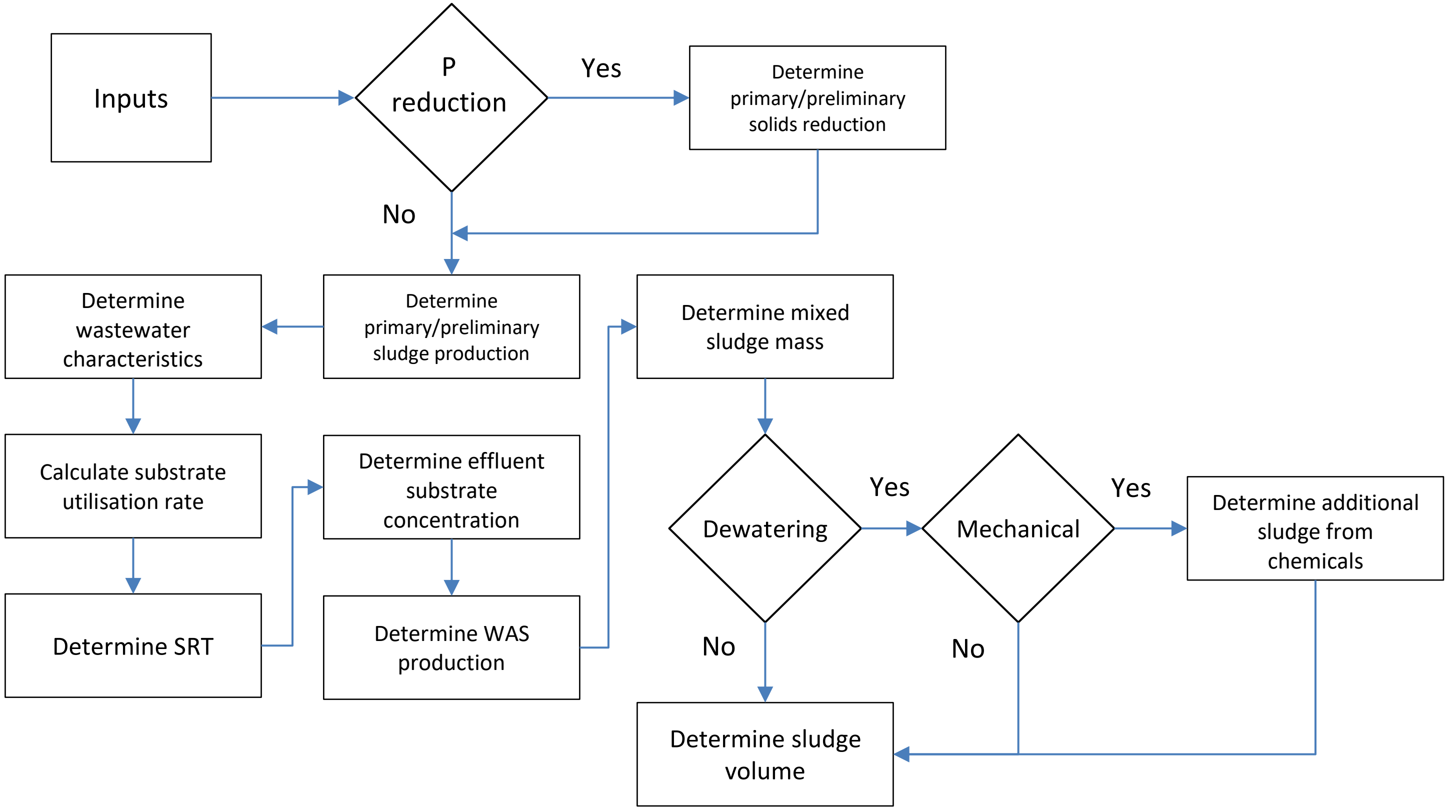
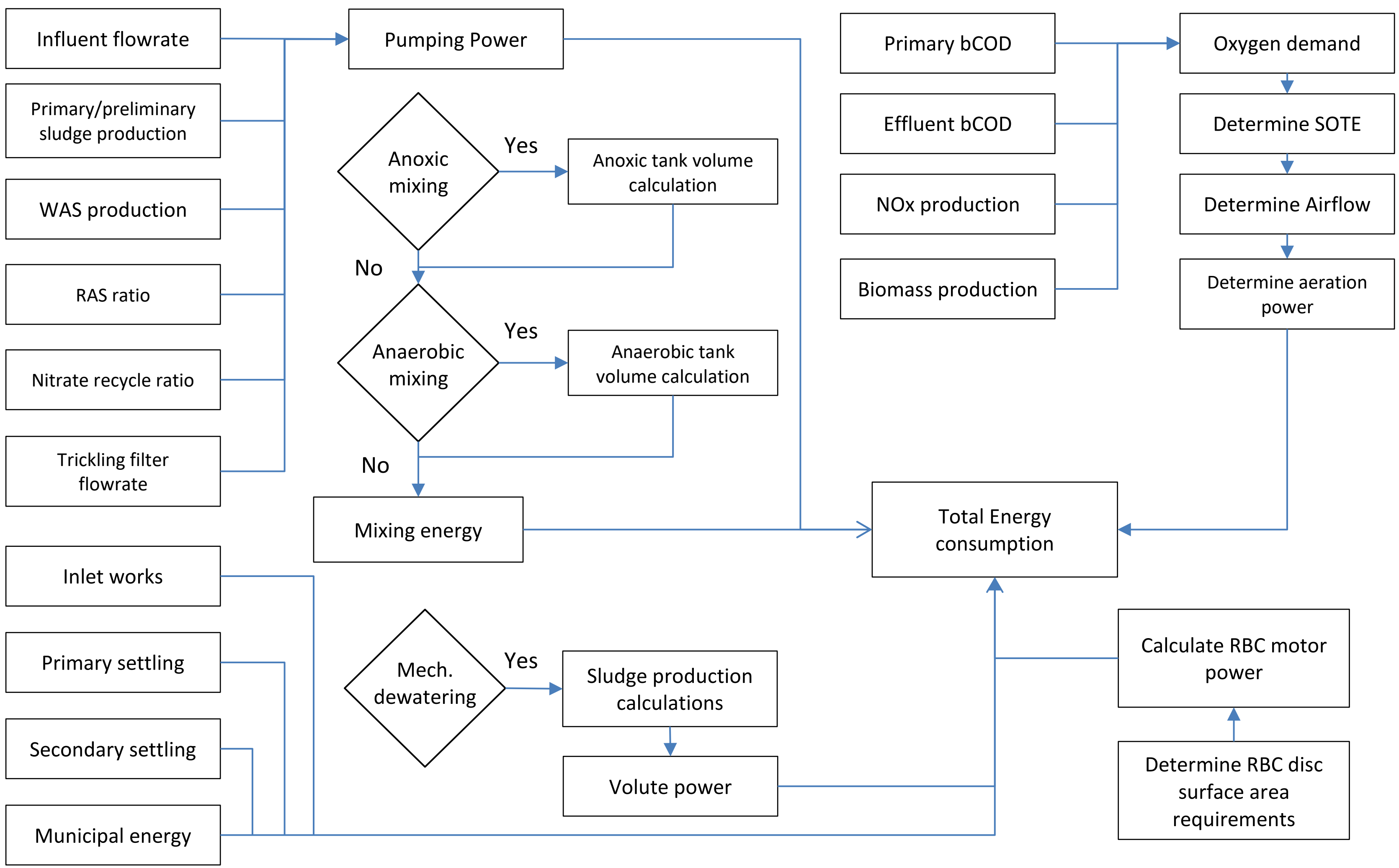


Figure 6-13: Labour-hours compilation logic

6.4.1 Sludge production



6.4.2 Energy consumption



6.4.3 Chemical use

6.4.4 Sodium Hydroxide

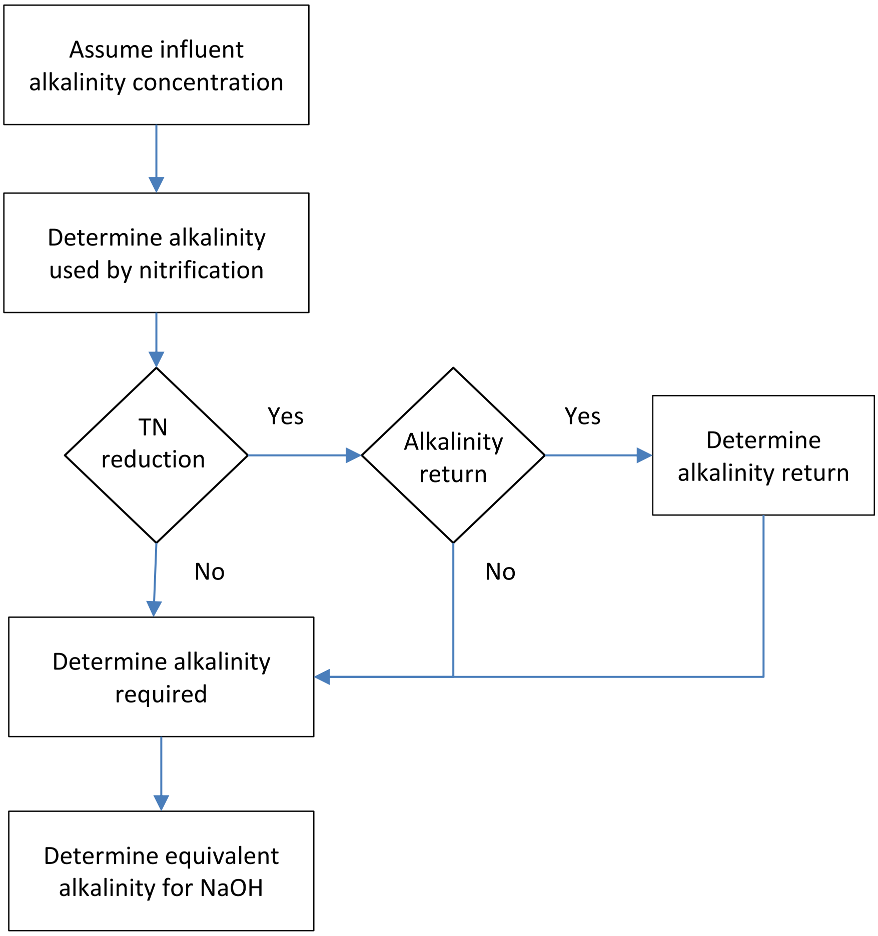


Figure 6-14: Calcium hydroxide determination

6.4.5 Ferric chloride

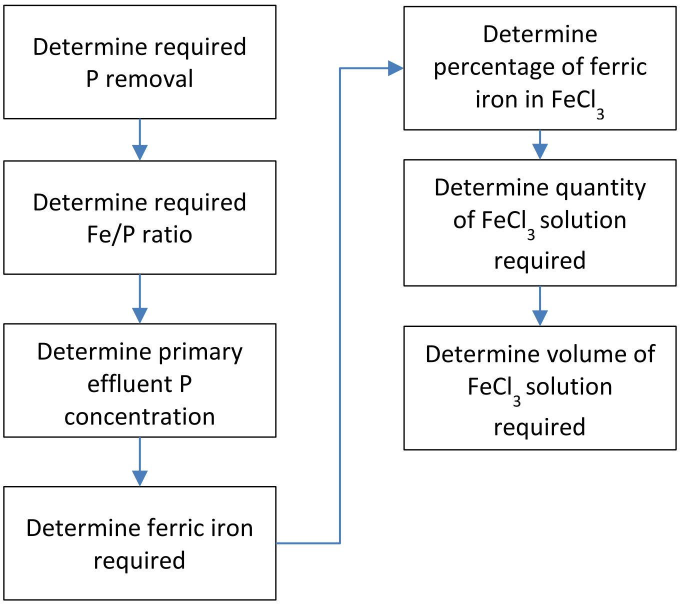


Figure 6-15: Ferric chloride quantity determination

6.4.5.1 Polymer and lime dosage

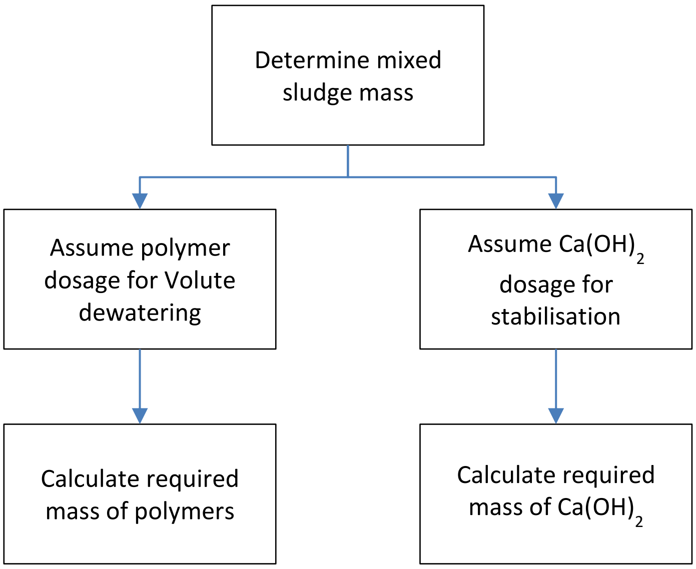


Figure 6-16: Sludge chemical quantity determination

6.4.5.2 Ethanol dosage (Post-anoxic denitrification)

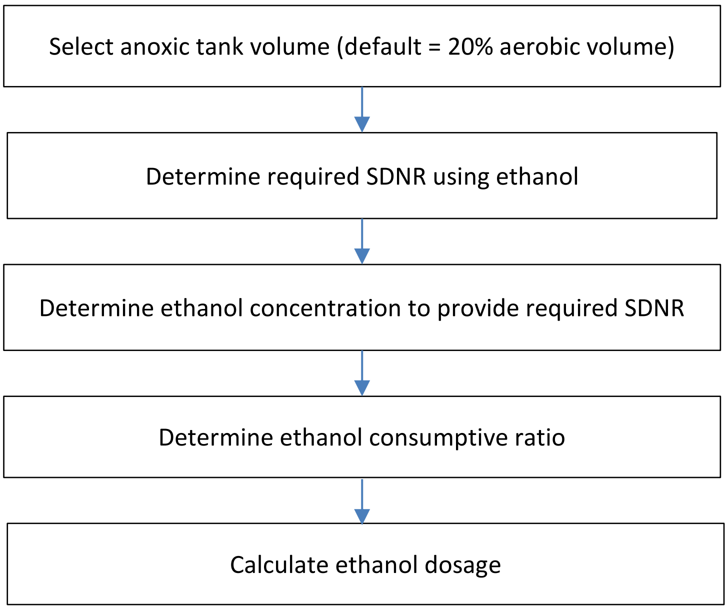


Figure 6-17: Ethanol quantity determination for post-anoxic denitrification

6.5 Conclusion

The DST provides an integrated framework to assess and compare small WWTS energy use, economic cost, and environmental impact. Life cycle cost analyses and environmental assessments can be time consuming, data intensive and expensive processes. The value of the toolkit lies in its ability to present energy estimations, LCCA, and LCIA outputs with minimum data acquisition and input from the user. User input is limited to plant scale, loading, discharge limits, and sludge disposal option. The absence of a user input area for more detailed wastewater characterisation could be considered a limitation because of the influence that COD fractionation can have on nutrient removal processes and efficiencies. However, it is unlikely that this level of water quality analysis would be carried out during the initial stages of project planning, and therefore, values of COD fractionation have had to be assumed. Specific costs and other regional specific parameters have been soft-coded into the software, but have also been assigned default values based on average data from Ireland. The main constraints for a more universal application of the toolkit outside of Ireland are the hard-coded CAPEX estimations. A platform for the input of detailed, region-specific, CAPEX data would improve the scope of the toolkit. From an environmental perspective, nation-specific electrical grid-mixes, and normalisation factors would have to be included to facilitate region specific life cycle assessments.

Although the primary purpose of the toolkit is to assist with the system selection process, it can also be used in an auditing capacity for existing systems where operators are interested in identifying efficiency losses, or planning benchmarking exercises. Finally, future versions of the toolkit would benefit from a wider selection of systems, system configurations, and sludge management options; however, for the purpose of demonstrating a methodology and framework, the current version has been deemed to sufficient.

7 Systems Analyses

7.1 Introduction

The objective of this chapter is to evaluate the economic and environmental performance of each system in a given scenario by applying the methodologies described in the previous chapters. Systems analyses are carried out through a series of 72 predetermined scenarios that vary with scale, loading, discharge limits, and sludge treatment option. It is assumed in all scenarios that each system has been designed to an optimal standard that limits energy and resource inefficiencies. It is also assumed that the treatment plants are being operated efficiently and that appropriate maintenance schedules are being followed. It should be noted at this stage that the specific costs used in these analyses may vary considerably with location. As mentioned previously, specific cost elements such as energy, chemicals, labour and sludge disposal are soft coded in the DST to allow for regional variation. The analyses that are presented here are intended to demonstrate how a systems economic and environmental performance changes with variations in site-specific conditions, and the importance of considering costs from a life cycle perspective. It is not the intention for the results of these analyses to be a determining factor for any future WWTS selection.

7.2 Scale variation

The International Water Association (IWA) specialist group on small WWTPs has defined small plants as those serving agglomeration sizes of below 2,000 PE, or processing influent flowrates of below 200 m^3/day [223]. In Ireland, the requirement to obtain a discharge licence applies to WWTPs above 500 PE. Therefore, the variations in scale considered for these analyses are examined in three discrete intervals: 500, 1,250, and 2,000 PE.

7.3 Organic load variation

Henze et al. [8] describe high, medium, and low loading as presented in Table 7-1. However, based on water quality analysis from the preliminary LCA study, the high loading described here seems unlikely to occur very often. This may relate to the Irish climate, or there may be extensive infiltration in many of the sewer networks. Regardless of the cause, the more probable range of loading is between what are defined here as 'medium' and 'low', and these are the loading magnitudes that are used in the system analyses. The terminology used to describe loading from this point forward is 'high' and 'low'.

Table 7-1: Typical concentrations of wastewater pollutants

Parameter	*High (mg/l)*	*Medium (mg/l)*	*Low(mg/l)*
BOD	560	350	230
TSS	600	400	250
TN	100	60	30
TP	25	15	6
NH_3	75	45	20
PO_4^3	15	10	4

7.4 Discharge limit variation

Discharge limit variations are classed in four discharge limit (DL) bands as presented in Table 7-2. The values included in each DL band are chosen for the purpose of demonstrating the effect of the gradual introduction of a new pollutant removal requirement. In reality, it is rare that there would be a nitrogen limit and not a phosphorus limit. However, it is considered that the limits presented here are adequate for the purpose of demonstration. Band 'A' is a BOD removal only limit[14], and is the least stringent set of limits that are generally found in coastal area WWTPs that discharge their final effluent to the sea. Moving down through the bands, additional substrates and the level of removal builds gradually. Total phosphorus (TP) was not included because a TP limit does not require any additional unit processes or mechanisms not already included for PO_4^3 removal, unlike the addition of a TN limit that can require the addition of a pre or post anoxic zone, additional pumping, mixing, and monitoring. Systems that are deemed excessive for a DL band are excluded from the analyses. For example, DL bands 1 and 2 do not have a phosphorus removal requirement, and therefore, the AAO system is

[14] All limits include baseline limits for TSS and COD. The substrates presented in Table (1-11) represent the controlling substrates.

not considered in the analyses of any of these scenarios. Similarly, the EA and AO systems are not considered for any scenarios that do not have an ammonia limit.

Table 7-2: Discharge limit variation

Discharge limit band	*BOD (mg/l)*	*NH_3 (mg/l)*	*PO_4^3 (mg/l)*	*TN(mg/l)*
1	30	--	--	--
2	30	1	--	--
3	30	1	0.5	--
4	30	0.5	0.5	15

7.5 Sludge treatment variation

The purpose of including different sludge treatment options is primarily to assess the economic consequences associated with a given treatment option. There are three sludge treatment options included in the systems analyses (Table 7-3)[15]. Option 1 involves sludge treatment with a Volute all-in-one thickening and dewatering unit with polymer and lime addition. The sludge is then removed from the treatment plant site for application to farmland at a cost of €60/m^3 [specific cost sourced from personal communication, (23/03/2017)]. Option 2 involves sludge storage with no treatment and removal from site by an external contractor at a cost of €75/m^3. Option 3 is the employment of sludge drying beds with lime addition for stabilisation and final removal by external contractor. The CW system is assumed to employ option 3 in all scenarios. In all three options, the final terminus is assumed to be farmland because this reflects the most common sludge disposal practice in Ireland. There is scope in future work to include a greater range of disposal options in the DST such as composting and incineration, and include different on-site sludge treatment technologies. However, for the purpose of demonstrating the effect on cost, the three options included here are deemed to be sufficient.

The WWTP scale range adopted for this study is subject to trade-offs between sludge treatment capital and operational costs. It is postulated that for a given system in a given scenario the

[15] There are additional sludge disposal options included in the DST such as transport to parent plant. However, it was determined that no additional knowledge would be gained from its inclusion in the analyses.

economic feasibility of investing in sludge treatment equipment will vary because of the different volumes of sludge that are produced. Sludge production will also vary with changes in discharge limits. For example, WWTPs that have a nitrification requirement may produce less sludge due to extended solid retention times. Conversely, WWTPs with phosphorus limitations may produce more sludge as a result of chemical precipitation. It is, therefore, necessary to assess the influence of varying conditions on the volumes of sludge being produced and the effect that this has on the life cycle costs.

Table 7-3: Sludge treatment options

Sludge option number	*Description*
1	Dewatering – land spreading
2	No dewatering – external contractor – land spreading
3	Drying beds – external contractor – land spreading

Although much of the focus centres on economic cost, there are some environmental implications associated with the choice of sludge treatment option. The most significant environmental impact from sludge disposal is the heavy metal and nutrient deposition in the soil. The LCIA methodology used here determines that the nutrients spread on land could leach into the watercourse and provide the potential for eutrophication. However, there are regulations regarding the proximity to watercourses that nutrients can be spread in order to mitigate risk. Therefore, it can be assumed that the greater risk is the potential for terrestrial toxicity from the metal concentration in the sludge. It is assumed that the reduction in volume achieved by dewatering acts to increase the concentration of metals in the sludge that is being applied to the land. In option 2 it is assumed that the metal concentrations will remain the same from removal from site to final application to land. The level of sludge treatment undertaken by the external contractor is unknown. There may by some dewatering, or sludge bulking applied, which in either case would affect the metal concentration in a negative or positive way respectively. However, without details of the treatment process that occurs after the sludge leaves the site, any assumptions of metal content are merely speculative. The drying beds provide the best alternative to reduce the toxicity risk from heavy metal concentration.

According to Stefanakis and Tsihrintzis [224], the average metal concentrations of the residual sludge in sludge drying reed beds is about 30%, with most of the metals accumulating in the gravel layer (49%), minimal plant uptake (3%) and 16% lost to drained water. Therefore, it is assumed that: option 1 will increase the metal concentration with the increase in dry solids concentration, option 2 will not alter the metal concentration, and option 3 will reduce metal concentrations relative to dry solid concentration. Table 7-4 presents the list of scenarios, and corresponding scale, loading, and discharge limit band.

Table 7-4: Scenarios 1 – 72 description

scenario	Scale	loading	limits	sludge	scenario	Scale	loading	limits	sludge	scenario	Scale	loading	limits	sludge
1	500	high	4	1	**25**	500	high	4	2	**49**	500	high	4	3
2	1250	high	4	1	**26**	1250	high	4	2	**50**	1250	high	4	3
3	2000	high	4	1	**27**	2000	high	4	2	**51**	2000	high	4	3
4	500	low	4	1	**28**	500	low	4	2	**52**	500	low	4	3
5	1250	low	4	1	**29**	1250	low	4	2	**53**	1250	low	4	3
6	2000	low	4	1	**30**	2000	low	4	2	**54**	2000	low	4	3
7	500	high	3	1	**31**	500	high	3	2	**55**	500	high	3	3
8	1250	high	3	1	**32**	1250	high	3	2	**56**	1250	high	3	3
9	2000	high	3	1	**33**	2000	high	3	2	**57**	2000	high	3	3
10	500	low	3	1	**34**	500	low	3	2	**58**	500	low	3	3
11	1250	low	3	1	**35**	1250	low	3	2	**59**	1250	low	3	3
12	2000	low	3	1	**36**	2000	low	3	2	**60**	2000	low	3	3
13	500	high	2	1	**37**	500	high	2	2	**61**	500	high	2	3
14	1250	high	2	1	**38**	1250	high	2	2	**62**	1250	high	2	3
15	2000	high	2	1	**39**	2000	high	2	2	**63**	2000	high	2	3
16	500	low	2	1	**40**	500	low	2	2	**64**	500	low	2	3
17	1250	low	2	1	**41**	1250	low	2	2	**65**	1250	low	2	3
18	2000	low	2	1	**42**	2000	low	2	2	**66**	2000	low	2	3
19	500	high	1	1	**43**	500	high	1	2	**67**	500	high	1	3
20	1250	high	1	1	**44**	1250	high	1	2	**68**	1250	high	1	3
21	2000	high	1	1	**45**	2000	high	1	2	**69**	2000	high	1	3
22	500	low	1	1	**46**	500	low	1	2	**70**	500	low	1	3
23	1250	low	1	1	**47**	1250	low	1	2	**71**	1250	low	1	3
24	2000	low	1	1	**48**	2000	low	1	2	**72**	2000	low	1	3

7.6 Discussion

The discussion presented here reviews the results of the economic assessment, beginning with an overview of CAPEX estimations. Operational expenditure results are presented and the effects of variation in site-specific conditions are discussed individually. The economic assessment concludes with a discussion of the LCCA results. This is followed by the environmental assessment. The chapter concludes with further discussion of some of the more significant findings of the analyses.

7.6.1 Capital expenditure

Table 7-5 presents the CAPEX totals for each system in all scenarios. System CAPEX is primarily a function of scale, and therefore, is not influenced by variations in organic load. Sludge option 1 includes an additional 5% of the total CAPEX for the Volute dewatering unit. The variation in CAPEX due to additional land for sludge drying beds was found to be negligible (< 0.4%), which resulted in similar CAPEX totals for sludge options 2 and 3.

Table 7-5: Capital expenditure estimations (€1 x 10^6)

	Sludge option 1			Sludge option 2 + 3		
	500	1250	2000	500	1250	2000
AO	0.95	1.34	1.60	0.90	1.27	1.52
AAO	1.00	1.39	1.64	0.95	1.32	1.56
CMAS	0.91	1.32	1.59	0.86	1.25	1.52
CW	0.26	0.61	0.97	0.26	0.61	0.97
EA	1.00	1.45	1.75	0.95	1.38	1.67
IFAS	0.94	1.39	1.70	0.89	1.32	1.62
OD	0.99	1.44	1.74	0.94	1.36	1.65
RBC	1.04	1.50	1.82	0.98	1.43	1.73
SBR	1.19	1.61	1.87	1.13	1.53	1.78
TF	0.88	1.28	1.54	0.84	1.21	1.47

The CW system had the lowest CAPEX in all scenarios. Economies of scale were not as evident with the CW system as with the electro-mechanical systems, and the relationship between CAPEX/PE and scale was generally linear (Figure 7-1). Extrapolating CAPEX estimations beyond the scale range in question would see the electro-mechanical and the CW system reach CAPEX parity at agglomeration scales of between 4,500 and 5,000 PE depending on site-specific conditions.

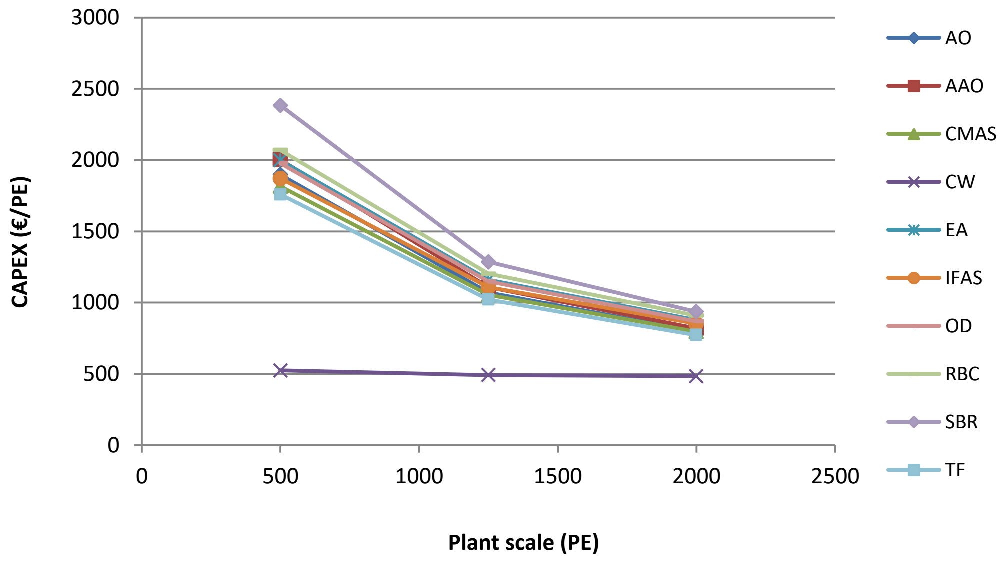


Figure 7-1: Capital expenditure per PE. Scenarios 1 – 3

A specific land cost of €5/m^2 was chosen for the study. The cost of land was found to be a small percentage of total CAPEX for all systems including CWs, which in most cases accounted for less than 10% of the total CW CAPEX, and less than 1% for electro-mechanical systems. The cost of land will vary with location; however, for electro-mechanical systems to compete with CW systems on a CAPEX basis, the specific cost of land would have to exceed €45/m^2 at 2,000 PE and €191/m^2 at 500 PE. Therefore, it is more probable that the availability of land rather than cost will be the determining factor in the implementation of CW systems. Of the electro-mechanical systems, the trickling filters had the lowest CAPEX across all scales, which was a constant 12% lower than the next lowest CAPEX of the CMAS system. Sequence batch reactor systems had the highest CAPEX in all scenarios, which is consistent with the findings from the study carried out by Jafarinejad [225]. The variation in system CAPEX from lowest to highest was 25% across all scales, which falls within the margin of uncertainty observed at 500 PE during the CAPEX model validation.

7.6.2 Operational expenditure

System OPEX ranged from 12 – 225 €/PE-year. The lowest OPEX was estimated for the CW system for a 2,000 PE agglomeration with low organic loading. Operational costs for CW systems were the lowest in every scenario and were dominated by the cost of labour which varied from 65 – 91% of the operational costs. The remainder of the OPEX discussion focuses on the electro-mechanical systems.

Table 7-6 presents the electro-mechanical systems with the lowest OPEX for all scenarios.

Table 7-6: Operational cost results for electro-mechanical systems

		Sludge option 1			Sludge option 2			Sludge option 3		
	Load	Scenario	System	(€/PE-year)	Scenario	System	(€/PE-year)	Scenario	System	(€/PE-year)
DL band 4	High	S1	RBC	94.0	S25	RBC	139.7	S49	RBC	89.9
	High	S2	AAO	57.1	S26	RBC	109.6	S50	AAO	52.5
	High	S3	AAO	47.1	S27	RBC	102.1	S51	AAO	42.3
	Low	S4	RBC	75.2	S28	RBC	99.1	S52	RBC	73.3
	Low	S5	RBC	40.7	S29	RBC	69.1	S53	RBC	38.0
	Low	S6	RBC	32.0	S30	RBC	61.6	S54	RBC	29.2
DL band 3	High	S7	RBC	92.8	S31	RBC	138.4	S55	RBC	88.6
	High	S8	AAO	59.1	S32	RBC	109.2	S56	RBC	54.1
	High	S9	AAO	49.5	S33	RBC	101.9	S57	AAO	44.5
	Low	S10	RBC	72.6	S34	RBC	96.6	S58	RBC	70.7
	Low	S11	RBC	39.0	S35	RBC	67.4	S59	RBC	36.3
	Low	S12	RBC	30.6	S36	RBC	60.1	S60	RBC	27.7
DL band 2	High	S13	RBC	79.9	S37	RBC	111.9	S61	RBC	77.3
	High	S14	RBC	46.2	S38	RBC	83.7	S62	RBC	42.9
	High	S15	RBC	37.8	S39	RBC	76.6	S63	RBC	34.3
	Low	S16	RBC	67.3	S40	RBC	83.9	S64	RBC	66.2
	Low	S17	RBC	33.6	S41	RBC	55.7	S65	RBC	31.7
	Low	S18	RBC	25.2	S42	RBC	48.6	S66	RBC	23.1
DL band 1	High	S19	TF	68.7	S43	TF	105.7	S67	TF	65.9
	High	S20	TF	36.1	S44	TF	78.5	S68	TF	32.5
	High	S21	TF	27.9	S45	TF	71.7	S69	TF	24.1
	Low	S22	TF	63.3	S46	RBC	80.0	S70	TF	62.2
	Low	S23	TF	30.6	S47	RBC	52.6	S71	TF	28.7
	Low	S24	TF	22.4	S48	RBC	45.8	S72	TF	20.3

The attached growth systems were generally found to have lower operational costs than the suspended growth systems. The TF system had the lowest OPEX (€20.3/PE-year, scenario 72) and had consistently lower costs in DL band 1. The RBC system had the lowest OPEX in all scenarios of DL band 2, and most scenarios in sludge option 2 with values ranging from 23.1 – 139.7 €/PE-year. The RBC system has the advantage of low energy consumption, and produces higher density, lower volume sludge. This is significant with sludge option 2 where sludge volumes are at their highest. The AAO system had the lowest OPEX with high loading in DL band 3 and 4 at 1,250 and 2,000 PE. The highest OPEX (€225/PE-year) estimation was for the EA system in scenario 25 where the loading and discharge limits are at their highest and lowest respectively. Notwithstanding the reduced sludge volumes that are achieved with EA systems, the increased energy demand results in consistently higher operational costs in most scenarios, with only the OD system having higher OPEX in some cases.

7.6.2.1 Effect of site-specific variation on operational cost

The effect that site-specific variation has on OPEX is different for each treatment system. Discussion of the variation in OPEX for each system in every scenario is not practical and deemed excessive. For demonstration purposes, the effect that site-specific variation has on the CMAS system's OPEX is discussed here.

7.6.2.1.1 Variation in scale

The effect of an increase in scale on the OPEX distribution for most systems is a reduction in the percentage of OPEX attributed to labour. Figure 7-2 and Figure 7-3 show an almost two-fold reduction in the percentage of labour for the CMAS system from scenario 1 – 3. As the system size increases the other operational cost elements experience a much higher rate of increase relative to plant scale. Energy costs increase from €19 - €72/d, chemicals from €18 - €73/d, and sludge disposal from €17 - €67/d, but the cost of labour increases by only €8 (€92 - €100/d).

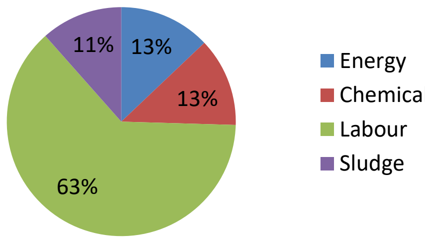


Figure 7-2: Operational cost distribution of CMAS system, scenario 1

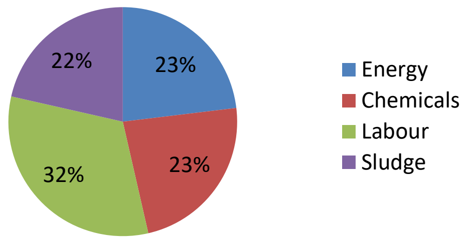


Figure 7-3: Operational cost distribution of CMAS system, scenario 3

Although there may be an increase in the hours spent on particular areas of operation and maintenance relative to plant scale, some areas of operation will require as much time for a 500 PE plant as a 2,000 PE plant e.g. the time spent on water quality analysis will be the same regardless of plant scale.

7.6.2.1.2 Variation in organic loading

A reduction in organic loading reduces required quantities of energy and chemicals, and also reduces sludge handling costs. Figure 7-4 and Figure 7-5 present the OPEX distribution of the CMAS system with high and low loading respectively in DL band 4, sludge option 1.

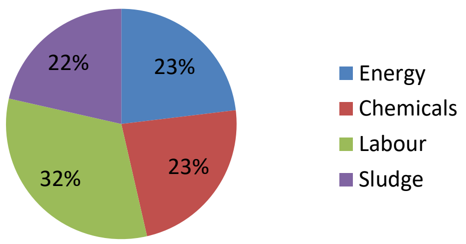


Figure 7-4: Operational cost distribution of CMAS system, scenario 3

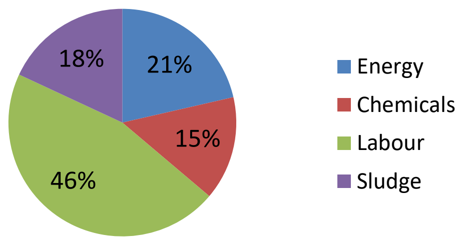


Figure 7-5: Operational cost distribution of CMAS system, scenario 6

There is a 30% overall reduction in OPEX from high to low loading, from €312/d - €219/d. The largest reduction occurs in the cost of chemicals from €73/d - €32/d. Energy cost is reduced from €72/d - €47/d, and sludge disposal from €67/d - €40/d. The cost of labour is not affected by variations in organic load; however, it is conceivable that variations in sludge volume may necessitate additional sludge handling time and should be factored in to labour hour calculations.

7.6.2.1.3 Variation in discharge limits

The overall OPEX reduction for the CMAS system from DL band 4 to DL band 1 is 31% (Figure 7-6 and Figure 7-7). The largest reduction in cost is attributed to chemicals (€73 – €16/d). Energy costs are reduced from €72 - €42/d, and labour from €100 - €91/d. It had been postulated that the reduction in the required SRT for higher limits would result in a higher sludge volumes. However, the addition of chemicals used for phosphorus precipitation and sludge dewatering and stabilisation produced a marginally higher sludge volume for the CMAS system in DL band 4, which results in a decrease of €11/d (€67 - €56/d) from DL band 4 to DL band 1.

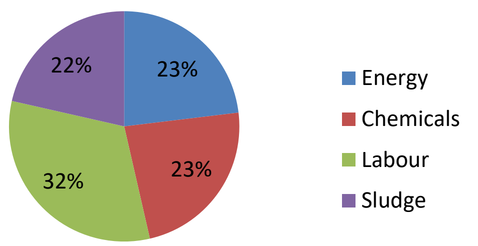


Figure 7-6: Operational cost distribution of CMAS system, scenario 3

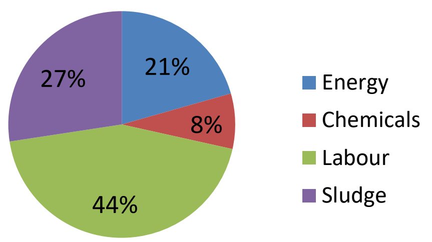


Figure 7-7: Operational cost distribution of CMAS system, scenario 21

7.6.2.1.4 Effect of variation in sludge disposal option

In scenarios without sludge dewatering, it is the cost of sludge disposal that dominates the OPEX distribution. Figure 7-8 and Figure 7-9 show a 61% difference in the percentage of OPEX attributed to sludge disposal for a CMAS system without sludge dewatering and with sludge dewatered in drying beds. However, the actual reduction in sludge disposal cost is 94% from €580/d to €32/d. It is worth pointing out once again that these figures refer only to the cost of removing the sludge from site and do not include the cost of chemicals and sludge handling. The additional costs incurred in the drying bed option include a 17% increase in the cost of chemicals due to sludge stabilisation, and a 13% increase in labour due to sludge handling. There is no difference in the cost of energy between these sludge disposal categories. The variation in the magnitude of the difference between systems is minimal. Attached growth and EA systems may have lower percentages of OPEX attributed to sludge disposal, but the magnitude of the difference is largely the same.

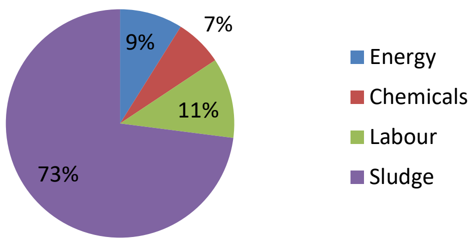


Figure 7-8: Operational cost distribution of CMAS system, scenario 25

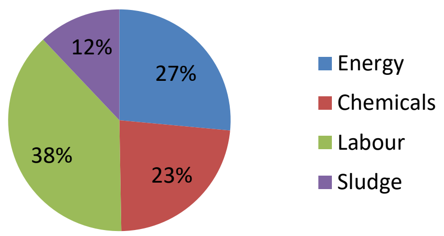


Figure 7-9: Operational cost distribution of CMAS system, scenario 49

7.6.3 Life cycle cost

The LCCA determined that the CW system had the lowest LCC in all scenarios, and in many cases were orders of magnitude lower than the electro-mechanical systems. Therefore, the remainder of the LCC discussion focuses on the electro-mechanical systems. Table 7-7 and Table 7-8 present the electro-mechanical systems with the lowest and highest LCC respectively.

Table 7-7: Life cycle cost analyses (lowest LCC)

		Sludge option 1			Sludge option 2			Sludge option 3		
	Load	**Scenario**	**System**	**LCC (€1x10^6)**	**Scenario**	**System**	**LCC (€1x10^6)**	**Scenario**	**System**	**LCC (€1x10^6)**
DL band 4	High	S1	AAO	1.94	S25	TF	2.31	S49	AAO	1.85
	High	S2	AAO	2.83	S26	RBC	4.00	S50	AAO	2.66
	High	S3	AAO	3.58	S27	RBC	5.52	S51	AAO	3.33
	Low	S4	TF	1.73	S28	TF	1.87	S52	TF	1.67
	Low	S5	AAO	2.51	S29	TF	3.02	S53	AAO	2.38
	Low	S6	AAO	3.05	S30	TF	4.02	S54	AAO	2.87
DL band 3	High	S7	TF	1.89	S31	TF	2.20	S55	TF	1.80
	High	S8	AAO	2.92	S32	TF	3.87	S56	AAO	2.75
	High	S9	AAO	3.73	S33	TF	5.38	S57	AAO	3.48
	Low	S10	TF	1.67	S34	TF	1.81	S58	TF	1.60
	Low	S11	TF	2.40	S35	TF	2.90	S59	TF	2.28
	Low	S12	TF	2.99	S36	TF	3.84	S60	TF	2.80
DL band 2	High	S13	TF	1.78	S37	TF	1.99	S61	TF	1.71
	High	S14	TF	2.68	S38	TF	3.35	S62	TF	2.54
	High	S15	TF	3.42	S39	TF	4.57	S63	TF	3.21
	Low	S16	TF	1.62	S40	TF	1.71	S64	TF	1.56
	Low	S17	TF	2.30	S41	TF	2.66	S65	TF	2.18
	Low	S18	TF	2.81	S42	TF	3.47	S66	TF	2.66
DL band 1	High	S19	TF	1.74	S43	RBC	2.01	S67	TF	1.66
	High	S20	TF	2.49	S44	RBC	3.32	S68	TF	2.33
	High	S21	TF	3.10	S45	RBC	4.44	S69	TF	2.85
	Low	S22	TF	1.64	S46	TF	1.76	S70	TF	1.57
	Low	S23	TF	2.25	S47	TF	2.71	S71	TF	2.12
	Low	S24	TF	2.71	S48	TF	3.52	S72	TF	2.53

Table 7-8: Life cycle cost analyses (highest LCC)

		Sludge option 1			Sludge option 2			Sludge option 3		
	Load	Scenario	System	LCC (€1x10^6)	Scenario	System	LCC (€1x10^6)	Scenario	System	LCC (€1x10^6)
DL band 4	High	S1	SBR	2.20	S25	EA	2.95	S49	SBR	2.09
	High	S2	OD	3.36	S26	EA	5.46	S50	OD	3.19
	High	S3	OD	4.42	S27	EA	7.81	S51	OD	4.17
	Low	S4	SBR	2.00	S28	EA	2.40	S52	SBR	1.92
	Low	S5	SBR	2.75	S29	EA	4.10	S53	EA	2.67
	Low	S6	EA	3.49	S30	EA	5.63	S54	OD	3.26
DL band 3	High	S7	SBR	2.21	S31	EA	2.93	S55	SBR	2.11
	High	S8	SBR	3.31	S32	EA	5.45	S56	EA	3.13
	High	S9	OD	4.30	S33	EA	7.80	S57	OD	4.06
	Low	S10	SBR	1.98	S34	EA	2.38	S58	SBR	1.90
	Low	S11	SBR	2.74	S35	EA	4.07	S59	SBR	2.59
	Low	S12	OD	3.38	S36	EA	5.60	S60	OD	3.21
DL band 2	High	S13	SBR	2.10	S37	EA	2.72	S61	SBR	2.01
	High	S14	SBR	3.05	S38	EA	4.93	S62	EA	2.90
	High	S15	OD	3.90	S39	EA	6.99	S63	OD	3.70
	Low	S16	SBR	1.94	S40	EA	2.28	S64	SBR	1.86
	Low	S17	EA	2.67	S41	EA	3.84	S65	EA	2.56
	Low	S18	EA	3.30	S42	EA	5.23	S66	EA	3.14
DL band 1	High	S19	SBR	2.00	S43	OD	2.76	S67	SBR	1.90
	High	S20	OD	2.96	S44	OD	5.11	S68	OD	2.79
	High	S21	OD	3.78	S45	OD	7.33	S69	OD	3.54
	Low	S22	SBR	1.85	S46	OD	2.22	S70	SBR	1.77
	Low	S23	OD	2.50	S47	OD	3.77	S71	OD	2.44
	Low	S24	OD	3.16	S48	OD	5.16	S72	OD	2.98

Life cycle costs ranged from a low of €1.56 x 10^6 (TF, S64) to a high of €7.81 x 10^6 (EA, S27). Sludge options 1 and 3 produced similar results in terms of the systems with the lowest life cycle costs. The attached growth systems had the lowest LCCs in all but 14 of the 72 scenarios. The TF system had the lowest LCC from scenarios 10 to 24 in sludge option 1, and from 58 to 72 in sludge option 3. It also had the lowest LCC in 19 of the 25 scenarios in sludge option 2, with the RBC system accounting for the remainder. The TF LCC values ranged from €1.56 x 10^6 (S64) to €5.38 x 10^6 (S33). The AAO system had the lowest LCC in 14 scenarios, ten of which are in DL band 4 sludge options 1 and 3, and the remaining four in DL band 3. The EA, OD and SBR systems had the highest life cycle costs. The EA LCC values ranged from €2.28 x 10^6 (S40) to €7.81 x 10^6 (S27), the OD from €2.22 x 10^6 (S46) to €7.33 x 10^6 (S45), and the SBR from €1.77 x 10^6 (S70) to €3.31 x 10^6 (S8).

The TF system had the lowest LCC in 21 of the 24 scenarios without ammonia removal. This is mainly due to the reduction in pumping energy requirements for BOD removal only scenarios. The primary factors that affect pumping energy requirements are the specific organic loading rate (OLR), and the hydraulic loading rate (HLR). Firstly, the OLR for BOD removal only (0.6 – 2.4 kg BOD/m^3.d [28]) is much higher than for BOD and nitrogen removal (0.08 – 0.4 kg BOD/m^3.d [28]), which means that a much greater growth media surface area is required for nitrogen removal, which increases pipe lengths, static head, and distributor arm head. Secondly, the HLR for BOD removal only is much higher than for BOD and nitrogen removal. To maintain minimum recommended wetting rates, the recirculation ratio is higher for nitrogen removal; therefore, a greater volumetric flow is being pumped. This means that the addition of an ammonia removal limit results in a significant increase in pumping energy for TF systems.

The AAO system was the optimal choice at 1,250 and 2,000 PE in scenarios with phosphorus reduction requirements (Table 9-7). There are several contributing factors considered here. Firstly, there are reductions in phosphorus precipitating chemical requirements through the employment of enhanced biological phosphorus removal. This accounted for a 2.5 – 5% lower LCC than that of the AO system. Secondly, the inclusion of a pre-anoxic tank reduces the oxygen demand as oxygen is released during nitrate reduction; thus, lowering aeration requirements. Finally, the AAO system does not require the addition of external carbon as do systems with post-anoxic zones. Figure 7-10 and Figure 7-11 illustrate the variation in the chemical cost distribution profiles of an AAO system (€32/d chemical costs) and a CMAS system (€50/d chemical costs) with a post anoxic tank with external carbon source (scenario 1).

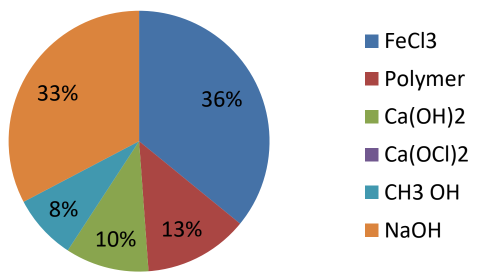


Figure 7-10: Chemical cost distribution of CMAS system (S1)

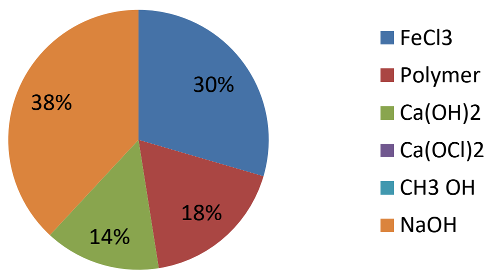


Figure 7-11: Chemical cost distribution of AAO system (S1)

The attached growth systems perform optimally in sludge option 2. The primary reason for this relates to the sludge dry solids concentration (DSC). Attached growth systems generally produce secondary sludge with higher dry solid concentrations. Trickling filter sludge or *humus* DSC is reported to range from 1 - 4% [114]. The average TF and RBC DSC value adopted for this study is 2.3%, whereas the value adopted for WAS is 1.3%. Although the difference is small, the effect that this has on sludge volume is significant. For a 2,000 PE plant with high organic loading and phosphorus removal the sludge mass is 200 kg DS/d. Without any treatment, the TF system sludge volume is 5.06 m^3/d, and the AAO system is 6.95 m^3/d. The difference of 1.89 m^3/d equates to an additional removal cost of €141.75/d, and €51,738/year for disposal by external contractor at a cost of €75/m^3. This has a significant impact on the operational costs over the lifetime of the system. Similarly, despite the higher CAPEX associated with the SBR system, because of the 4.3% sludge DSC value adopted for the study, the systems outperformed other CAS based systems in this sludge disposal category [226].

The EA, OD, and SBR systems incur higher aeration energy demands as a result of the lack of a primary settling tank which increases the organic load going into the aeration process. The low OTE associated with the horizontal surface aerator used in the OD system was found to be the most significant factor in the energy demand of this system. The AAO and AO systems were found to perform better at larger scales when discharge limits were low. Both systems benefit from the oxygen gain associated with having a pre-anoxic tank and the reduction in alkalinity addition. The post anoxic tank used for the CMAS and TF systems does not have the same oxygen benefits. Furthermore, systems with the post-anoxic configuration used for denitrification also incur the cost of carbon addition. The attached growth systems performed better with less stringent limits and were the systems least affected by variations in organic load.

7.6.4 Variation in scale

The effect of increasing scale is a reduction in the percentage of the LCC that is attributed to capital expenditure. Figure 7-12 presents the LCC distribution for all systems in scenario 1. The CAPEX accounts for an average of 48% of total life cycle costs. Operational expenditure accounts for 37%, energy for 9%, and parts for 6%. In Figure 7-13 (scenario 3) CAPEX is reduced to 42%, OPEX to 36%, energy is increased to 18%, and the parts cost is reduced to 5%. The increase in the percentage of the LCC attributed to energy occurs because scale economies are higher for other LCC elements i.e. CAPEX and labour costs experience a greater decrease with decreasing scale than energy costs. Most systems exhibit a reduction in specific energy use with an increase in scale e.g. the estimated CMAS specific energy consumption is reduced from 0.75 – 0.72 kWh/m^3 from 500 – 2,000 PE. Furthermore, because the discount rate used to calculate the energy UPV (5%) is greater than the OPEX UPV (3.5%), differences in the rate of change of LCC with respect to scale are increased.

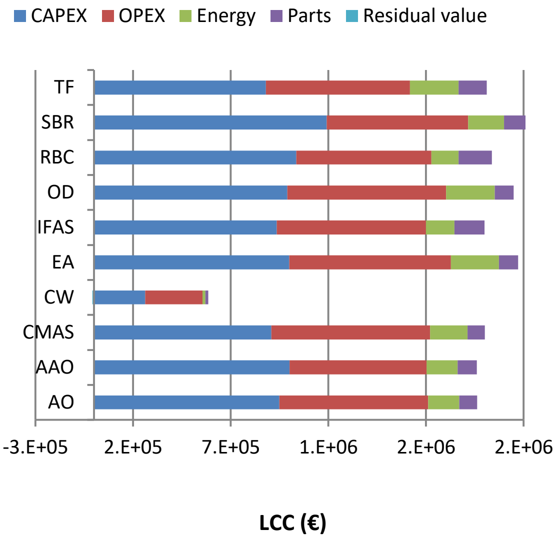


Figure 7-12: Life cycle cost distribution (S1) 500 PE

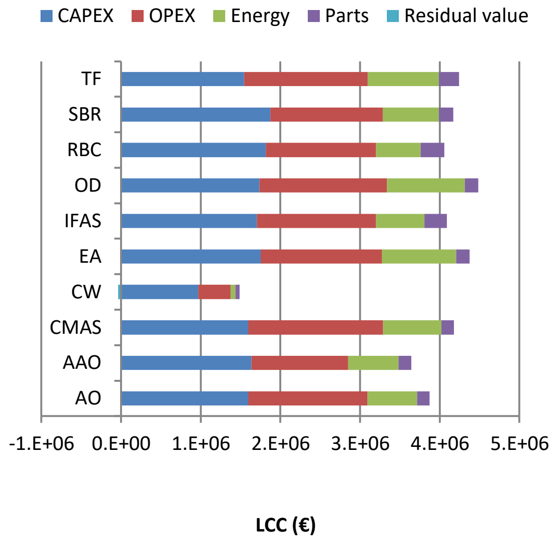
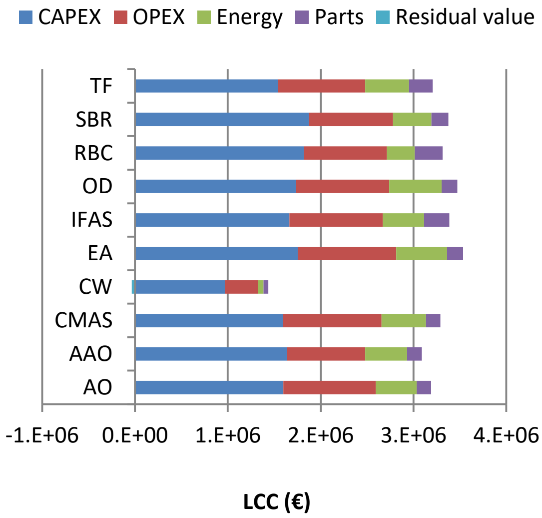


Figure 7-13: Life cycle cost distribution (S3) 2,000 PE

7.6.4.1 Load variation

The variation in organic loading between scenarios 1 and 4, from low loading to high loading (Figure 7-14 and Figure 7-15) results in an average increase in OPEX (29 – 36%), and in energy (14 – 18%). Capital expenditure is a function of scale only and therefore, while the estimated CAPEX does not change with respect to loading, the percentage of the LCC attributed to CAPEX decreases from 51 – 42% because of the increase in operational costs. The example of the effect of loading presented in Figure 9-14 and Figure 9-15 is based on sludge option 1 where the cost of sludge disposal is minimal. Similar variations in cost were observed for scenarios in sludge options 3. In sludge option 2 where sludge disposal costs are at a maximum, OPEX is increased from 52 – 60%, energy from 9 – 11%, and CAPEX is reduced from 35 – 25% of the total lifecycle cost (scenarios 30 – 27).



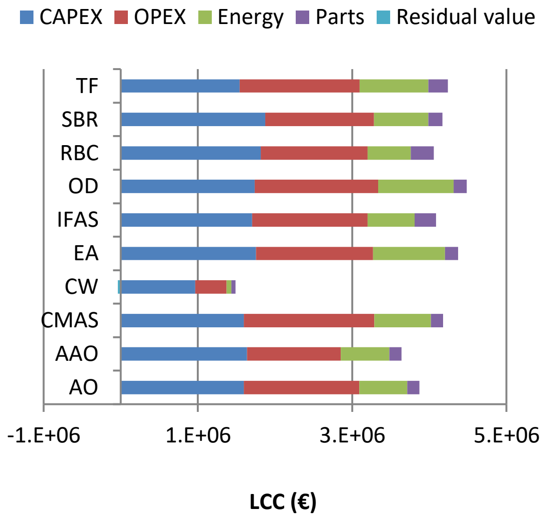


Figure 7-14: Life cycle cost distribution (S6) low loading

Figure 7-15: Life cycle cost distribution (S3) high loading

7.6.4.2 Discharge limit variation

The variation in discharge limits from DL band 4 to DL band 1, sludge option 1, results in a 20% average reduction of total life cycle cost for systems that operate in all DL bands. Operational costs are reduced from 36 – 33%, energy from 18 – 13% and CAPEX is increased from 42 – 48% of the total life cycle cost. The reduction in LCC is mainly due to reductions in energy and chemical use; however, the CAPEX estimations are based primarily on scale and surface area (± 5% for investment in mechanical dewatering unit process) and do not account for reductions in construction costs associated with decreasing ammonia and nitrate removal requirements. It is, therefore assumed that the total percentage reduction in the systems' LCCs will be greater when CAPEX adjustments are made. It is unclear the extent to which the CAPEX adjustment will have on the outcome of the life cycle cost analysis. While suspended growth systems require additional aeration tank volume and diffusers to move from a BOD removal only, to BOD and ammonia removal, attached growth systems will require a significant increase in growth media material. Therefore, it is conceivable that attached growth systems may exhibit even greater reductions in LCC as the discharge limits become less stringent.

7.6.4.3 Sludge disposal variation

Variation in the method of sludge disposal found that in all scenarios drying beds had the lowest LCCs of the three options evaluated. The variation in the LCCs between option 1 and option 3 ranged from 4 - 15%. The smallest difference in the values between options 1 and 3 – when the LCC with the drying bed option is at its highest - occurs at small scales when organic loading is low, which results in a lower surface area requirement because drying bed surface area is a function of organic loading. Land is assumed not to lose its value and therefore, systems with large surface areas have a greater residual value at the end of their lifetime. The percentage difference in the LCCs between options 1 and 2 ranged from 1 - 49%. Option 1 always yielded a lower LCC than option 2. The largest difference in values occurred at large scales, high loading, and high limits when SRTs for suspended growth systems were at their lowest and sludge production at its highest (Figure 7-16 and Figure 7-17).

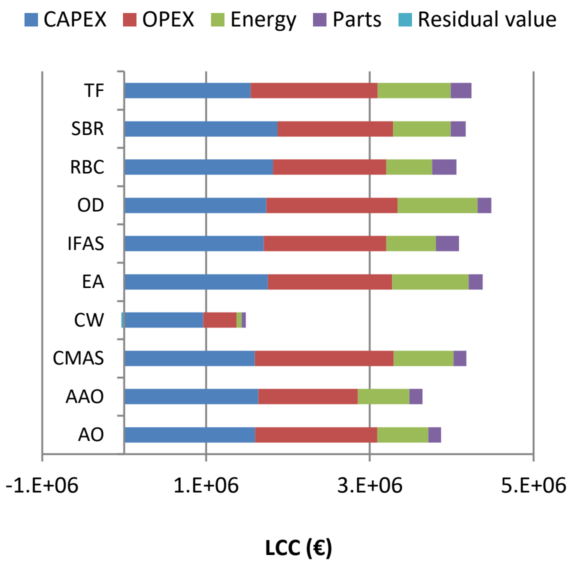


Figure 7-16: Life cycle cost distribution (S3)

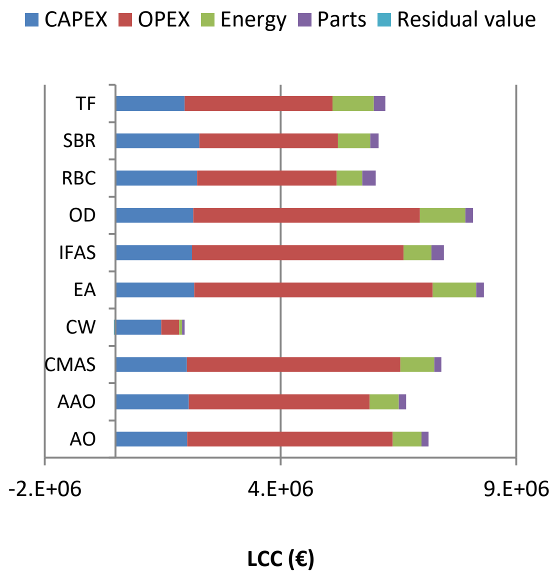


Figure 7-17: Life cycle cost distribution (S27)

Because the cost of both methods of disposal is dependent on volume, it could have been assumed that the option to dewater and land spread would result in much lower lifecycle costs. Moreover, the specific cost of removal of sludge from the site for land spreading was 20% lower than the external contractor at €60 and €75/m^3 respectively. However, for small scale systems with low loading and ammonia reduction requirements, the external contractor option becomes more economical when the specific cost of disposal falls below €65/m^3. For the contractor option to be economically feasible in all scenarios the specific cost of sludge removal from site would have to fall just below €7/m^3 (Figure 7-18).

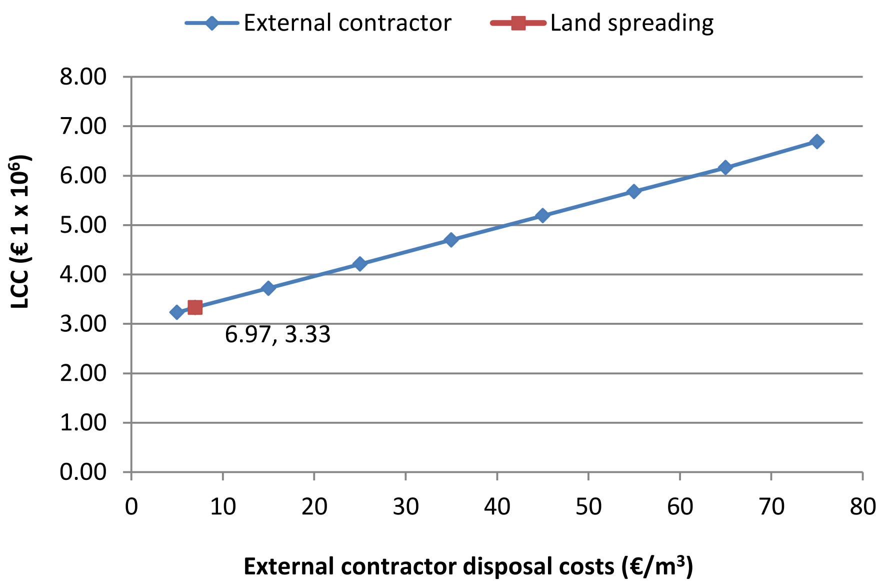


Figure 7-18: Variation in LCC for a CMAS system in scenario 69 with variations in the specific disposal cost of external contractor

7.6.5 Life cycle impact assessment

The CML LCIA methodology includes the eleven impact categories as described in Chapter 4 (Table 4-2). The decision support tool includes a resource and emissions distribution profile for all categories; however, provision of results for all eleven categories has been deemed superfluous to the objective of demonstration, and would not provide any further benefit or understanding. The categories included in the discussion are those that demonstrate clearly the effect of changes in scenario; these include: GWP, AP, EP, ADPf, and HTP. Elemental ADP is also discussed for the purpose of illustrating differences in the outputs of the ADPe and ADPf categories. Impact category outputs have been normalised with Western European normalisation factors (2001 – 2013) [189].

7.6.5.1 General overview

Provision of all LCIA results for each impact category in all scenarios is not practical. The results presented below (Figure 7-19 to Figure 7-23) are from scenarios 1 – 3. These scenarios were chosen to provide a general overview because they include all considered systems.

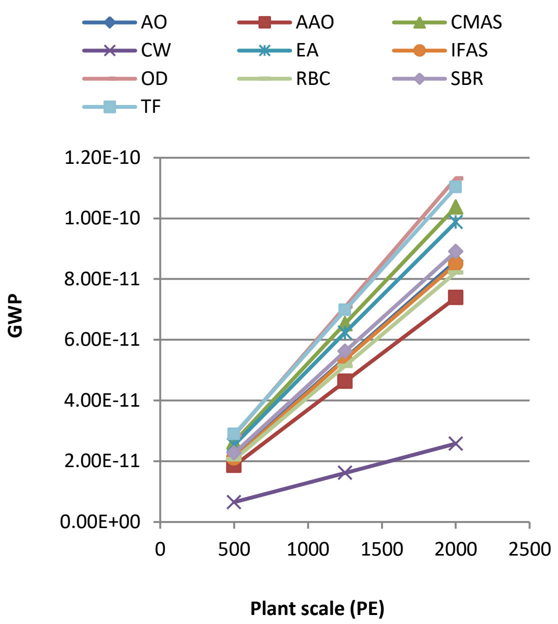


Figure 7-19: GWP (S1-S3)

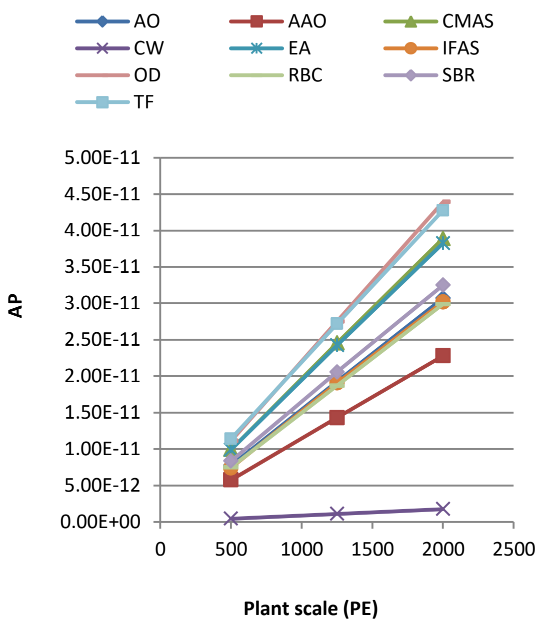


Figure 7-20: AP (S1-S3)

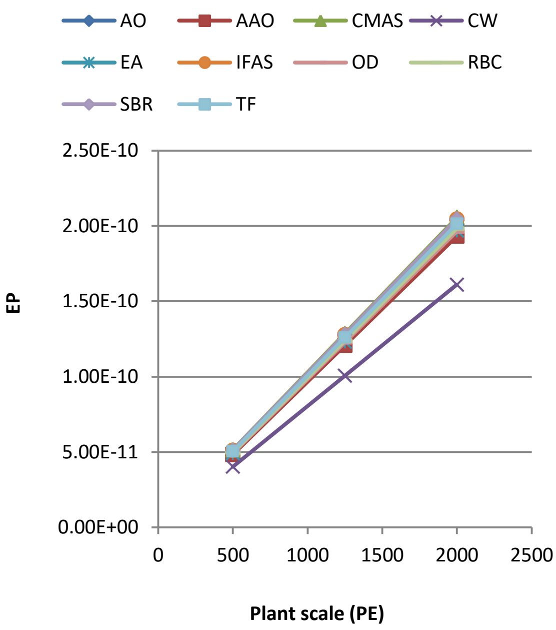


Figure 7-21: EP (S1-S3)

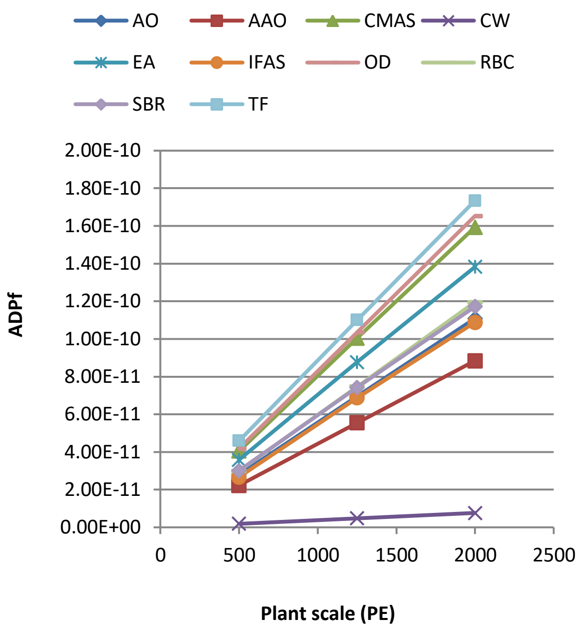


Figure 7-22: ADPf (S1-S3)

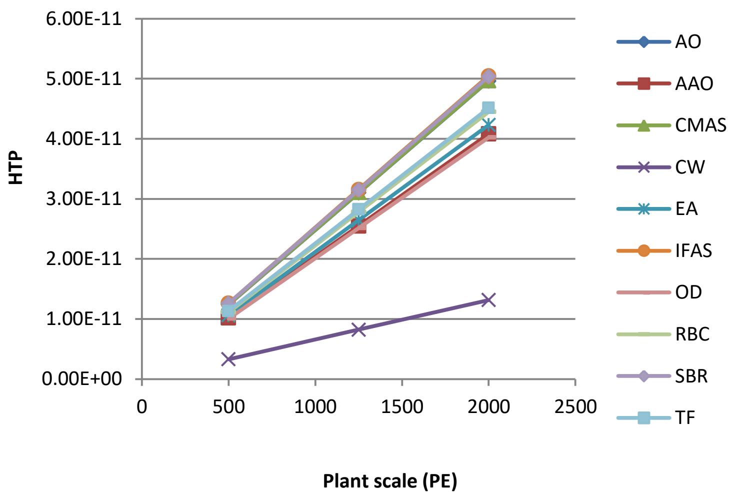


Figure 7-23: HTP (S1-S3)

The CW systems had the lowest environmental impact in all impact categories and scenarios. The remaining discussion is limited to the electro-mechanical systems. The electro-mechanical systems' environmental profiles varied with the amount of energy and chemicals used. The general trend observed was that the AAO system exhibited the best performance in DL band 4 where it benefited from the combination of EBPR and the reduction in oxygen demand through the use of a pre-anoxic zone. The RBC system exhibited the best environmental performance in the lower DL bands as the required growth media surface area reduced and lowered the required motor power. The effect of the variation in sludge disposal option had no effect on which system yielded the lowest impact. Therefore, to avoid repetition the discussion will be limited to the first sludge option except in cases where there are points of significance in sludge option variation. The effects of variation in site specific conditions and sludge option are discussed with each impact category in the following sections.

7.6.5.2 Global warming potential

Proceeding from most to least stringent DL bands, the AAO system had the lowest GWP in DL bands 3 and 4 when loading was high, and the RBC system had the lowest LCC when the loading was low (Table 7-9 and

Table 7-10). The OD system had the highest output in both DL bands regardless of loading. The RBC had the lowest GWP in DL bands 2 (Table 7-11) and DL band 1 (Table 7-12). The OD and EA system had the highest GWP in DL band 2. In DL band 1, the RBC system had the lowest GWP and CMAS systems had the highest GWP when loading was high and low respectively.

Table 7-9: Global warming DL band 4, scenarios 1 – 6

	High			Low		
PE	500	1250	2000	500	1250	2000
AO	2.16E-11	5.38E-11	8.59E-11	1.43E-11	3.56E-11	5.66E-11
AAO	1.85E-11	4.63E-11	7.39E-11	1.27E-11	3.15E-11	5.01E-11
CMAS	2.63E-11	6.52E-11	1.04E-10	1.57E-11	3.90E-11	6.20E-11
CW	6.44E-12	1.61E-11	2.58E-11	6.08E-12	1.52E-11	2.43E-11
EA	2.53E-11	6.23E-11	9.87E-11	1.57E-11	3.86E-11	6.10E-11
IFAS	2.09E-11	5.34E-11	8.49E-11	1.38E-11	3.56E-11	5.70E-11
OD	2.85E-11	7.08E-11	1.13E-10	1.64E-11	4.04E-11	6.45E-11
RBC	2.05E-11	5.13E-11	8.21E-11	1.24E-11	3.10E-11	4.97E-11
SBR	2.27E-11	5.62E-11	8.90E-11	1.40E-11	3.48E-11	5.52E-11
TF	2.89E-11	6.97E-11	1.10E-10	1.62E-11	3.88E-11	6.12E-11

Table 7-10: Global warming DL band 3, scenarios 7 – 12

	High			Low		
PE	500	1250	2000	500	1250	2000
AO	2.40E-11	5.98E-11	9.50E-11	1.49E-11	3.78E-11	6.01E-11
AAO	2.06E-11	5.14E-11	8.19E-11	1.32E-11	3.36E-11	5.34E-11
CMAS	2.59E-11	6.42E-11	1.02E-10	1.52E-11	3.81E-11	6.05E-11
CW	6.44E-12	1.61E-11	2.58E-11	6.08E-12	1.52E-11	2.43E-11
EA	2.64E-11	6.50E-11	1.03E-10	1.59E-11	3.91E-11	6.20E-11
IFAS	2.30E-11	5.92E-11	9.49E-11	1.38E-11	3.56E-11	5.75E-11
OD	2.74E-11	6.79E-11	1.09E-10	1.59E-11	3.93E-11	6.27E-11
RBC	2.06E-11	5.16E-11	8.26E-11	1.19E-11	2.98E-11	4.78E-11
SBR	2.55E-11	6.32E-11	1.00E-10	1.45E-11	3.60E-11	5.71E-11
TF	2.51E-11	6.07E-11	9.59E-11	1.43E-11	3.44E-11	5.42E-11

Table 7-11: Global warming DL band 2, scenarios 13 – 18

	High			Low		
PE	500	1250	2000	500	1250	2000
AO	1.96E-11	4.87E-11	7.73E-11	1.32E-11	3.36E-11	5.33E-11
CMAS	2.15E-11	5.32E-11	8.43E-11	1.35E-11	3.39E-11	5.38E-11
CW	6.35E-12	1.59E-11	2.54E-11	5.99E-12	1.50E-11	2.40E-11
EA	2.19E-11	5.39E-11	8.53E-11	1.43E-11	3.49E-11	5.52E-11
IFAS	1.86E-11	4.82E-11	7.72E-11	1.21E-11	3.14E-11	5.07E-11
OD	2.29E-11	5.69E-11	9.08E-11	1.42E-11	3.51E-11	5.59E-11
RBC	1.62E-11	4.06E-11	6.49E-11	1.02E-11	2.56E-11	4.10E-11
SBR	2.11E-11	5.22E-11	8.27E-11	1.28E-11	3.17E-11	5.03E-11
TF	2.07E-11	4.96E-11	7.82E-11	1.26E-11	3.01E-11	4.74E-11

Table 7-12: Global warming DL band 1, scenarios 19 – 24

PE	500	1250	2000	500	1250	2000
CMAS	2.63E-11	6.52E-11	1.04E-10	1.57E-11	3.90E-11	6.20E-11
CW	6.44E-12	1.61E-11	2.58E-11	6.08E-12	1.52E-11	2.43E-11
IFAS	2.09E-11	5.34E-11	8.49E-11	1.38E-11	3.56E-11	5.70E-11
RBC	2.05E-11	5.13E-11	8.21E-11	1.24E-11	3.10E-11	4.97E-11
SBR	2.27E-11	5.62E-11	8.90E-11	1.40E-11	3.48E-11	5.52E-11
TF	2.89E-11	6.97E-11	1.10E-10	1.62E-11	3.88E-11	6.12E-11

The systems' GWP profile is dominated by energy and chemical production, and direct emissions from secondary processes. Energy contributions to GWP ranged from 10 – 60%, chemicals from 5 – 47%, and direct emissions from 15 – 90%. The GWP contribution from direct emissions is a function of flowrate (estimated as 0.3 kg CO_2/m^3 influent) and, therefore, is the same for all systems. The direct emissions percentage contribution to GWP increased as the discharge limits became less stringent and the contribution from energy and chemical use gradually reduced. The direct emissions values ranged from an average of 20% in DL band 4 to over 60% in DL band 1. The largest variance in GWP output for all systems was as a result of changes in loading. The effect of variations in scale was negligible on a per capita basis. The relationship between GWP output and scale was linear; hence, no significant GWP economies of scale were observed (Table 7-13).

Table 7-13: Global warming DL band 4, scenarios 1 – 6. Presented in per capita values

	High			Low		
PE	500	1250	2000	500	1250	2000
AO	4.31E-14	4.30E-14	4.29E-14	2.86E-14	2.85E-14	2.83E-14
AAO	3.71E-14	3.70E-14	3.69E-14	2.54E-14	2.52E-14	2.51E-14
CMAS	5.25E-14	5.22E-14	5.18E-14	3.14E-14	3.12E-14	3.10E-14
CW	1.29E-14	1.29E-14	1.29E-14	1.22E-14	1.22E-14	1.22E-14
EA	5.06E-14	4.98E-14	4.94E-14	3.14E-14	3.09E-14	3.05E-14
IFAS	4.19E-14	4.28E-14	4.25E-14	2.76E-14	2.84E-14	2.85E-14
OD	5.70E-14	5.66E-14	5.66E-14	3.27E-14	3.23E-14	3.23E-14
RBC	4.11E-14	4.11E-14	4.11E-14	2.48E-14	2.48E-14	2.48E-14
SBR	4.53E-14	4.49E-14	4.45E-14	2.80E-14	2.78E-14	2.76E-14
TF	5.77E-14	5.58E-14	5.51E-14	3.24E-14	3.11E-14	3.06E-14

The attached growth systems generally performed better in lower DL bands. The RBC system had the lowest output in DL bands 1 and 2 as a result of the reduced growth media surface area and corresponding energy requirements. The variations in GWP output with changes in the sludge disposal option are minimal (Figure 7-24). However, the option to dewater the sludge in option 1 is seen to have a negative impact on GWP because of the energy and chemical inputs used in the process which produce a slightly higher GWP output than the other two options. In option 3 there is no energy input to the sludge treatment process and the chemical input is limited to the lime used for sludge stabilisation. Option 2 has the lowest GWP output; however, this result is subject to the boundary definitions used in the study. The LCI of the external contractor system is limited to transport and sludge emissions. Inputs to the treatment process used by the external contractor are unknown. It could be assumed that there are chemical inputs in the external contractor's sludge treatment process, which would have a negative effect on the GWP profile. Alternatively, stabilisation may occur through anaerobic digestion which has the potential to have a net positive effect (reduction) on the GWP output if the boundaries of the system were extended to include energy supplied back into the national electricity grid. Without compilation of a complete LCI of the external contractor sludge treatment process the GWP output is subject to significant uncertainty. The magnitude of the difference in GWP outputs between sludge disposal options was not affected by changes in DL band (Figure 7-25).

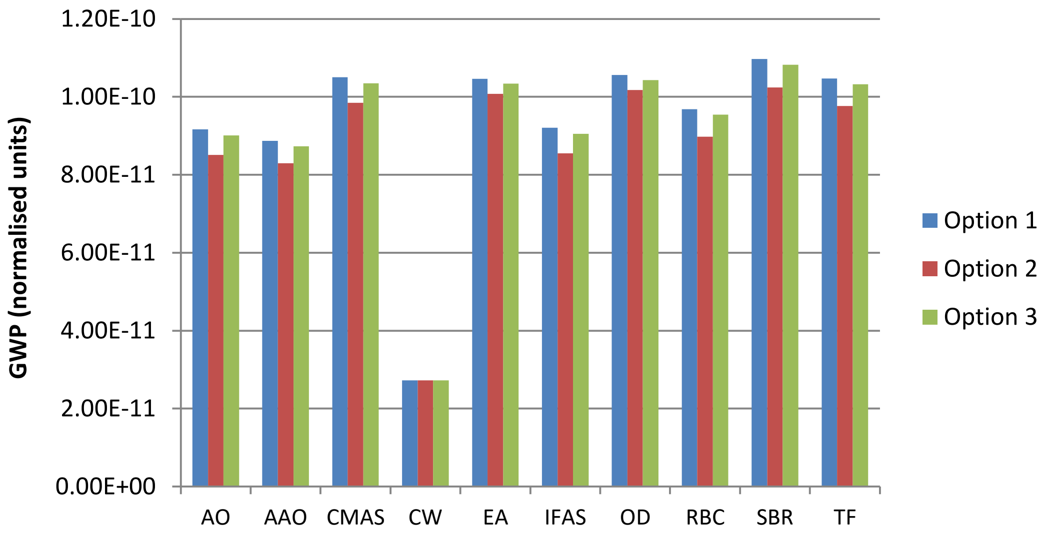


Figure 7-24: Variation in GWP with changes in sludge treatment and disposal (DL 4)

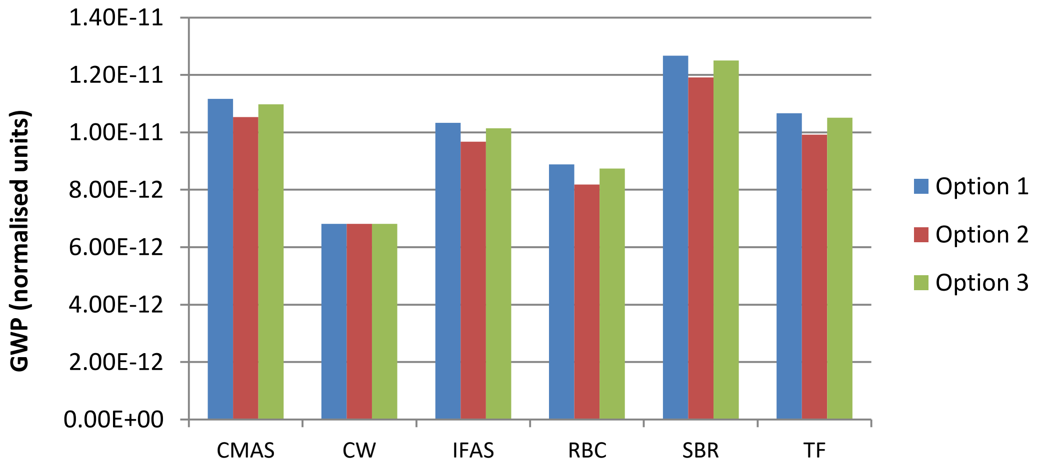


Figure 7-25: Variation in GWP with changes in sludge treatment and disposal (DL 1)

7.6.5.3 Acidification potential

Energy and chemical production are the primary processes responsible for acidification potential. Transportation of sludge and chemicals accounts for a small percentage of the total output, the greater percentage of which is attributed to the production of diesel. The AP category is particularly sensitive to variations in the quantities of chemicals used. The contribution from chemical production is significantly reduced from DL band 4 to DL band 1 (Figure 7-26). To illustrate, in DL band 4 the RBC chemical inventory includes: ferric chloride, sodium hydroxide, ethanol, calcium hydroxide, and polymers (acrylic acid) for sludge dewatering (Figure 7-27). This list is reduced to those chemicals needed for sludge stabilisation and dewatering in DL band 1 (Figure 7-28). Therefore, as the discharge limits become less stringent, the dominant process responsible for AP shifts from chemical to energy production.

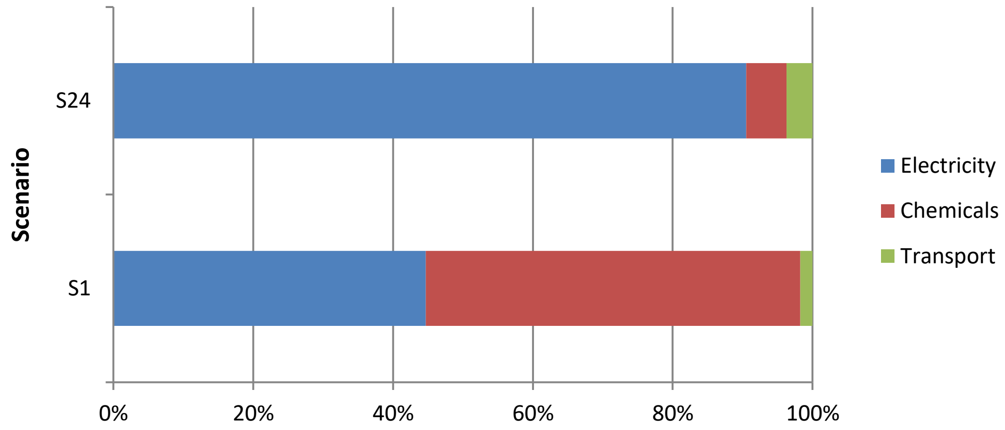


Figure 7-26: Variation in RBC AP distribution profile with changes in discharge limits

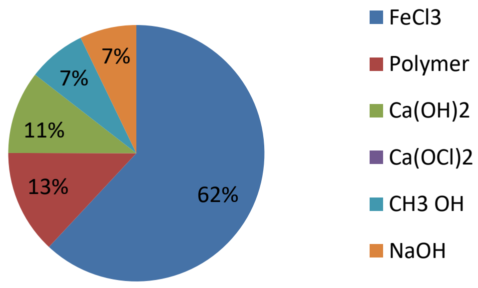


Figure 7-27: Scenario 1 RBC chemical consumption distribution

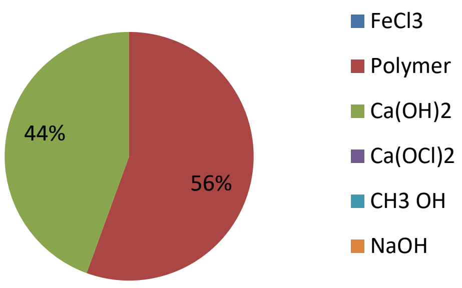


Figure 7-28: Scenario 24 RBC chemical consumption distribution

The AAO and AO systems benefit from their reduced chemical demand when loading and limits are at their highest and lowest respectively (Figure 7-29). The production of 1 kg of ferric chloride results in 4.29 x 10^{-3} kg SO_2 equivalent emissions. This reduces the AAO system AP marginally below the AO system acidification potential. The most significant reduction in AP comes from the reduction of sodium hydroxide. Although sodium hydroxide production AP is less than 60% (2.55 x 10^{-3} kg SO_2 equivalent) of ferric chloride production AP, the alkalinity recovery from denitrification reduces sodium hydroxide requirements by almost half. It can be seen that when the primary contributor to AP switches from chemicals to energy in DL band 1 (Figure 7-30) the attached growth systems perform more favourably than the suspended growth systems. The high AP output exhibited by the SBR system may be as a result of the higher organic load going into the aeration process due to the lack of a primary clarifier.

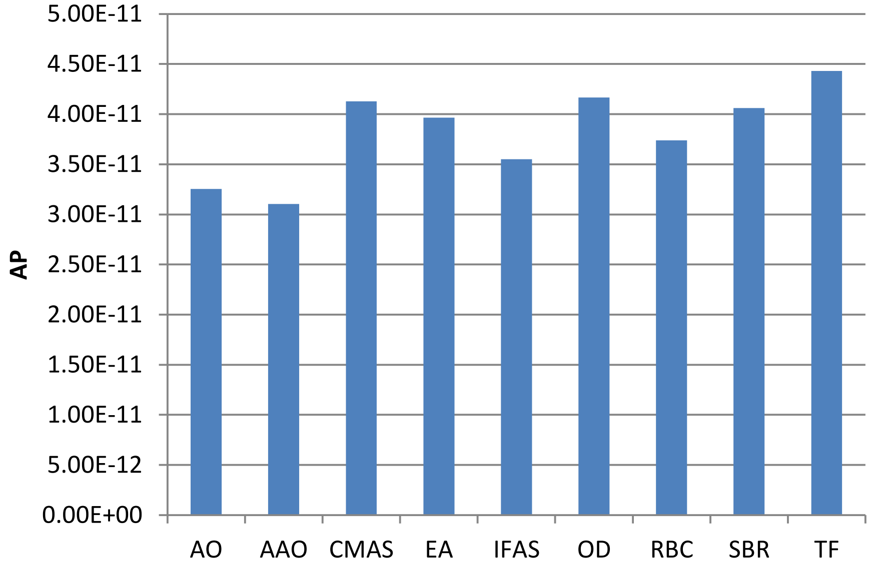


Figure 7-29: Acidification potential (S1)

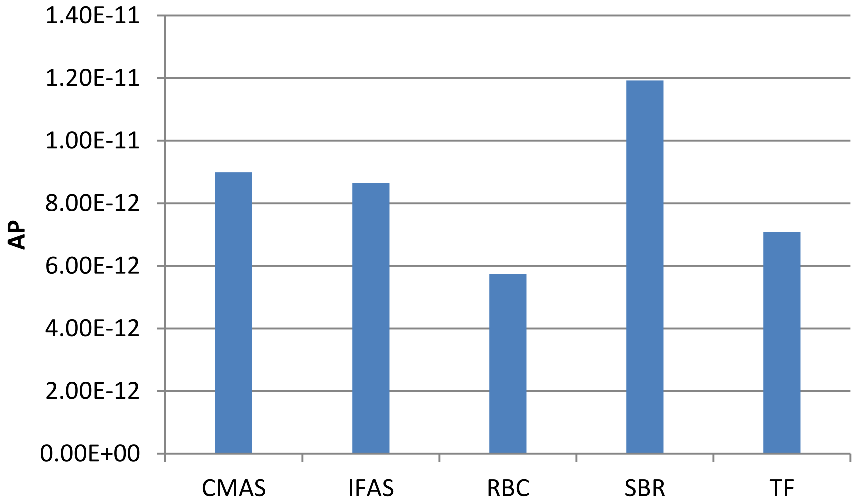


Figure 7-30: Acidification potential (S24)

Sludge disposal

The magnitude of AP is influenced by the sludge disposal option. The contribution to AP from sludge transport is higher for option 2 where the volume of untreated sludge being transported is greater (Figure 7-31). The difference in the magnitudes of AP between sludge disposal options remains largely consistent for attached growth systems as the discharge limits become less stringent. For suspended growth systems there is an increase in the difference between magnitudes as the SRTs are reduced and sludge volumes increase (Figure 7-32). The SBR system is not as sensitive to the reduction in SRT as are CAS based systems because of the higher DSC that is produced.

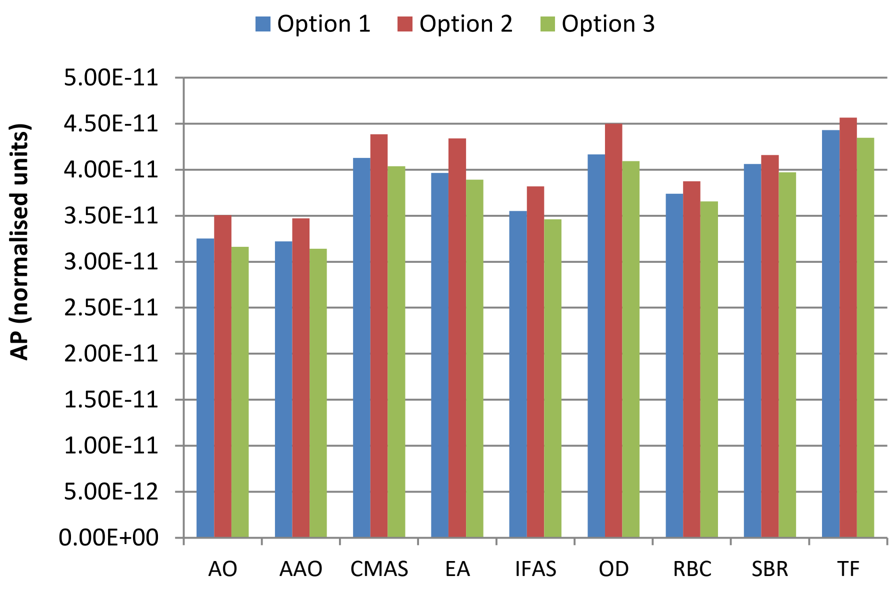


Figure 7-31: Variation in AP with changes in sludge treatment and disposal (DL 4)

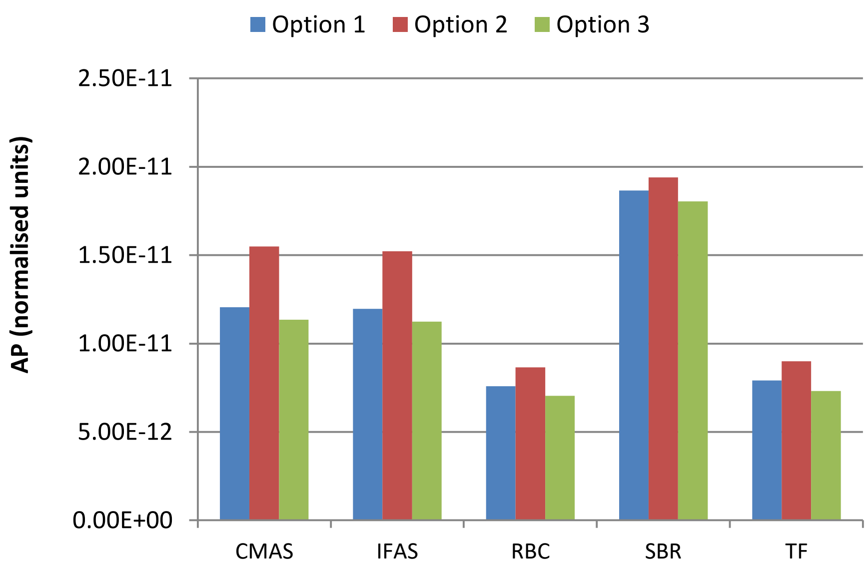


Figure 7-32: Variation in AP with changes in sludge treatment and disposal option (DL 1)

7.6.5.4 Eutrophication potential

The magnitude of EP is largely a function of a plant's discharge limits. The discharge limits define acceptable levels of eutrophication for a given final effluent receiving water body as determined by the environmental protection agency. Inland freshwater bodies are generally more sensitive to concentrations of eutrophying substances when compared with seawater bodies, and usually require WWTPs to reduce final effluent concentrations of nitrogen and phosphorus. However, the final effluent discharge is not the only source of eutrophication from a wastewater treatment plant. Wasted sludge contains concentrations of nitrogen, phosphorus and potassium that can provide valuable nutrient enrichment for agricultural soil. However, the application of sludge to agricultural soil also provides the potential for nutrient leaching into connected watercourses. Assuming that WWT is a steady state process from the perspective that there is no accumulation of nitrogen or phosphorus within the system, then it can be stated that the mass of TN and TP entering the system must leave the system in one form or another. The phosphorus entering the system must leave in either the final effluent or the sludge, and the nitrogen in either the final effluent, sludge, or through gaseous emissions from the denitrification process. Therefore, denitrification can mitigate some of the potential risk of eutrophication by reducing the quantities of nitrogen being emitted to either the terrestrial or aquatic environment.

In addition to the EP that results from a plant's direct emissions to the environment, there are also indirect emissions from upstream and downstream processes such as energy generation or chemical production that have the potential to cause eutrophication. The magnitudes of these emissions are generally small in comparison to a plant's direct emissions, and the potential for eutrophication from these processes usually occurs in other areas some distance away from the plant's location. Thus, the EP presented in the LCIA of WWTPs accounts for all eutrophying emissions in the entire life cycle of the WWTP and not just the direct emissions from the plant itself. In this study the nitrogen, phosphorus, and COD in the final effluent discharge accounted for 76 – 96% of the total EP output (Figure 7-33). The EP contribution from nitrogen and phosphorus in the sludge ranged from 4 – 20%. The combined contribution from energy and chemical production ranged from 0 – 4%. The most significant reduction in EP occurred from DL band 2 – 3 when the phosphorus limit was introduced. Even though the mass of phosphorus leaving the system is the same, the characterisation factor for emissions to soil is lower than that of emissions to the water body, and this is why the large reduction in the EP from the effluent resulted in only a small increase in the EP of the sludge. The phosphorus limit has a much greater effect on total EP reduction because of the difference between the phosphorus and nitrogen LCIA characterisation factors. The characterisation factors for phosphorus and nitrogen are 3.06 and 0.42 respectively [141]. Therefore, a 1 g reduction in phosphorus equates to a 7.2 g reduction in nitrogen. The reduction in EP from DL band 3 – 4 can be attributed to denitrification of nitrate to nitrogen gas. The small contribution from sludge application to land meant that there was no significant variation in EP with regard to changes in the sludge disposal option.

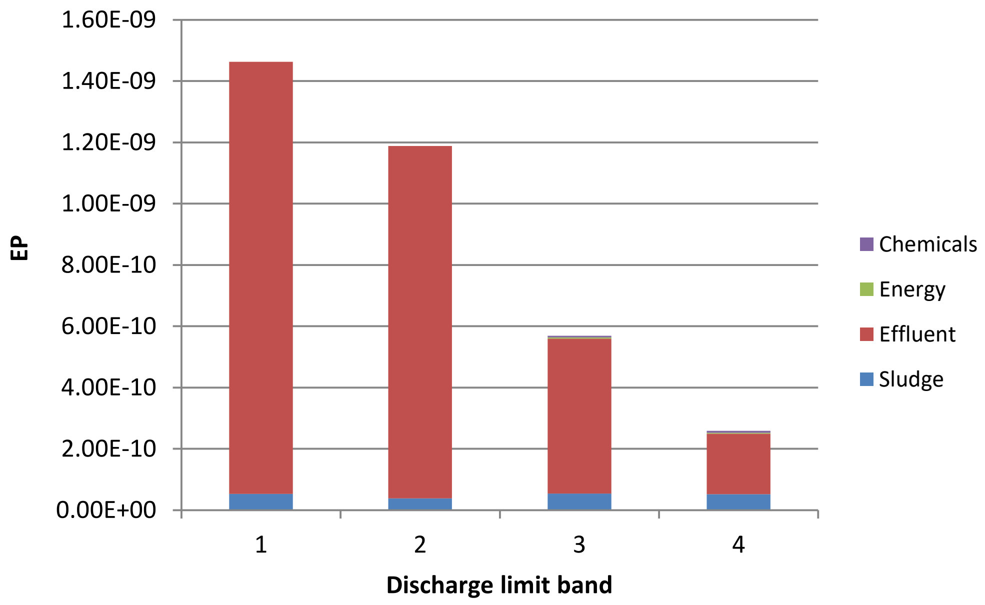


Figure 7-33: CMAS system EP for 2,000 PE, high loading, DL band 1 - 4

The simplification made for calculating EP outputs was that the applied discharge limits are said to represent the quantities of discharged pollutants. Therefore, the EP outputs will generally be the same for most systems apart from negligible differences where a system may have higher chemical or energy inputs. In reality, there will be variations of EP levels between systems because of safety factors adopted by plant operators, process monitoring, and automation. Small unmanned plants in particular may operate with larger safety factors so as to avoid any limits breach. The trade-off in these cases is higher operational costs in return for mitigation of non-compliance risk and subsequent financial penalties. In the preliminary LCA study, it was observed that the WWTP with the least stringent limits was producing final effluent with the lowest levels of biochemical oxygen demand. From consultation with the plant manager it was understood that from a cost-benefit perspective the additional cost of a higher level of treatment (longer SRTs) was deemed more favourable than the potential financial penalties for non-compliance.

7.6.5.5 Abiotic resource depletion potential

Abiotic resource depletion potential in WWT is a measure of a system's non-renewable global resource consumption. Reports in academic literature regarding ADP in WWTSs are sparse and references to resource consumption generally focus on energy use. In a review of 12 LCA-WWTP/S journal papers, only 7 made reference to ADP; of those, only 4 provided any discussion, and only 2 of those discussions involved reference to non-energy related resource depletion. This is because in most studies energy use is the primary cause of resource depletion, particularly in countries with a high percentage of fossil fuels in their electrical grid-mix. Therefore, although the site-specific conditions will affect ADP to some degree, the national electricity grid-mix may have a greater influence on the magnitude of resource depletion, and great care should be taken when making system comparisons on an international basis. Renou et al. [176] reported that energy generation, lime and ferric chloride production account on average for 95% of resource depletion. Hospido et al. [164] maintain that the ADP impact from the production of chemicals can be balanced out by including sludge as a fertiliser, and therefore, mitigating the impact of synthetic fertiliser production. However, there is some debate regarding the validity of this assertion. Renou et al. [176] claim that sludge cannot be applied to growing crops and therefore it cannot be assumed to have the same value as synthetic fertilisers. Moreover, some countries have moved towards prohibiting that application of sludge to farmland completely, or require a very high level of sludge treatment prior to application, which may not be economically feasible for some smaller systems. Pasqualino et al. [3] report that systems employing AD can reduce ADP through the use of biogas to replace fossil based energy sources. However, smaller systems do not produce adequate feedstock (sludge) to make AD economically feasible. The CML LCIA methodology used in this study includes two ADP impact categories: ADP elemental (ADPe), and fossil ADP (ADPf), and the interpretation of impact is dependent on the ADP category type being assessed.

7.6.5.5.1 Elemental based abiotic resource depletion

The ADPe impact is measured relative to the ultimate reserves of a substance and expressed in units of antimony equivalence (kg Sb_{eqv}./kg substance). It was found that in most scenarios, the magnitude of ADPe impact is a function of chemical use only. For systems that carry out denitrification in a pre-anoxic zone, or in a single stage tank with intermittent aeration, the alkalinity return from denitrification reduces the amount alkalinity to be replaced. Figure 7-34 presents the ADPe for a 2,000 PE EA system with high and low loading. When the system incurs low loading the difference in ADPe impact is negligible between DL bands 3 and 4. However, when the system incurs high loads there is a 23% drop in the magnitude of the impact when TN reduction is introduced. This suggests that based on elemental resource depletion, when organic loading is high, in addition to lowering eutrophication potential, denitrification can also reduce elemental abiotic resource depletion potential.

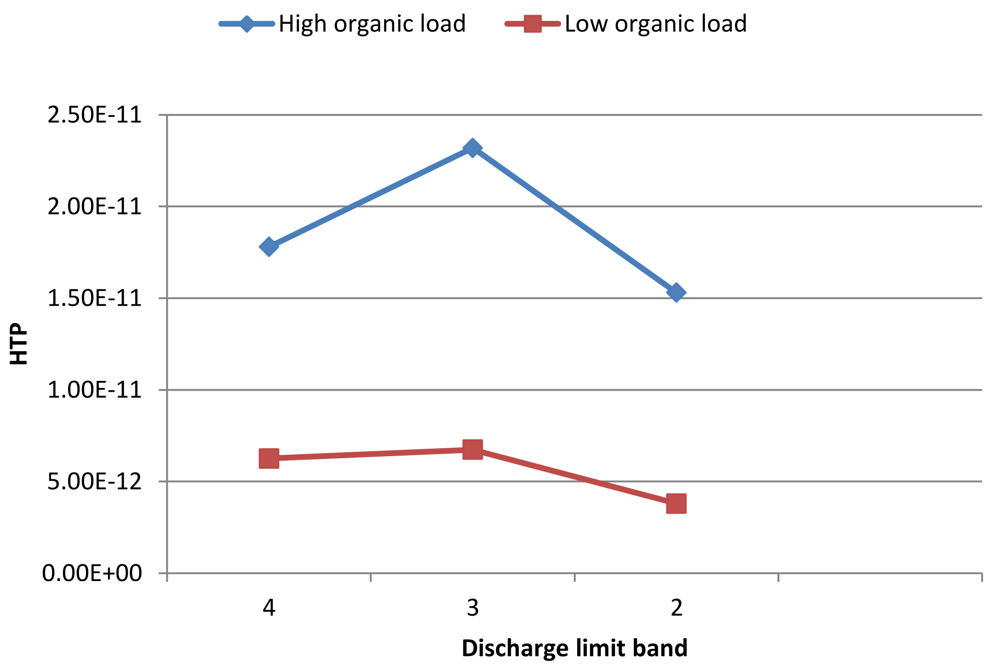


Figure 7-34: ADP (element) for the EA system for 2,000 PE, DL band 4 – 2

7.6.5.5.2 Fossil based abiotic resource depletion

The ADPf impact is based on the exergy content of a substance expressed in units of MJ/kg. The contributors to this category are energy and chemicals which, when averaged across all scenarios, accounted for 22-60% and 40-78% of the impact respectively. Contrary to ADPe, there was a 132% increase in the ADPf output from DL band 3 – 4 with high loading and a 34% increase with low loading (Figure 7-35). The difference in the outputs observed between the ADP categories relates to the exergy values of the included substances. In the previous section it was shown that the magnitude of ADPe was sensitive to variations of alkalinity replacement. However, the specific exergy value of the alkalinity replacement (NaOH used in this study) is 85.56 kJ/mol. The Irish electrical grid-mix is dominated by oil (2000+ kJ/mol) and natural gas (831.7 kJ/mol). Therefore, in the ADPf category the effect of reductions in alkalinity replacement from DL bands 3 – 4 is negligible when compared with the effect of an increase in the required energy consumption.

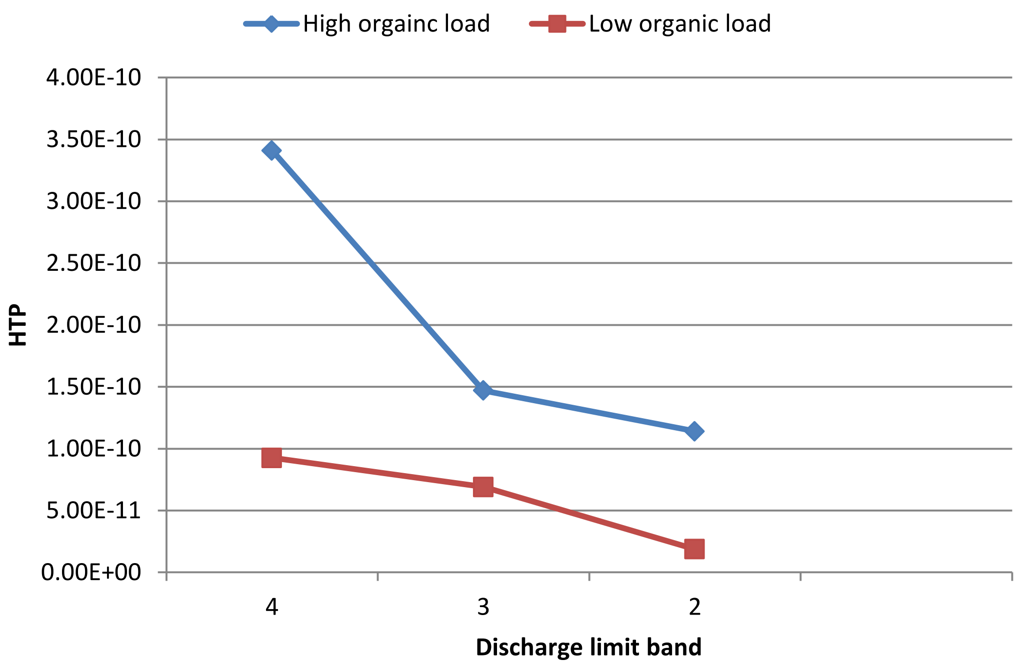


Figure 7-35: ADP (fossil) for the EA system for 2,000 PE, DL band 4 – 2

The value of the ADPf category as an indicator of resource depletion could be considered questionable, or more specifically, the method of measurement may not be appropriate. The use of the exergy value of a substance as an indicator for resource depletion may be misleading in this case. While exergy values can be used as an indication of resource use, it does not appropriately describe resource depletion because renewable forms of energy such as timber have an exergy value but their stocks are replaced. That is not to say, however, that there is no value in the information being provided. If the exergy depletion rates are to be used as an indicator for system performance, it should be as part of an overall system exergy balance and not considered in isolation.

The effect of sludge disposal variation in both ADP categories was negligible, with only small changes resulting from energy and chemical inputs used in dewatering. However, if the boundary definitions of the study were adjusted to include the production of fertiliser, the ADP impact would be more sensitive to the sludge disposal option because of the variation in the nutrient concentrations with respect to changes in sludge volume.

7.6.5.6 Human toxicity potential

As with the other toxicity impact categories in the CML LCIA methodology, HTP is measured relative to 1,4-dichlorobenzene (1,4-DCB). Huijbregts et al. [227] developed a toxicity potential calculation model and presented characterisation factors for 181 different substances. Of the three compartments (air, water, soil), it was determined that emissions to air and soil were the primary pathways for HTP and the associated characterisation factors were orders of magnitude greater than those of water. Among the substances presented in the study it was found that with the exception of PAHs and benzene, heavy metals had the largest characterisation factors (Table 7-14). In the current study aerial emissions of metals were limited to upstream and downstream processes, as heavy metal aerial emission data for on-site unit processes were unavailable. Moreover, no reference to aerial heavy metal emissions in WWT could be found in any of the reviewed WWT LCA literature, suggesting that any associated impact could be considered negligible. Therefore, the primary contributor to HTP was found to be the heavy metal concentrations in sludge being applied to agricultural soil (>90%), with the remaining 10% varying between energy and chemical production depending on the scenario in question. The HTP values produced by the DST ranged from 0.015 – 0.02 kg 1,4-DCB/m^3, and were lower than the 0.046 – 0.075 kg 1,4-DCB/m^3 range reported by Pradip et al. [160]. The source of the variation here is difficult to determine as the reported individual metal concentrations were aggregated into a single mass value.

Table 7-14: HTP characterisation factors and specific potentials of metal concentrations in Ireland [104]

Metals	*Concentrations (mg/kg DS)*	*HTP characterisation factors*			*Specific concentrations (soil) (1,4-DCB$_{equiv.}$/kg DS)*
		Soil	Air	Water	
Cadmium	2.8	2.0×10^4	3.5×10^5	23	56000
Chromium	165	8500	1.5×10^5	2.1	1402500
Copper	641	94	4300	1.3	60254
Mercury	0.6	5900	6000	1400	3540
Nickel	54	2700	3.5×10^4	330	145800
Lead	150	3300	470	12	495000
Zinc	562	64	100	0.58	35968
PAH	14.15	7.1×10^4	5.7×10^5	2.8×10^5	1004650

The concentration of heavy metals is estimated as a percentage of the sludge dry solids concentration. The metal concentrations used in the study are based on average values reported by [104]. However, details of the sludge condition were not included in the report. It is unknown if the sludge had been conditioned with lime or AD, if it was solely municipal sludge, or was there industrial contributions. These details are important because they affect the overall sludge DSC and consequently the concentration of heavy metals. The assumption made is that the concentration of metals is a function of the sludge DSC and does not vary with system. This may seem to be an over simplification; however, the purpose of including a toxicity based impact category is to demonstrate the effect that the choice of sludge disposal can have on the concentration of metals being deposited on farmland. The actual risk posed, or potential for toxicity is open to debate, and in most cases the metal concentration levels in Ireland are well below the required limits. The provision of toxicity categories simply provides an indication as to which sludge disposal option is likely to produce the greater level of toxicity potential. Further work is required to develop more system-specific effects. The most significant aspect of the discussion related to the toxicity categories dominated by sludge emissions is how the results are presented. When presenting the HTP on the basis of the percentage of the sludge DSC, options 3 could be considered the most favourable because there is an actual reduction in metal concentration with respect to sludge dry solid concentration. However, if the results are presented in terms of the metal concentration per volume of sludge, the outcome is very different. To illustrate this point the scenarios of 2,000 PE, with high loading in DL band 4 are presented for all three sludge disposal options. Figure 7-36 presents the HTP where the concentrations of metals are reported as a function of the sludge dry solids concentration. Excluding the CW system, there is little variation in the magnitude of impact between the electro-mechanical systems. The EA and OD systems have a slightly lower output because of reduced sludge quantities from the longer solids retention times. Option 3 has the lowest value as there is an actual reduction in the metals as they are deposited in the gravel layer of the sludge drying bed. Options 1 and 2 have similar values because there is no reduction in the metal concentration on a sludge DSC basis. However, when the HTP is presented with the metals as a function of sludge volume, which is how they will actually leave the treatment plant site, option 2 is seen as having the least potential for toxicity because of the effective concentration (Figure 7-37).

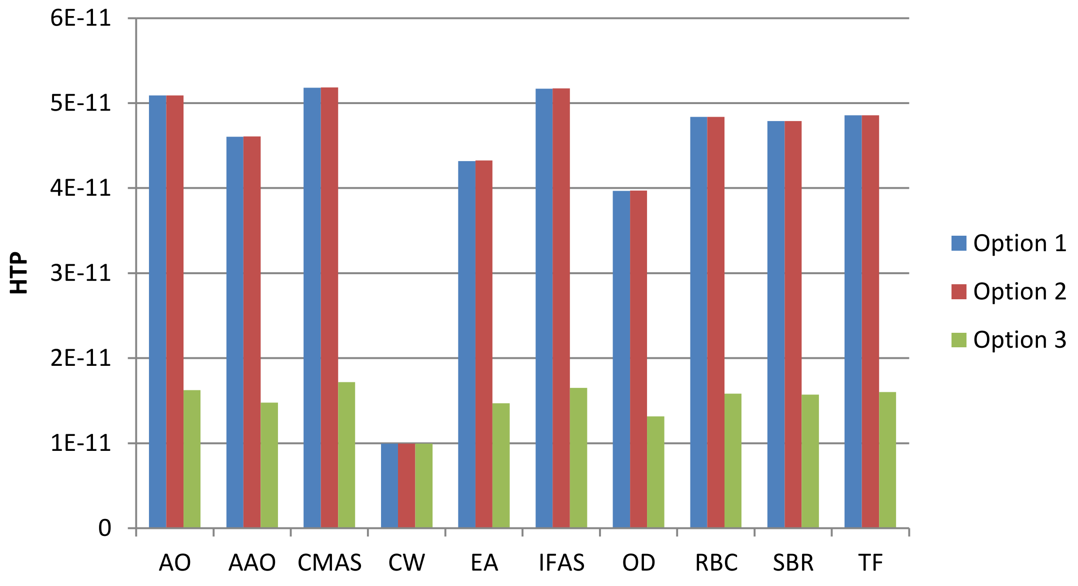


Figure 7-36: Human toxicity potential for 2,000 PE, DL band 4, with high loading, presented with metal concentration as a function of sludge dry solids concentration

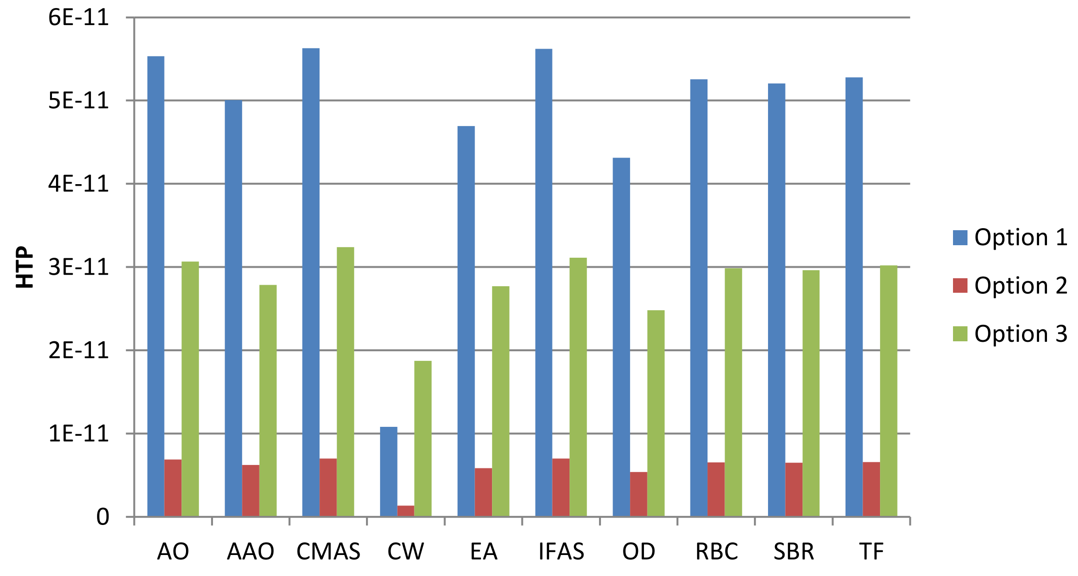


Figure 7-37: Human toxicity potential for 2,000 PE, DL band 4, with high loading, presented with metal concentration as a function of sludge volume

7.7 Further discussion

The findings from the systems analyses have demonstrated how the economic and environmental performance of each system can change with variations in site-specific conditions, method of sludge treatment and ultimate disposal. It is evident that as discharge limits became more stringent the economic cost of treatment increased with the increase in energy and chemical requirements. In addition, systems with the capacity to mitigate a percentage of the additional costs produced more favourable LCC estimations. This is an outcome that could have been predicted before any testing occurred. However, the extent to which the additional capital and operational investment would reduce the LCC was unknown. It was only through the application of LCCA that the net economic gain could be understood. It is difficult to come to any definitive conclusions as to the extent of any economic gains or losses because of the level of variability that exists in CAPEX estimations. In many scenarios the difference in LCCs between systems was small enough to be within a margin of uncertainty, which, without a more detailed, itemised CAPEX inventory, may reduce confidence that the most appropriate system is being selected. It was shown that the scale of a treatment plant will dictate the extent to which CAPEX influences life cycle costs. For small scales with low loading, the electro-mechanical systems' CAPEX accounted for over 50% of the total life cycle costs, which means that CAPEX estimations are more critical for small scale system selection. Conversely, for larger scale systems with high loading, the operation and maintenance costs can account for over 60% of the life cycle costs, which indicates that in these scenarios the focus for cost reduction shifts from capital to operational expenditure. This illustrates why it is important to consider CAPEX and OPEX together from a life cycle perspective. Only by considering these two cost elements together is it possible to gain a true understanding of the cost trade-offs that exist with respect to changes in site-specific conditions.

There are very clear trade-offs between environmental impact categories as a result of the variations in site-specific conditions. As discharge limits become more stringent the level of EP is reduced while the magnitude of potential impact in other categories increases. Global warming and acidification potential categories exhibit a significant increase in magnitude as discharge limits are reduced because of the contribution of energy and chemical production to these categories. Resource depletion categories follow a similar pattern; however, it has been shown that the method of measuring ADP can produce conflicting conclusions whereby the elemental ADP is reduced when moving from DL band 3 to 4, while fossil ADP increases. The rate of change of magnitude in both categories with respect to the DL band is much greater with a higher organic load. A more definitive method of ADP measurement or assessment is required to remove any ambiguity that may exist. The impact in toxicity categories is predominately a function of the heavy metal concentration of sludge. It was shown that HTP is sensitive to the method of sludge disposal because of the variations in the DSCs of sludge volumes that occur as a result of different methods of treatment.

The trade-offs between the economic costs and the environmental costs are not as clearly defined as the trade-offs between a system's CAPEX and OPEX, or between the impact categories in a system's environmental profile. The main objective of a WWTP is to reduce the quantity of pollutants in the final effluent discharge and reduce levels of potential eutrophication and aquatic toxicity. The economic cost increases as the quantity and range of pollutants required to be removed increases. However, the magnitudes of several other impact categories also increase with the increase in economic cost, and decrease in eutrophication potential. Therefore, if the discharge limits are considered to represent the acceptable level of EP for a given scenario, then the focus of system assessment should be on the economic and environmental cost elements that have common cost reduction potentials. It can be seen that a reduction in energy and chemical use will reduce both economic cost and environmental impact.

The effect of sludge treatment option was largely independent of system type. It had been postulated that reductions in sludge volumes from systems with long SRTs would have a positive effect on the life cycle costs. For systems with primary clarifiers the greater percentage of solids are removed at a higher DSC than the solids removed through wasted sludge. Conversely, the greater percentage of solids is removed at a lower DSC with the EA system because less than half as many solids are removed in pre-screening as are in primary sedimentation. Therefore, although the mass of dry solids has been reduced, the wet sludge volume is actually higher when compared with other suspended growth systems. Consequently, in Option 2 when the volume of sludge leaving site is at a maximum because no dewatering has taken place, the EA system has a higher sludge disposal cost than other suspended growth systems with primary clarifiers. In options 1 and 3, as the volume of sludge is reduced through dewatering, the lower solids inventory in the EA system results in lower sludge volumes.

The method of sludge disposal produces economic and environmental cost conflicts. It was shown that for most scenarios the investment in sludge dewatering equipment resulted in a net reduction in the LCCs of most systems because of the high specific per unit volume cost of removing sludge from site. The reduction in volume has a positive effect in reducing transport emissions and impact from diesel production. However, the additional energy and chemicals required for the process has a negative effect on a system's environmental profile. Furthermore, the reduction in sludge volume results in a higher concentration of heavy metals in the sludge per unit volume.

The environmental impact of the construction phase of a system's life cycle was considered to be negligible when compared with the use phase. However, it has been shown that at small scales the economic cost associated with the construction phase is the dominant element of the life cycle cost profile. It is, therefore, reasonable to assume that the same rationale could be applied to the environmental life cycle. Without the LCIs for the construction phase of the systems' life cycles, it is difficult to determine if the same trade-offs exist between the use-phase and construction phase in the environmental life cycle that were observed with the CAPEX and OPEX in the economic life cycle. Should the data become available, this topic is worthy of future work.

Remote monitoring and control systems have the potential to reduce labour costs at small plants. The economic benefits would have to be assessed on a site specific basis as locational factors may have a large negative or positive influence on a cost assessment. There are additional environmental benefits associated with reducing transport emissions, and improving plant efficiency will reduce resource consumption. However, it may take some time to develop robust, cost effective control systems, and while the cost of remote monitoring is higher than the cost of site visits and manual control, it could be anticipated that the latter practice may continue. One further point of note relates to system values that were not included in either type of analysis.

In many scenarios, the attached growth systems were found to perform economically and environmentally better than suspended growth system. This occurred most frequently in less stringent DL bands. There may be different outcomes in other locations depending on CAPEX variation and other specific cost variability, but if it is assumed that the results here are an accurate representation of the economic performance of a system, then it would be prudent to consider some qualitative system values. Attached growth systems exhibit good energy efficiencies with steady state flows, they have minimal levels of control, and therefore, limited human interaction. Their removal efficiencies are good, but for low nutrient limits they require a much greater specific surface area to accommodate low organic loading rates. This may necessitate an additional 4 stages to an RBC train (assuming 7 stages required for $NH_3 < 1$ mg/l), or triple the TF biotower volume. This means that there is a greater initial investment in growth material costs to move from BOD removal only, to BOD and ammonia removal. Furthermore, the cost of material replacement will add to the operational cost over the lifetime of the systems. Moreover, the environmental costs associated with the production of the growth material it is yet to be determined. In many cases attached growth systems would employ a tertiary process to achieve low levels of effluent nutrient concentrations. Rotating biological contactors have been used to good effect with a reed bed for tertiary polishing [46]. Trickling filters are used both pre and post aerobic tank to achieve high levels of nutrient removal [39]. Phosphorus removal with attached growth systems has had limited success. Removal rates of 70% in RBC systems have been reported by Hassard et al. [228], but their research also reported difficulties with regard to controlling oxic and anaerobic conditions. Chemical addition is generally required to achieve phosphorus removal in stand-alone RBC and TF systems. In short, while attached growth systems exhibit good economic and environmental performances in many scenarios, they are limited in versatility and may prove more difficult or costly to adapt to changes in scale or discharge limits than would suspended growth systems. As a potential go-between, the hybrid IFAS system exhibited good economic and environmental performance throughout the analyses. This system provides the stability of attached growth systems and the versatility of suspended growth systems which makes it robust and adaptable to changing conditions.

The systems analyses have provided insight into how systems perform economically and environmentally in various scenarios. However, as discussed, there may be some benefit in considering qualitative values associated with a given system. A systems ability to adapt to changing conditions may be an important asset in developing locations, and may have significant cost reducing potential. The level of human interaction required to operate a system may be an important asset in more rural locations. However, these are also location sensitive values that are subject to opinion. Their inclusion in the cost models would break the line of numerical traceability and reduce the value of the results being presented. It is, therefore, recommended that some form of qualitative evaluation should be included in system selection, but only after LCCA and LCIA has been completed.

8 Conclusions

The primary objective of this research was to provide a methodology and framework to evaluate the economic and environmental costs of small wastewater treatment systems from a life cycle perspective. It was postulated that variations in site-specific conditions would influence the economic and environmental performance of systems in different ways, and that each system should be evaluated under these conditions in order to assess their performance, and ultimately their suitability for implementation. The research was carried out in two stages. A preliminary LCA study was conducted first to assess the energy and resource efficiency of WWTPs in Ireland. It was postulated that during efforts to reduce eutrophication potential, the impact potential in other impact categories is often increased; thereby, reallocating environmental impact both regionally and globally. The main findings from the preliminary study were that

- the primary energy sinks in the WWTPs were the aeration blowers used in secondary treatment and the pumps;
- energy consumption is a central contributor to the environmental profile of the studied plants, in that it contributes to 8/11 LCIA categories in varying degrees. The impact categories dominated by energy consumption were generally of a more global nature (GWP, AP, ADPf, MAETP);
- the potentials of the impact categories dominated by energy consumption are heavily influenced by the national electrical grid-mix. Plants operating in countries with high levels of fossil fuels in the electrical grid-mix may exhibit a higher GWP than a plant with similar energy consumption rates operating in a country with a greener electrical grid-mix, and care should be taken when making comparisons internationally;
- the organic loading rate had the largest influence on energy consumption rates because it is directly related to the oxygen demand and subsequently the required aeration power;
- the effect of variation in discharge limits was most evident between the BOD removal only plant and the plants required to remove nutrients. The primary source of EP at the

BOD removal only plant was the final effluent discharge, while the sludge application to land was the primary source of EP at the freshwater plants. The use of ferric chloride to remove phosphorus at the freshwater plants also increased their acidification, global warming, and resource depletion potentials;

- the effect of variation in scale was inconclusive. While there was some evidence of environmental economies of scale between the largest plant and the others, the largest plant also had the least stringent discharge limits, which makes it difficult to identify the exact source of any economy. On reflection, it would have been more prudent to have included another coastal plant with similar site-specific conditions at a smaller scale;
- life cycle assessment was found to be a suitable tool for WWTP environmental assessment. The holistic nature of the methodology accounts for many of the upstream and downstream processes not included in other assessment methods. In many cases the upstream and downstream processes have proven to be the largest contributor to certain impact categories e.g. the contribution of energy to GWP.

Following the preliminary LCA stage, it was postulated that variations in site-specific conditions would influence a system's performance to the extent that it would affect a system's suitability for selection, and that a methodology that accounts for variations in site-specific conditions could more accurately predict a system's environmental and economic performance, and thus, its suitability for implementation. Furthermore, it was stated that the economic and environmental costs should be evaluated over a system's lifetime in order to understand the true cost of system ownership. Life cycle cost analysis and LCA were determined as being the most appropriate economic and environmental assessment tools respectively for system evaluation. The tools were then combined into a methodological framework and integrated into a decision support tool (DST) designed to assess the performance of WWTSs serving small agglomerations.

The DST provides a platform to assess the performance of a selection of WWTSs under a set of user- defined site-specific conditions. Economic and environmental costs are presented together with the aim of providing a more holistic overview of system performance, but without aggregation of weighted indicators into a single result or score. This allows the user to identify any economic or environmental trade-offs that may exist. The DST provides a detailed breakdown of several types of cost distribution associated with a given system in a given scenario, and facilitates comparisons of systems' CAPEX, OPEX, LCC, energy, footprint, and LCIA outputs. However, the program does have some limitations. The CAPEX estimations provided in the toolkit are based on data from countries outside Ireland and are prone to some level of uncertainty. Life cycle cost estimations provided by the DST would benefit from a more comprehensive, region specific, CAPEX estimation methodology. Furthermore, the cost of replacement parts can only be assessed with an itemised bill of quantities, warrantees, and details of a parts replacement regime. However, this approach involves a significant data collection exercise that may not produce a much higher level of accuracy to warrant such an undertaking.

Using the developed methodology and DST, system analyses were carried out in a range of predetermined scenarios to assess the methodology and the effect of variations in site-specific conditions. Systems analyses determined that for the scenarios considered in this study, CWs are the most cost effective system in terms of capital investment, operational expenditure, and from an environmental perspective CWs produce the least amount of harmful emissions, and require minimal resources when compared with electro-mechanical systems. The main constraint associated with implementing CW systems is the large surface area requirements. It was observed that at larger scales (4,500 – 5,000 PE) the electro-mechanical systems may become more economically competitive. For electro-mechanical systems, the general observation was that attached growth systems performed better at small scales, low loading, and high discharge limits. Suspended growth systems performed better at large scales, high loading and low discharge limits.

The most influential site-specific parameter for suspended growth systems in terms of operational cost was the organic loading rate. The OLR is a direct measure of the oxygen required for substrate oxidation, and is therefore, directly proportional to energy consumption. Additionally, higher OLRs produce greater sludge volumes; require more chemicals; thus, increased operational costs. In attached growth systems the OLR has a greater influence on a system's CAPEX because it determines the required growth media surface area. Similarly, CW systems are sized based on the OLR and discharge limits, which relates directly to their initial capital expenditure.

The discharge limits were shown to have a greater influence over system selection. It was found that some systems or system configurations were more suited to removing ammonia and nitrogen compounds, while other systems required significant additional capital and operational expenditure. Only one system (AAO) had the capacity to remove phosphorus biologically which proved to be beneficial in scenarios with low P limits; however, all systems still required some chemical input for P precipitation. In the least stringent discharge limit scenarios, systems with high nutrient removal capacity were surplus to requirement.

The most significant effect of variation in scale occurred between capital investment for electro-mechanical and CW systems. Electro-mechanical systems exhibit large scale economies from 500 to 5,000 PE after which, the cost per PE reaches a more steady state. Constructed wetlands capital expenditure has an almost linear relationship with PE, and therefore, assuming reasonable land prices, for very small systems CW capital expenditure is much lower than that of electro-mechanical systems, while at agglomeration sizes from 4,000 – 5,000 PE the costs are more aligned.

The most appropriate sludge disposal option must be determined on a case by case basis. It has been demonstrated that each treatment system will produce varying volumes of sludge in different conditions, and that these variations may be large enough to warrant the selection of alternative sludge treatment and disposal methods. However, the economic cost associated with each method is sensitive to location. The specific disposal cost values used in this study are nominal and subject to regional variation. External contractor costs may vary depending on distance to plant, sludge volume, concentration, or level of treatment prior to removal from site.

From an environmental perspective the sludge disposal solution could be viewed as a series of trade-offs between resource-use and waste emissions. Application of sludge to farmland is a pathway to return nitrogen and phosphorus back into the ecosystem. The trade-off here is that in addition to the nutrients, potentially toxic metal and phenol concentrations are also being introduced to the soil. The reduction in sludge volume can have both positive and negative environmental effects. Reducing sludge volume will reduce resource use and transport emissions, but may also increase sludge metal concentrations. Furthermore, depending on the method of dewatering, there may also be additional environmental costs associated with energy and chemical inputs.

The study highlighted the importance of considering CAPEX and OPEX together from a life cycle perspective. At small scales, the dominant component of the total LCC is the initial capital expenditure. As plant scales increase the OPEX becomes the more significant cost component. The exact point at which the balance shifts from CAPEX to OPEX depends on system and location. In certain scenarios, systems with high CAPEX and low OPEX had the more favourable life cycle cost. Conversely, in other scenarios, systems with low CAPEX but high OPEX had the more favourable life cycle cost. It is, therefore, conceivable that if system selection were based solely on CAPEX or OPEX alone, the most appropriate system may not be implemented.

Similar assertions can be made about the environmental aspects of the study. The LCA approach used in the study provides a numerically traceable method of quantifying the potential for environmental impact. The objective of wastewater treatment is to reduce the eutrophication and aquatic toxicity potential associated with final effluent discharge, but assessing environmental performance based on these two categories alone does not provide the system's full environmental profile. Only by applying the LCA methodology is it possible to gain an understanding of the environmental cost associated with the upstream and downstream processes.

However, it is conceivable that project commissioners may be more interested in determining the actual impact. The CML LCIA methodology has limited site-specific parameterisation which makes accurate prediction of actual impact difficult for more regionally sensitive impact categories such as toxicity, eutrophication and photochemical oxidation. The LCA component would benefit from the addition of more site-specific parameterisation that allows for user definition in areas such as soil composition, and substance background concentrations. This would help close the gap between the potential for impact, and the actual impact.

8.1 Thesis contributions

This thesis has made contributions to the wastewater treatment knowledge base in several areas:

This research has evaluated the environmental cost associated with wastewater treatment practices in Ireland. It has identified the areas in Irish WWT that contribute the most to environmental impact, and shown how and why the magnitude of impact varies with WWT in other international studies.

It has provided a unique methodology and framework to assess the economic and environmental performance of small wastewater treatment systems by accounting for variation in key site-specific parameters such as loading, discharge limits and sludge disposal option. Also, unlike other assessment methods, energy use estimations are based on first principles calculations and not on empirical data, which is a more numerically traceable method of energy use estimation.

It has provided a novel platform to assess and compare small WWTS performance under a variety of site-specific conditions in the form of a decision support tool by providing economic, environmental and energy data all in one software tool. It limits the amount of user interaction, and simplifies the assessment process making it more amenable to non-technical users.

It has demonstrated the importance of evaluating the economic and environmental cost of WWTSs from a life cycle perspective, and has elucidated the trade-offs that can exist between economic and environmental cost components.

It has demonstrated the influence that the choice of sludge treatment and disposal can have on economic and environmental cost. It has shown that even for very small systems, investment in sludge treatment technologies can have net-positive economic advantages due to reductions in sludge volumes, but depending on the type of treatment may have negative environmental consequences due to increased concentrations of heavy metals.

8.1.1 Policy implications

The main policy implications of this research relate to the WWTS procurement process. While it is understood that available capital expenditure constraints may restrict the options available to project commissioners, this study has demonstrated that system selections that are based on initial capital expenditure may prove more costly over the lifetime of the system when the annualised operational expenditure is accounted for in a lifecycle cost analysis. In essence, any project procurement that is based on capital expenditure alone may simply be *borrowing from the future*, which, depending on interest and discount rates, may reduce the cost effectiveness of the selected system. Therefore, it is recommended that all system procurement processes should include a full life cycle cost analysis.

The environmental impact associated with a given system is generally a secondary consideration in the procurement process. Most of the attention given to the environmental impact of WWT relates to protecting the water bodies that receive a plant's final effluent discharge, and managing the disposal of sludge that is produced. Both of these relate to the immediate regional impact of WWT, which is generally the main concern of the community being served. However, the introduction of a carbon tax creates a direct link between a plant's financial cost and its GHG emissions. Other environmental assessment tools such as EIA or ERA do not account for global emissions, and therefore, do not provide a complete environmental assessment. It is recommended that the procurement process should include an LCA to not only provide a full environmental profile, but also to contribute to a complete economic evaluation.

The final point on policy implication relates to the implementation of CW systems. The population spread in Ireland is such that 87% of agglomerations are below 2,000 PE. Lifecycle cost analysis and LCA determined that in every scenario (assuming land availability was not a constraint) from 500 to 2,000 PE the CW systems outperformed the electro-mechanical systems both economically and environmentally. Arguments against CW systems generally focus on the large initial capital investment in the land required, but the low operational costs will outweigh the initial capital over the life time of the system. Furthermore, because land is assumed not to lose value, CW systems have a much greater residual value at the end of their lifetime. Therefore, it is recommended that in locations where land availability is not a constraint, CW systems should be given due consideration subject to an economic assessment.

8.2 Further work

Much of the potential for further work relates to improving the accuracy of the decision support tool.

1. The first principle energy estimations did not exhibit the same economies of scale as reported in the literature, and were generally lower than empirical values. This is an area worthy of address because the quantity of energy used is so central to both the economic and environmental profiles of the treatment systems.
2. The LCC estimations would benefit from a more comprehensive, region-specific CAPEX estimation methodology. This would involve development of a system-specific, itemised, capital cost database. This data could then provide a more accurate assessment of the cost of parts replacement.
3. A more site-specific set of environmental parameters would enhance the value of the LCA component of the decision support tool.

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APPENDICES

Appendix A

Biological nitrogen removal

Biological nitrogen removal is the most common removal method currently in use. The process uses several species of bacteria in a series of reduction oxidation reactions that convert ammonia (NH_3)/ammonium (NH_4^+) to nitrogen gas (N_2) that dissipates into the atmosphere. It has lower operational costs than other physical or chemical processes. There are many different configurations for the conventional process, but the basic mechanism involves two stages: nitrification and denitrification.

Nitrification

During nitrification, autotrophic ammonium oxidizing bacteria (AOB) oxidize ammonium to nitrite (NO_2^-) and then nitrite oxidizing bacteria (NOB) carry out further oxidation, converting nitrite to nitrate (NO_3^-). The stoichiometric equations for both reactions are given below in Eq. A.1 and Eq. A.2. This two-stage oxidation process increases energy consumption due to the extra oxygen requirements. Based on stoichiometry calculations, 4.57 g of O_2 is required to oxidize 1 g of N.

$$\text{Ammonium} \rightarrow \text{Nitrite} \rightarrow \text{Nitrate}$$

$$2\ NH_4^+ + 3\ O_2 \rightarrow 2\ NO_2^- + 2\ H_2O + 2\ H^+ \tag{A.1}$$

$$2\ NO_2^- + O_2 \rightarrow 2\ NO_3^- \tag{A.2}$$

Denitrification

The denitrification stage takes place in anoxic[1] conditions and involves heterotrophic bacteria that require carbon and oxygen to multiply. In the absence of free oxygen (O_2), the bacteria can use the oxygen in a nitrate compound, reducing it to nitric oxide (NO), and then to nitrous oxide (N_2O) and finally to nitrogen gas (N_2). The stoichiometric equation is given below in Eq. A.3.

$$\text{Nitrate} \rightarrow \text{Nitric Oxide} \rightarrow \text{Nitrous Oxide} \rightarrow \text{Nitrogen gas}$$

$$2\ NO_3^- + 10\ e^- + 12\ H^+ \rightarrow N_2 + 6\ H_2O \tag{A.3}$$

[1] Anoxic = only bound oxygen available.

Appendix B.1

Table 1: Wastewater treatment plant characteristics

CHARACTERISTIC	WWTP A	WWTP B	WWTP C	WWTP D	WWTP E
REGISTRATION NUMBER	D0038-01	D0137-01	D0138-01	D0488-01	D0479 -01
TREATMENT TECHNOLOGY	Activated Sludge	Activated sludge with P removal	Activated sludge with P removal	Activated sludge with P removal	Activated sludge with P removal
INFLUENT CHARACTERISTICS	Municipal Wastewater only	Municipal Wastewater only	Municipal Wastewater only	Municipal Wastewater only	Municipal Wastewater only
TERTIARY TREATMENT	None	None	None	None	None
DESIGN CAPACITY (BOD)	186,000 p.e.	12,000 p.e.	1,2000 p.e.	820 p.e.	600 p.e.
ORGANIC LOADING	79,133 p.e. (2015)	12,284 p.e. (2014)	9,036 p.e. (2015)	590p.e. (2015)	1,024 p.e. (2015)
HYDRAULIC CAPACITY (DWF) (M^3/YEAR)	13,140,000	1,642,500	821,250	36,500	49,275
HYDRAULIC CAPACITY (PEAK FLOW) (M^3/YEAR)	39,420,000	4,927,500	2,463,750	109,500	147,825
HYDRAULIC LOADING (M^3/YEAR)	14,940,180	839,135	1,072,005	41,245	110,960
DISCHARGES INTO	Sea	River	River	River	River
TEST FREQUENCY	Monthly	Monthly	Monthly	Monthly	Bi-monthly

Table 2: Wastewater treatment plant characteristics

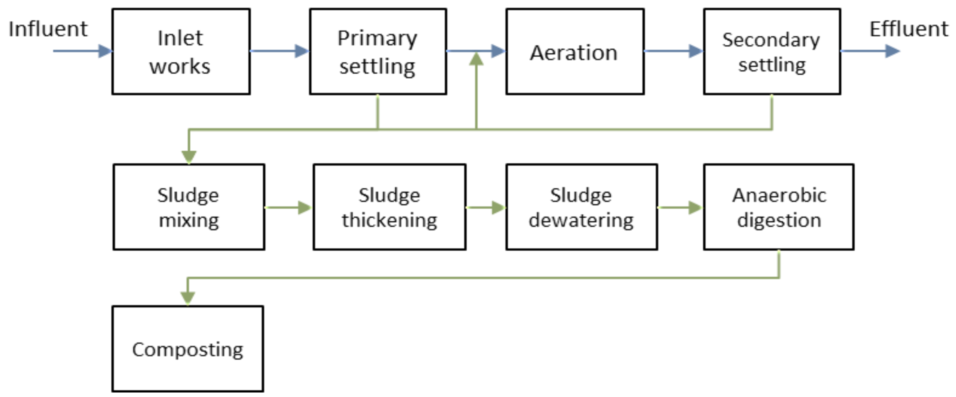
CHARACTERISTIC	WWTP A	WWTP B	WWTP C	WWTP D	WWTP E
DISCHARGE REQUIREMENTS					
pH	-	6 - 9	6 - 9	6 - 9	6 - 9
Temperature	-	-	-	-	-
CBOD	25mg/l	25mg/l	20mg/l	25mg/l	10mg/l
COD	125mg/l	125mg/l	125mg/l	125mg/l	50mg/l
Suspended solids	35mg/l	35mg/l	30mg/l	35mg/l	25mg/l
Total nitrogen (as N)	-	-	20mg/l	-	-
Total phosphorus (as P)	-	2 mg/l	1 mg/l	-	-
Ammonia (as n)	-	5mg/l	-	5mg/l	1mg/l
Orthophosphate (as p)	-	1 mg/l	-	2mg/l	0.5 mg/l
SLUDGE TREATMENT					
Yearly sludge output (kg - ds)	1,394,395	183,600	108,000	N/A	N/A
Sludge out per m^3 of influent (kg - ds)	0.09	0.22	0.10	N/A	N/A
Sludge treatment	Centrifugal dewatering and thickening, chemical stabilisation anaerobic digestion	Picket fence thickeners Centrifugal dewatering and thickening, chemical stabilisation	Picket fence thickeners Centrifugal dewatering and thickening, chemical stabilisation	None (Sent for external treatment)	None (Sent for external treatment)
Sludge disposal method	Composting	Land application	Land application	Land application	Land application

Table 3: Wastewater Treatment Plant Testing Methods

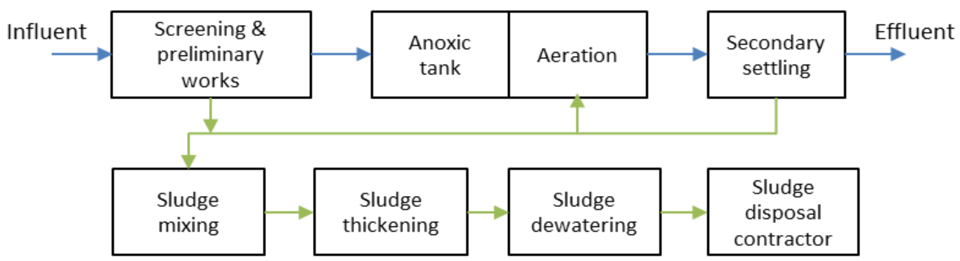
CHARACTERISTIC	WWTP A	WWTP E	WWTP F	WWTP H	WWTP J
Sampling dates	03 to 07, 10 to 14 17 to 21, 24 to 26 of Nov. (2013)	02/09/2014 to 07/09/2014	07, 08, 09, 14, 15, 16, 19 of October 2015	18, 19, 20, 24 of November 2015	06/11/2015 to 09/11/2015
Number of days	18 days	6 days	7 days	4 days	4 days
Flow streams sampled	Influent and Effluent	Influent and Effluent	Influent and Effluent	Influent	Influent
Number of samples per stream per day	As per plant managers schedule	6	6	6	6
Time between samples	N/A	4 hours	4 hours	4 hours	4 hours
Influent testing location	Influent Stream	Screening	Screening	Influent Stream	Influent Stream
Influent sampling method	Grab Sample (Automatic Sampler)	24 hour composite	24 hour composite	24 hour composite	24 hour composite
Effluent testing location	Outfall channel	Leaving Final Clarifier	Leaving Final Clarifier	Effluent Channel	Leaving Final Clarifier
Effluent sampling method	Grab Sample (Automatic Sampler)	24 hour composite	24 hour composite	24 hour composite	24 hour composite
Energy data	Yes	Yes	Yes	Yes	Yes
Data point frequency	Daily totals and process breakdown	30-60 seconds	30-60 seconds	30-60 seconds	30-60 seconds
Influent flow data	Yes	Yes	Yes	Yes	Yes
Frequency and type	Daily Total	Daily Total	Daily Total	Daily Total	Daily Total
Effluent flow data	Yes	Yes	Yes	No	No
Frequency	Daily Total	Daily Total	Daily Total	N/A	N/A

Appendix B.2

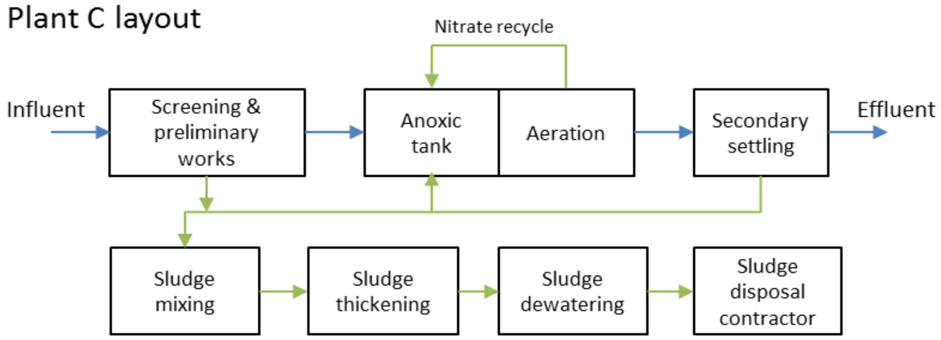
Plant A layout



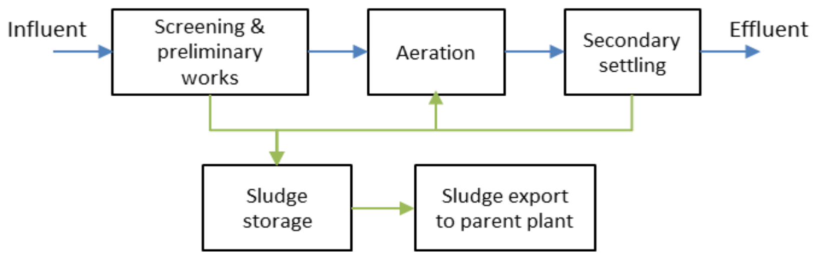
Plant B layout



Plant C layout



Plant D and E layout



Appendix B.3

Table 4: Wastewater Treatment Plant Testing Methods

CHARACTERISTIC	WWTP A	WWTP B	WWTP C	WWTP D	WWTP E
Sampling dates	03 to 07, 10 to 14 17 to 21, 24 to 26 of Nov. (2013)	02/09/2014 to 07/09/2014	07, 08, 09, 14, 15, 16, 19 of October 2015	18, 19, 20, 24 of November 2015	06/11/2015 to 09/11/2015
Number of days	18 days	6 days	7 days	4 days	4 days
Flow streams sampled	Influent and Effluent	Influent and Effluent	Influent and Effluent	Influent	Influent
Number of samples per stream per day	As per plant managers schedule	6	6	6	6
Time between samples	N/A	4 hours	4 hours	4 hours	4 hours
Influent testing location	Influent Stream	Screening	Screening	Influent Stream	Influent Stream
Influent sampling method	Grab Sample (Automatic Sampler)	24 hour composite	24 hour composite	24 hour composite	24 hour composite
Effluent testing location	Outfall channel	Leaving Final Clarifier	Leaving Final Clarifier	Effluent Channel	Leaving Final Clarifier
Effluent sampling method	Grab Sample (Automatic Sampler)	24 hour composite	24 hour composite	24 hour composite	24 hour composite
Energy data	Yes	Yes	Yes	Yes	Yes
Data point frequency	Daily totals and process breakdown	30-60 seconds	30-60 seconds	30-60 seconds	30-60 seconds
Influent flow data	Yes	Yes	Yes	Yes	Yes
Frequency and type	Daily Total	Daily Total	Daily Total	Daily Total	Daily Total
Effluent flow data	Yes	Yes	Yes	No	No
Frequency	Daily Total	Daily Total	Daily Total	N/A	N/A

Appendix B.4

Water quality analysis data - Plant A

Influent (m^3/day)	Effluent (m^3/day	BOD in (mg/L)	BOD out (mg/L)	BOD removed (mg/l)	COD in (mg/L)	COD out (mg/L)	COD removed (mg/L)	TSS in (mg/L)	TSS out (mg/L)	TSS removed (mg/L)
31530	29840	101.59	4.85	96.74	285.00	35.00	250.00	98.00	13.30	84.70
32570	30560	56.16	5.48	50.68	264.00	50.00	214.00	138.00	13.20	124.80
32190	30100	295.93	5.46	290.47	297.00	41.00	256.00	456.00	7.53	448.47
31020	29070	219.14	4.71	214.43	313.00	22.00	291.00	298.00	8.49	289.51
29450	27520	235.58	4.69	230.89	301.00	34.00	267.00	400.00	13.14	386.86
33690	31800	126.03	5.16	120.87	427.00	42.00	385.00	196.00	13.26	182.74
33600	32220	178.86	7.91	170.95	371.00	50.00	321.00	260.00	16.36	243.64
28780	26690	244.96	4.70	240.26	318.00	19.00	299.00	400.00	9.40	390.60
28250	26330	201.14	5.25	195.89	379.00	33.00	346.00	280.00	8.44	271.56
27270	25570	132.36	4.72	127.64	374.00	31.00	343.00	188.00	12.26	175.74
31870	29950	161.28	4.73	156.55	440.00	39.00	401.00	244.00	7.56	236.44
33470	31440	140.25	4.72	135.53	474.00	35.00	439.00	222.00	4.72	217.28
28680	26550	127.65	4.67	122.98	371.00	28.00	343.00	122.00	4.67	117.33
29240	27400	264.94	4.71	260.23	432.00	30.00	402.00	500.00	12.24	487.76
27340	25260	283.98	4.66	279.32	381.00	34.00	347.00	394.00	5.59	388.41
25970	23930	188.07	4.86	183.21	435.00	29.00	406.00	208.00	10.22	197.78
25200	23920	158.82	4.76	154.06	466.00	40.00	426.00	166.00	11.43	154.57
24670	24030	195.62	4.88	190.74	467.00	35.00	432.00	170.00	9.76	160.24
29710.6	27898.9	184.0	5.1	179.0	377.5	34.8	342.7	263.3	10.1	253.2

Plant B

Influent (m^3/day)	Effluent (m^3/day)	BOD in (mg/L)	BOD out (mg/L)	BOD removed (mg/l)	COD in (mg/L)	COD out (mg/L)	COD removed (mg/L)	TSS in (mg/L)	TSS out (mg/L)	TSS removed (mg/L)	TN in (mg/L)	TN out (mg/L)	TN removed (mg/L)	TP in (mg/L)	TP out (mg/L)	TP removed (mg/L)
1833	1739	235.0	6.0	229.0	384.0	80.0	304.0	532.0	6.4	98.8	63.7	53.3	10.4	5.6	0.1	5.5
1848	1606	211.1	16.1	195.0	480.0	138.7	341.3	164.0	34.8	78.8	63.8	59.7	4.1	6.4	1.0	5.3
1795	1641	183.3	9.4	173.9	421.3	32.0	389.3	300.0	10.0	96.7	77.8	38.0	39.8	8.3	0.7	7.6
1884	1742	223.9	7.1	216.8	455.0	144.0	311.0	348.0	17.0	95.1	67.6	48.2	19.4	8.1	1.2	6.9
1879	1739	193.5	8.0	185.5	390.0	128.0	262.0	136.0	20.0	85.3	84.3	51.1	33.2	9.9	1.9	8.1
1847	1709															
1848	1696	209.4	9.3	200.0	426.1	104.5	321.5	296.0	17.6	90.9	71.5	50.1	21.4	7.7	1.0	6.7

Plant C

Influent (m^3/d)	Effluent (m^3/d)	BOD in (mg/L)	BOD out (mg/L)	BOD removed (mg/l)	COD in (mg/L)	COD out (mg/L)	COD removed (mg/L)	TSS in (mg/L)	TSS out (mg/L)	TSS removed (mg/L)	TN in (mg/L)	TN out (mg/L)	TN removed (mg/L)	TP in (mg/L)	TP out (mg/L)	TP removed (mg/L)
2705	2803	113.1	12.7	100.4	288.0	53.3	234.7	172.0	26.0	84.9	33.2	21.2	12.0	5.0	1.7	3.3
2021	2071	90.0	6.5	83.5			0.0	110.0	13.2	88.0	40.0	18.4	21.6	4.5	0.7	3.7
1830	1855	105.0	8.1	96.9	245.3	85.3	160.0	193.3	19.2	90.1	26.9	19.5	7.3	3.0	0.4	2.6
2053	2010	126.6	12.9	113.7	149.3	53.3	96.0	178.0	14.5	91.8	29.8	21.0	8.8	3.6	0.8	2.8
1802	1742	49.4			256.0	48.0	208.0	170.0	19.2	88.7	25.1	0.0		3.7		
1730	1586	110.8	9.3	101.5	341.3	117.3	224.0	152.0	14.8	90.3	24.0	9.7	14.3	2.9	0.8	2.1
1719	1543				192.0	32.0	160.0	140.0	20.3	85.5	28.3	22.2	6.2	2.7	0.7	2.1
1980	1944	99.2	9.9	99.2	245.3	64.9	154.7	159.3	18.2	88.5	29.6	16.0	11.7	3.6	0.9	2.8

Plant D

Influent (m^3/d)	Effluent (m^3/d)	BOD in (mg/L)	BOD out (mg/L)	BOD removed (mg/l)	COD in (mg/L)	COD out (mg/L)	COD removed (mg/L)	TSS in (mg/L)	TSS out (mg/L)	TSS reduced (mg/L)	TN in (mg/L)	TN out (mg/L)	TN removed (mg/L)	TP in (mg/L)	TP out (mg/L)	TP removed (mg/L)
174	----	225.8	6.7	219.1	384.0	138.7	245.3	244.0	48.8	195.2	20.4	24.3		2.3	0.2	2.1
161	----	80.0	8.4	71.6	266.7	117.3	149.3	288.0	38.4	249.6	24.9	20.6	4.3	2.9	0.4	2.4
152	----	128.9	12.9	116.0	234.7	74.7	160.0	158.0	104.0	54.0	25.0	20.7	4.3	3.5	0.1	3.4
190	----	58.3	4.8	53.5	298.7	117.3	181.3	242.5	70.0	172.5	23.9	16.7	7.1	2.0	0.1	1.9
169		123.3	8.2	115.1	296.0	112.0	184.0	233.1	65.3	167.8	23.5	20.6	5.2	2.7	0.2	2.5

Plant E

Influent (m^3/d)	Effluent (m^3/d)	BOD in (mg/L)	BOD out (mg/L)	BOD removed (mg/l)	COD in (mg/L)	COD out (mg/L)	COD removed (mg/L)	TSS in (mg/L)	TSS out (mg/L)	TSS reduced (mg/L)	TN in (mg/L)	TN out (mg/L)	TN removed (mg/L)	TP in (mg/L)	TP out (mg/L)	TP removed (mg/L)
526	----	66.7	24.0	42.7	410.7	106.7	304.0	72.0	56.4	21.7	16.6	9.2	7.4	1.4	0.8	0.6
511	----	100.0	7.7	92.3	96.0	21.3	74.7	90.0	17.2	80.9	11.0	9.0	2.0	1.0	1.0	
782	----	263.3	8.7	254.6	160.0	21.3	138.7	55.0	56.0	0.0	6.8	7.2	0.0	1.0	0.6	0.4
532	----	106.7	16.0	90.7	117.3	21.3	96.0	352.0	148.0	58.0	20.1	9.3	10.8	4.0	1.3	2.6
588		134.2	14.1	120.1	196.0	42.7	153.3	142.3	69.4	40.1	13.6	8.7	5.1	1.8	0.9	1.2

Appendix B.5

Table 5: Average energy efficiencies with maximum and minimum values

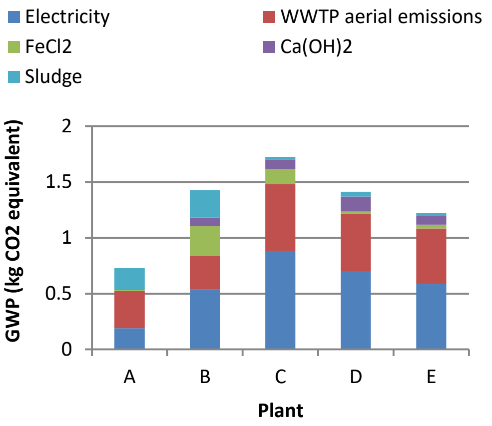
Plant	kWh/day	Max	Min	kWh/p.e. year	Max	Min	kWh/m^3	Max	Min
A	12524	15277	10953	23.36	33.19	18.41	0.48	0.69	0.38
B	1705	1780	1668	50.76	52.09	48.92	1.01	0.95	0.89
C	1451	1562	1387	41.11	49.45	28.13	0.75	0.90	0.51
D	115	119	104	37.39	43.05	32.75	0.68	0.79	0.60
E	230	234	213	22.09	25.00	16.40	0.60	1.04	0.30

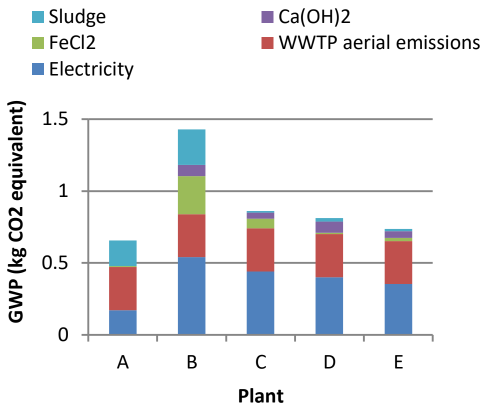
Plant	kWh/kg BOD removed	Max	Min	kWh/kg COD removed	Max	Min
A	2.79	6.64	1.20	1.28	1.82	0.86
B	4.68	5.47	4.00	2.93	3.41	2.44
C	7.30	8.90	5.12	4.60	7.44	2.19
D	7.79	11.27	2.73	3.92	4.98	2.44
E	5.21	10.05	1.18	3.53	6.12	1.41

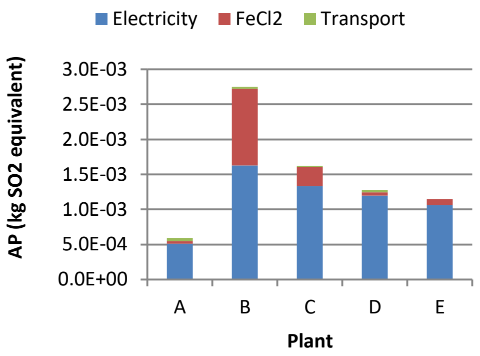
Plant	kWh/kg TSS removed	Max	Min	kWh/kg TN removed	Max	Min
A	2.16	5.37	0.78			
B	10.27	11.80	9.28			
C	8.48	10.01	6.05	76.67	137.04	32.47
D	6.03	14.56	2.98			
E	8.19	19.79	5.65			

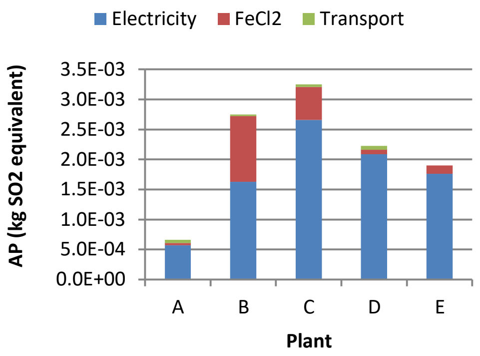
Appendix B.6

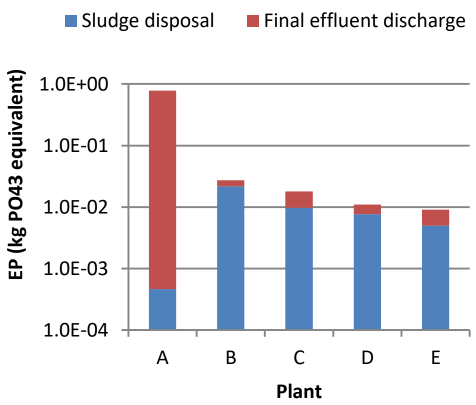
LCIA results – (hydraulic functional unit) (organic functional unit)

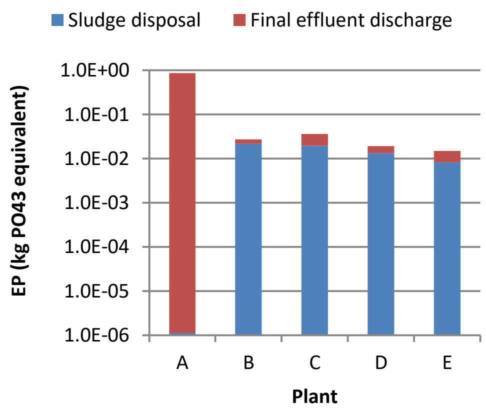


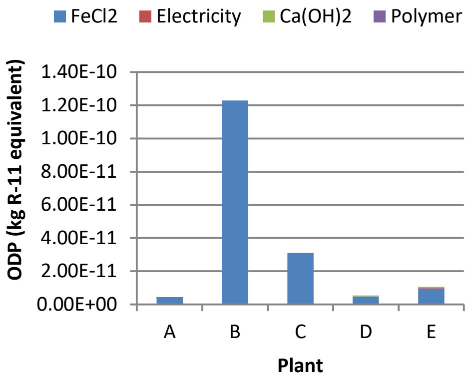








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ODP (kg R-11 equivalent)
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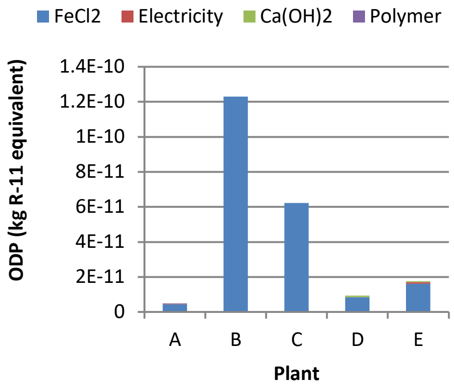
1.20E-10
1.00E-10
8.00E-11
6.00E-11
4.00E-11
2.00E-11
0.00E+00
A B C D E
Plant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ODP (kg R-11 equivalent)
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1.2E-10
1E-10
8E-11
6E-11
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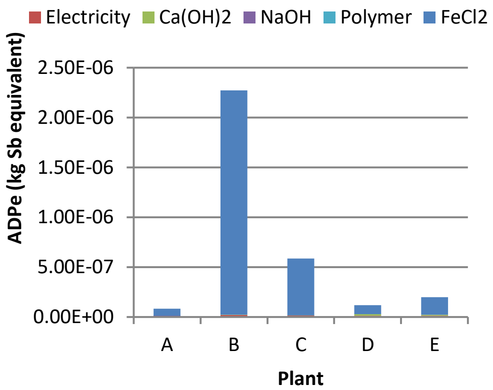
0
A B C D E
Plant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ADPe (kg Sb equivalent)
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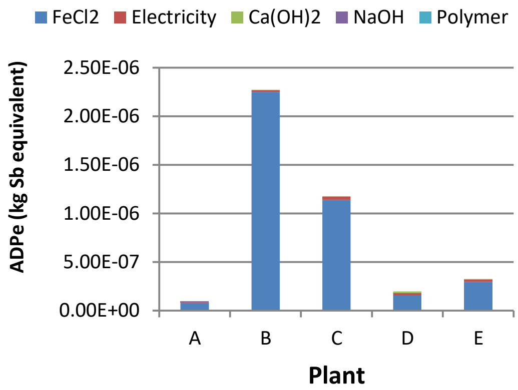
A B C D E
Plant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ADPe (kg Sb equivalent)
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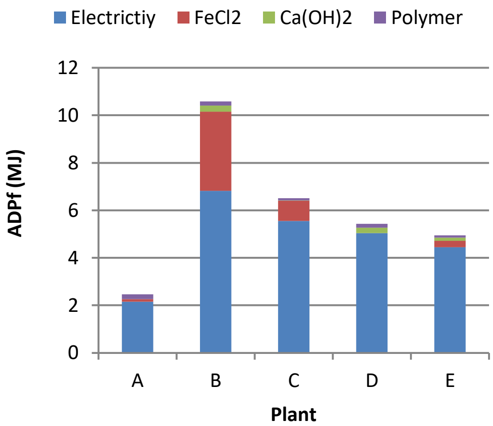
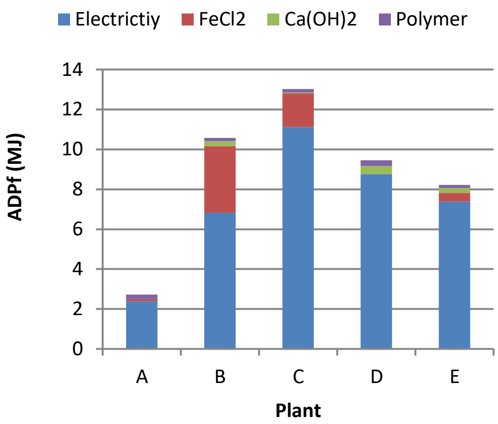
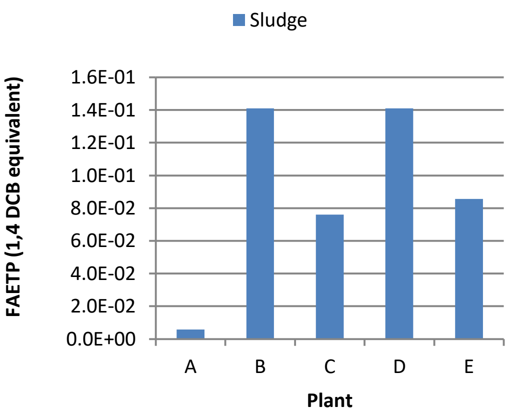
A B C D E
Plant


Electrictiy FeCl2 Ca(OH)2 Polymer
ADPf (MJ)
12
10
8
6
4
2
0
A B C D E
Plant


Electrictiy FeCl2 Ca(OH)2 Polymer
ADPf (MJ)
14
12
10
8
6
4
2
0
A B C D E
Plant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FAETP (1,4 DCB equivalent)
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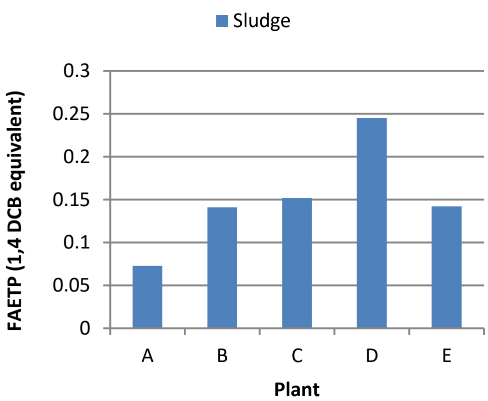
6.0E-02
4.0E-02
2.0E-02
0.0E+00
A B C D E
Plant


Sludge
FAETP (1,4 DCB equivalent)
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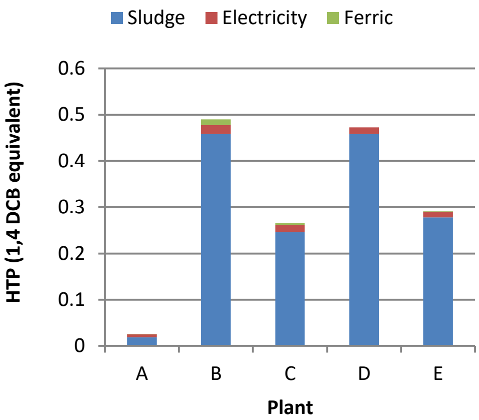
0.2
0.15
0.1
0.05
0
A B C D E
Plant


Sludge
Electricity
Ferric
HTP (1,4 DCB equivalent)
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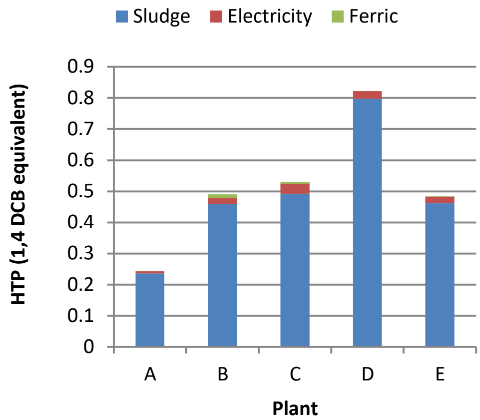
0.5
0.4
0.3
0.2
0.1
0
A
B
C
D
E
Plant


Sludge
Electricity
Ferric
HTP (1,4 DCB equivalent)
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0.8
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0.5
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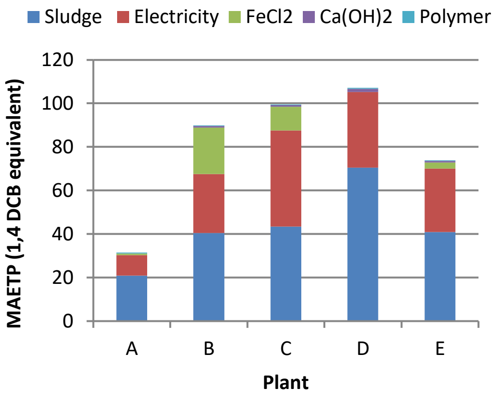
0.2
0.1
0
A
B
C
D
E
Plant


Sludge
Electricity
FeCl2
Ca(OH)2
Polymer
MAETP (1,4 DCB equivalent)
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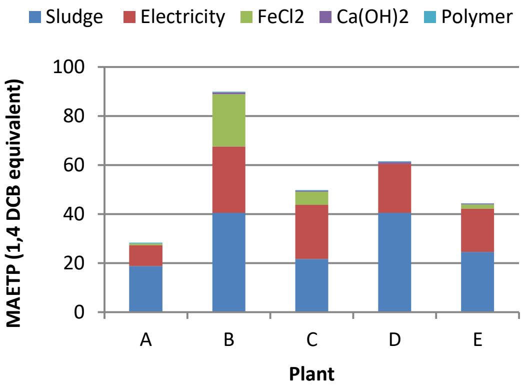
100
80
60
40
20
0
A
B
C
D
E
Plant


Sludge
Electricity
FeCl2
Ca(OH)2
Polymer
MAETP (1,4 DCB equivalent)
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80
60
40
20
0
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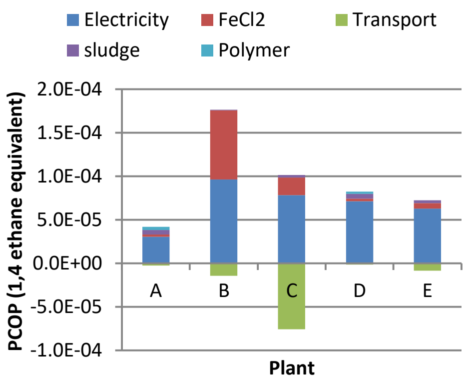
B
C
D
E
Plant


Electricity
FeCl2
Transport
sludge
Polymer
PCOP (1,4 ethane equivalent)
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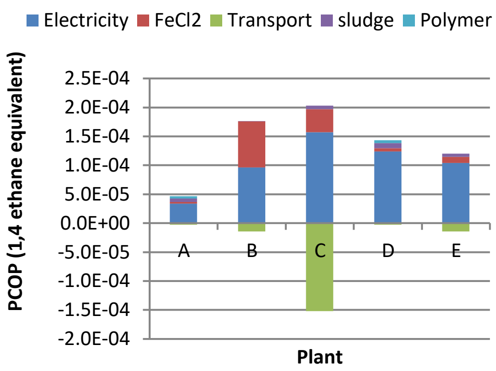
0.0E+00
-5.0E-05
-1.0E-04
A
B
C
D
E
Plant


Electricity
FeCl2
Transport
sludge
Polymer
PCOP (1,4 ethane equivalent)
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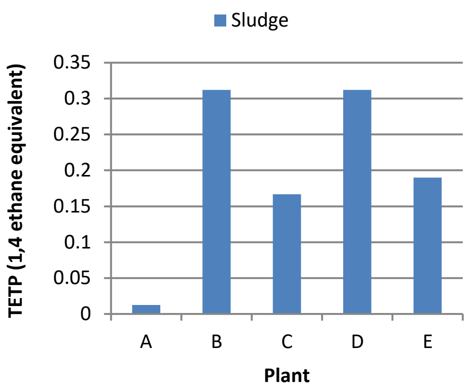
2.0E-04
1.5E-04
1.0E-04
5.0E-05
0.0E+00
-5.0E-05
-1.0E-04
-1.5E-04
-2.0E-04
A
B
C
D
E
Plant


Sludge
TETP (1,4 ethane equivalent)
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0.3
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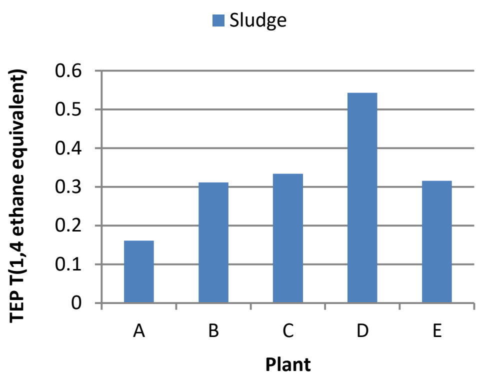
0.1
0.05
0
A
B
C
D
E
Plant


Sludge
TEP T(1,4 ethane equivalent)
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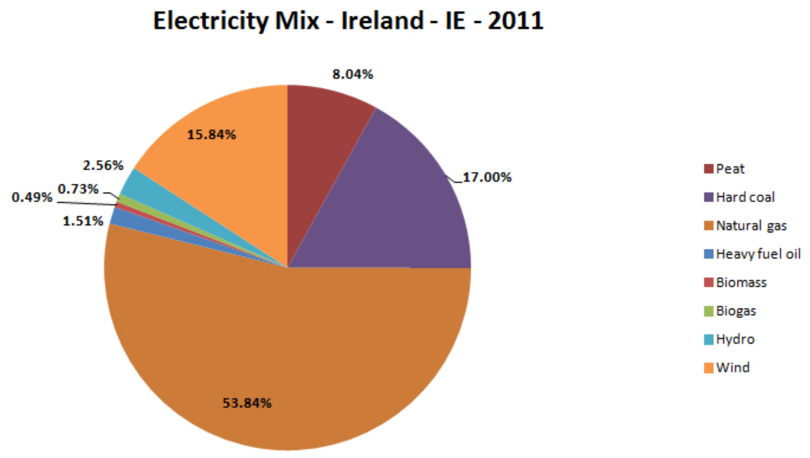
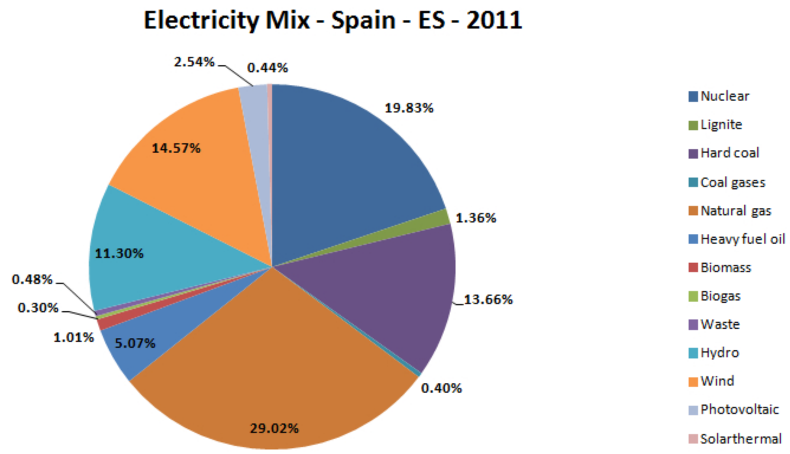
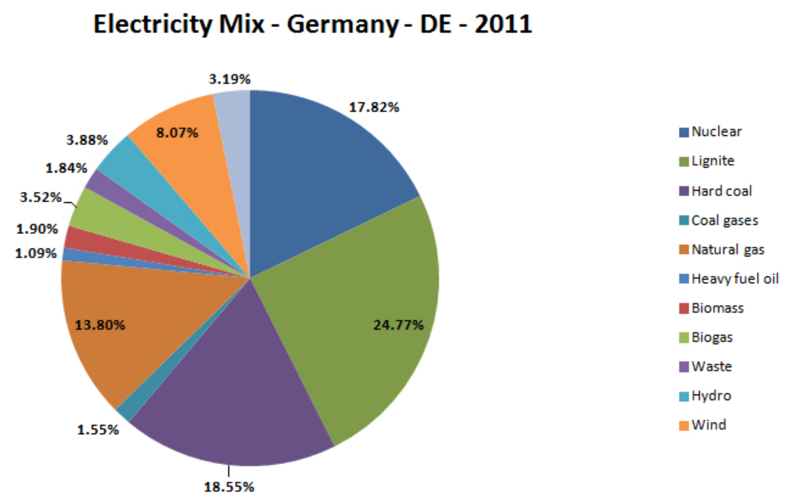
0.2
0.1
0
A
B
C
D
E
Plant

Appendix B.7


Electricity Mix - Ireland - IE - 2011
8.04%
17.00%
53.84%
1.51%
0.49%
0.73%
2.56%
15.84%
Peat
Hard coal
Natural gas
Heavy fuel oil
Biomass
Biogas
Hydro
Wind


Electricity Mix - Spain - ES - 2011
2.54%
0.44%
19.83%
1.36%
13.66%
0.40%
29.02%
5.07%
1.01%
0.30%
0.48%
11.30%
14.57%
Nuclear
Lignite
Hard coal
Coal gases
Natural gas
Heavy fuel oil
Biomass
Biogas
Waste
Hydro
Wind
Photovoltaic
Solarthermal


Electricity Mix - Germany - DE - 2011
3.19%
17.82%
24.77%
18.55%
1.55%
13.80%
1.09%
1.90%
3.52%
1.84%
3.88%
8.07%
Nuclear
Lignite
Hard coal
Coal gases
Natural gas
Heavy fuel oil
Biomass
Biogas
Waste
Hydro
Wind

Appendix C.1

System	Treatment Facility Design Capacity 4,000 (gpd)	10,000 (gpd)	25,000 (gpd)	50,000 (gpd)	100,000 (gpd)
1 MLE					
Construction Cost, $	$ 261,000	$ 311,000	$ 422,000	$ 601,000	$ 874,000
Annual O&M Cost, $/yr	$ 30,400	$ 35,500	$ 49,400	$ 66,600	$ 100,100
Uniform Annual Cost, $/yr	$ 53,200	$ 62,600	$ 86,200	$ 119,000	$ 176,300
Unit Cost, $/1,000 gal	$ 61.8	$ 29.1	$ 16.0	$ 11.1	$ 8.2
2 Four-Stage					
Construction Cost, $	$ 336,000	$ 368,000	$ 475,000	$ 666,000	$ 968,000
Annual O&M Cost, $/yr	$ 52,500	$ 57,600	$ 73,800	$ 95,900	$ 132,300
Uniform Annual Cost, $/yr	$ 81,800	$ 89,700	$ 115,200	$ 154,000	$ 216,700
Unit Cost, $/1,000 gal	$ 95.0	$ 41.7	$ 21.4	$ 14.3	$ 10.1
3 Three-Stage					
Construction Cost, $	$ 291,000	$ 333,000	$ 441,000	$ 627,000	$ 913,000
Annual O&M Cost, $/yr	$ 35,900	$ 41,900	$ 56,400	$ 76,200	$ 115,900
Uniform Annual Cost, $/yr	$ 61,300	$ 70,900	$ 94,800	$ 130,900	$ 195,500
Unit Cost, $/1,000 gal	$ 71.2	$ 32.9	$ 17.6	$ 12.2	$ 9.1
4 SBR					
Construction Cost, $	$ 336,000	$ 381,000	$ 482,000	$ 697,000	$ 966,000
Annual O&M Cost, $/yr	$ 28,000	$ 34,100	$ 49,100	$ 67,600	$ 100,000
Uniform Annual Cost, $/yr	$ 57,300	$ 67,300	$ 91,100	$ 128,400	$ 184,200
Unit Cost, $/1,000 gal	$ 66.5	$ 31.3	$ 16.9	$ 11.9	$ 8.6
5 Intermittent Cycle					
Construction Cost, $	$ 229,000	$ 374,000	$ 584,000	$ 861,000	$ 1,026,000
Annual O&M Cost, $/yr	$ 28,000	$ 34,100	$ 49,100	$ 67,600	$ 100,000
Uniform Annual Cost, $/yr	$ 48,000	$ 66,700	$ 100,000	$ 142,700	$ 189,400
Unit Cost, $/1,000 gal	$ 55.7	$ 31.0	$ 18.6	$ 13.3	$ 8.8
6 MLE + Deep Bed Filtration					
Construction Cost, $	$ 308,000	$ 368,000	$ 486,000	$ 664,000	$ 958,000
Annual O&M Cost, $/yr	$ 36,900	$ 42,700	$ 58,100	$ 75,900	$ 111,400
Uniform Annual Cost, $/yr	$ 63,800	$ 74,800	$ 100,500	$ 133,800	$ 194,900
Unit Cost, $/1,000 gal	$ 74.1	$ 34.7	$ 18.7	$ 12.4	$ 9.1
7 Submerged Biofilters					
Construction Cost, $	$ 247,000	$ 296,000	$ 450,000	$ 847,000	See Note (1)
Annual O&M Cost, $/yr	$ 19,500	$ 24,400	$ 41,100	$ 60,400	See Note (1)
Uniform Annual Cost, $/yr	$ 41,000	$ 50,200	$ 80,300	$ 134,200	See Note (1)
Unit Cost, $/1,000 gal	$ 47.6	$ 23.3	$ 14.9	$ 12.5	See Note (1)
8 RBCs					
Construction Cost, $	$ 263,000	$ 342,000	$ 527,000	$ 868,000	$ 1,092,000
Annual O&M Cost, $/yr	$ 20,400	$ 25,900	$ 43,400	$ 61,500	$ 89,400
Uniform Annual Cost, $/yr	$ 43,300	$ 55,700	$ 89,300	$ 137,200	$ 184,600
Unit Cost, $/1,000 gal	$ 50.3	$ 25.9	$ 16.6	$ 12.7	$ 8.6
9 Baseline – Secondary Treatment					
Construction Cost, $	$ 183,000	$ 223,000	$ 303,000	$ 461,000	$ 671,000
Annual O&M Cost, $/yr	$ 22,000	$ 26,500	$ 39,200	$ 52,100	$ 78,000
Uniform Annual Cost, $/yr	$ 37,900	$ 45,900	$ 65,600	$ 92,300	$ 136,500
Unit Cost, $/1,000 gal	$ 44.0	$ 21.3	$ 12.2	$ 8.6	$ 6.3

Note: (1) Exceeded manufacturer's sizes

Figure 1: Capital and operation costs for nine treatment systems [1]

Appendix C.2

Surface area calculations

Plant footprint estimations were included in the toolkit for several reasons. Firstly, surface area is a hard physical constraint that can exclude certain systems from consideration. In many situations constructed wetlands is the most economical choice of treatment system, but there may not be the surface area available to facilitate their implementation. Secondly, the cost of land in a particular region may be such, that systems with smaller footprints are more economically favourable.

Active area

The active surface area for electro-mechanical based treatment systems can be defined as the sum of the unit-process surface areas (Eq. C.1).

$$TA_{act.} = \sum A_{act.,i} \tag{C.1}$$

Where,

$TA_{act.}$= Total active surface area

$A_{act.,i}$ = is the surface area of a unit process.

Most of the calculated surface areas (aeration and anoxic tanks, primary and secondary settlers etc.) are based on either hydraulic or organic loading rates. However, there are some unit processes that have constant area values. For example, for systems that include mechanical dewatering, because of the plant scale range used in the study it was considered reasonable that these units could have a fixed area. The areas for these units are based on a survey of manufacturers design specifications and on-site investigation. The non-active area simply refers to space not directly linked to the treatment process (paths, roads, grass etc.).

Total area

It is difficult to make very accurate estimations of the total surface area required by a plant without knowing the plant layout in detail. A simple solution is provided here. It is assumed in all cases that surface area is limited and that proper utility of space is being practiced. Each process unit has been given an offset buffer (default of 1.5 m). This buffer maintains a distance between the unit processes. A 1.5 m buffer means that there will be a minimum clearance of 3 m between each of the unit processes. The offset value is soft-coded into the program to allow the user to increase or decrease the buffer. The user should note that a total surface area with an offset value of zero is equal to the active surface area, which is not practical. A minimum offset value of 1.0 is advised. Both primary and secondary clarifiers are circular but it is assumed that the required area is square. The total surface area is calculated with (Eq. C.2).

$$TSA = \sum_{i=1}\left[2\left(\sqrt{\frac{A_{act.,i}}{\pi}}+\Delta\right)^2\right]+\sum_{k=1}\left(\sqrt{A_{act.,k}}+2\Delta\right)^2 \tag{C.2}$$

Where,

TSA = Total surface area (m^2)

i = circular unit process

k = rectangular unit process

Δ = offset value (m)

Process areas

The following section provides information on surface area estimations and calculations for common unit processes. Surface area calculations of unit processes unique to a particular system are included in the individual system models.

Bar screen

The bar screen and skip area is assumed to be 4 m^2 for all system sizes**.**

Drum screen.

Estimates of rotating drum screens areas for the plant scales in question are generally below 2 m^2, and this is the value used for the study regardless of plant scale. Although, these estimates are based on the smallest industrial drum screens available, the units are reported to have a flowrate capacity over 3,000 m^3/day.

Primary sedimentation

Primary sedimentation surface area calculations are based on specified overflow rate (OR). Average overflow rates range from 30 – 50 m^3/m^2.d with a typical value of 40 m^3/m^2.d being reported by [2]. However, the variation in the ratio of peak-average flow is higher for small plants and overflow rates should be set low enough to maintain performance during peak flows. An OR of 25 $m^3/m^2.d$ and a sidewater depth of 4 m have been assumed for the primary tank model. The primary tank surface area ($A_{P,tank}$) is then given by Eq. C.3.

$$A_{P,tank} = Q/OR \tag{C.3}$$

Where Q is the influent flow rate (m^3/d)

Secondary sedimentation tank

The secondary sedimentation tank area is given by Eq. C.4.

$$SS\ tank\ area = \frac{Q}{HAR} \tag{C.4}$$

Where,

Q = flowrate (m^3/day)

HAR = hydraulic application rate (m^3/m^2/day)

Volute

Volute area is estimated as being 2 m^2 regardless of plant scale [3]

Sludge holding tank

Sludge volumes for 2,000 PE plants with high organic loading may be as much 10 m^3/day. Assuming that a storage time of no longer than 3 days is required, the average maximum sludge volume that could be expected is 30 m3. This can be accommodated with a standard 3 m diameter silo (≅ 7 m^2).

Administration building

A power law regression model was developed from cohort of areas of existing administration buildings and is given by (Eq. C.5)

$$Admin\ building\ area = 0.015Q + 0.008 \tag{C.5}$$

Car parking

Car parking space for one car is included. The average car park space is 11.52 m^2.

Wetlands area

Constructed wetlands area requirements per capita are given as 7.7 m^2/PE [4]. However, wetlands are sensitive to temperature and precipitation. The average mean annual temperature precipitation in Greece (location of the referenced study) is 18 °C and 3.25 mm respectively, compared with 10 °C and 11 mm in Ireland. Therefore, a value of 10 m^2/PE has been adopted for use in the toolkit giving a factor of safety close to 25%. This area requirement is less than half of the surface area required for horizontal flow wetlands (20 m^2/PE).

Sludge drying bed area

Sludge drying bed surface area calculations are based on organic loading rates of 80 kg DS/m^2-year [5].

Appendix C.3

Sludge management

Introduction

Sludge management can account for a significant percentage of the total operating costs of a wastewater system. Much of the cost is linked to the cost of sludge transport and final disposal, and therefore, it is economically beneficial to reduce the volume of sludge having to be disposed of as much as possible. However, for small systems, the capital and operational costs associated with sludge treatment and volume reduction may outweigh the cost of simply outsourcing sludge disposal to an external contractor, or transporting sludge to a larger parent plant. As with the other WWT operational cost components, system type, scale, loading and discharge limits will influence the quantities of sludge that are produced and therefore, it is of primary interest to know in what conditions it is economically feasible for a given system to include on-site sludge treatment.

Sludge management is considered here from four perspectives:

- Sludge quantity
- Sludge treatment
- Chemicals
- Disposal route

Sludge quantities

There are several factors that determine the quantity and quality of sludge that is produced for final disposal. Two uncontrollable factors are the influent organic load, and the final effluent discharge limits; both of which, will influence the type and configuration of WWTS that is initially selected. Sludge quantities from screening and primary treatment unit processes are generally consistent across systems. It is the type of secondary treatment and sludge treatment processes that have the largest influence over quantity. The following sections outline sludge quantity calculation methods and assumptions for each unit process.

Screened sludge

For EA and OD systems, a rotary drum medium screen (0.25 mm openings) is used in place of primary sedimentation. Average BOD and TSS removal percentages assumed for the study are 12.5% and 17.5% respectively.

Primary Sludge

For systems with primary sedimentation, primary sludge production is calculated based on the removal efficiency relationship developed by [2] (Eq. C6).

$$R = \frac{t}{a + bt} \qquad \text{(Eq. C6)}$$

Where: R is the removal efficiency, t is the nominal detention time, a and b are empirical constants (Table 5).

Table 6: Typical values for the primary sedimentation empirical constants at 20°C [1]

Constituent	*b*	*a*
BOD	0.02	0.018
TSS	0.014	0.0075

The detention time values are corrected for temperature with Eq. C7. Where M is the detention time multiplication factor, and *t* is the temperature.

$$M = 1.82e^{-0.03t} \qquad \text{(Eq. C7)}$$

Primary sedimentation BOD and TSS removal rates are subject to variations in detention time, temperature and organic loading. With detention times ranging from 1 to 4 hours, BOD removal rates range from 22 - 45%. Similarly, within the same detention time range, TSS removal rates can range from 43 - 65% [2]. If phosphorus removal is achieved by chemical precipitation in the primary stage, up to 15% of additional solids can be produced [2].

Wasted sludge

For AS based systems, wasted sludge quantities will vary with respect to solid retention time. For BOD and TSS removal only, SRTs are kept to a minimum in order to avoid nitrification and excess energy use. However, this results in larger sludge quantities because reduction from endogenous decay does not reach its full potential (Figure 2). Conversely, long SRT AS systems with low food/mass ratios such as EA and OD produce less sludge but expend more energy on aeration.

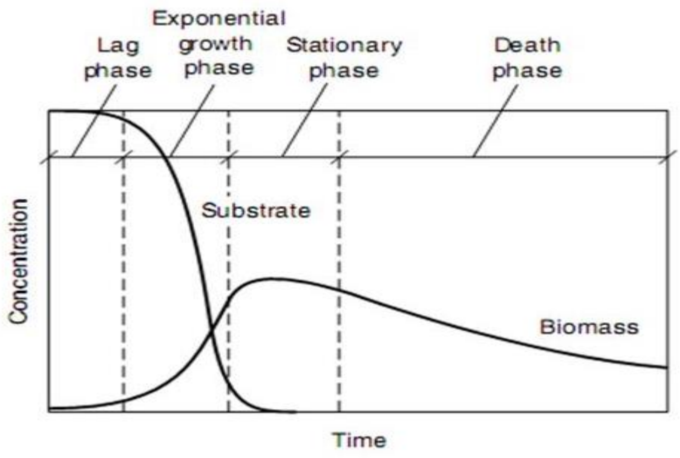


Figure 2: Short SRTs tend to fall within the stationary phase and produce greater sludge volumes

Sludge production from CAS based systems is calculated from primary or screened effluent characteristics. It is assumed that the quantity of sludge wasted per day is equal to the quantity of biomass produced plus inert solids. The equation for calculating the mass of wasted sludge ($P_{X,TSS}$) is given by Eq. C8 [2].

$$P_{X,TSS} = A + B + C + D + \mathrm{E} \qquad \text{(Eq. C8)}$$

Where A represents the mass of sludge produced from heterotrophic biomass growth given by (Eq. C9) [2].

$$A = \frac{QY_H(S_0 - S)}{1 + b_H(SRT)} \qquad \text{(Eq. C9)}$$

Where,

Q = flowrate (m^3/d)

Y_H = yield coefficient (g VSS/g COD)

S_0 = concentration of bCOD (mg/l)

S = concentration of bCOD in effluent (mg/l)

b_H = specific endogenous decay coefficient (g VSS/g VSS•d)

B represents the solids produced from cell debris and is given by (Eq. C10) [2].

$$B = \frac{(f_d)(b_H)QY_H(S_0 - S)SRT)}{1 + b_H(SRT)} \quad \text{(Eq. C10)}$$

Where f_d is the fraction of biomass that remains as cell debris (g VSS/g biomass VSS depleted by decay).

C represents the nitrifying bacteria mass and is given by (Eq. C11) [2].

$$C = \frac{QY_n(NO_x)}{1 + b_n(SRT)} \quad \text{(Eq. C11)}$$

Where, NO_x is the nitrogen concentration.

D represents the non-biodegradable VSS in the influent given by (Eq. C12) [2].

$$D = Q(nbVSS) \quad \text{(Eq. C12)}$$

E represents the influent inert solids given by (Eq. 13) [2].

$$E = Q(TSS_o - VSS_0) \quad \text{(Eq. C13)}$$

Attached growth sludge

Attached growth systems produce higher density, lower volume sludge with better settling quality than AS systems. Attached growth processes (RBC, TF) are reported to yield dry solid concentrations ranging from 1% - 4% [6]. An average value of 2.5 % DS is assumed for attached growth and hybrid system (IFAS). The quantity of sludge produced is calculated with (Eq. 14) [6].

$$AGS = P_x + I_o - E_t \quad \text{(Eq. C14)}$$

Where:

AGS = Attached growth sludge (kg/d)

I_o = influent non-volatile suspended solids (kg/d)

E_t = effluent suspended solids (kg/d)

P_x = net growth of biomass (kg/d), given by (Eq. C15) [6].

$$P_x = QY(S_o - S) - Qb(A_m) \quad \text{(Eq. C15)}$$

Where,

Y = yield coefficient (kg BOD/ kg VSS)

S_o = influent substrate BOD (kg/d)

S = effluent substrate BOD (kg/d)

b = decay coefficient (d^{-1})

A_m = Total media surface area (m^2)

Q = Influent flowrate (m^3/d)

On-site sludge treatment

Mechanical dewatering

The cost associated with transporting sludge to its terminal location is arguably the largest portion of the total sludge management cost. Therefore, it is economically imperative that the volume of sludge to be disposed of is reduced as much as possible. However, capital investment in mechanical sludge thickening and dewatering unit processes may not always be economically feasible, and in cases where there may surface area restrictions it may not be physically feasible. Considering the plant size range adopted for the study, it is assumed that the selected unit process would be compact, low maintenance, easy to operate, and yield good DS concentrations. Volute sludge treatment units provide sludge thickening and dewatering in a single unit process, and comply with most of the prerequisites. Dry solids concentrations range from 20 to 28% [7].

Sludge drying beds

Planted drying beds, also referred to as *humification beds* were chosen in preference to unplanted drying beds for a number of reasons. Firstly, planted beds need only to be desludged every 5 to 10 years; unlike unplanted beds that must be desludged every couple of weeks before a new layer of sludge can be applied. This reduces labour and transport costs significantly. Secondly, planted beds have the extra dewatering pathway through the roots of the plants, which allows for more frequent sludge application. It was also considered that because of the cold and wet climate in Ireland, evaporation levels would be at the lower end of the scale and the system would benefit from the additional dewatering pathway. Upon removal, the sludge dry solids concentrations range from 40% to 70% [5].

Dry solid concentrations

Sludge production values are presented as kg DS/day. Table 6 presents the sludge solids concentrations adopted for the study. It is assumed that the solids concentration of fine-screen sludge used in extended aeration systems is similar to that of primary treatment (4.3%).

Table 7: Sludge dry solids concentrations assumed for the study

Sludge type	*Range of DS concentrations (%)*	*Assumed value (DS) (%)*	*Reference*
Primary	2 - 7	4.3	[6]
Drum screen	2 – 7	4.3	---
SBR	2.6 - 5.7	4.3	[8]
Waste activated	0.4 – 1.5	1.3	[6]
Attached growth	1 - 4	2.5	[6]
Volute		24	[7]
Drying beds	40 - 70	50	[5]

Appendix C.4

Chemicals

The chemicals used for sludge thickening, dewatering and stabilisation represent a large percentage of the economic cost associated with sludge management. Furthermore, in addition to the specific cost of the chemicals themselves, chemicals produce additional sludge that has to be disposed of.

It was evident from the preliminary LCA study that chemical use is also responsible for a significant portion of the environmental impact associated wastewater treatment. As with most of the other elements of a system's LCI and LCCI, the quantities of chemicals required are directly related to site-specific conditions. Table 7 presents the chemicals used in this study.

Table 8: Chemicals and specific costs

Chemical	*Formula*	*Cost*	*Reference*
Ferric chloride	$FeCl_3$	€ 0.70 /L	(personal communication, Acorn Water, Bandon, Co. Cork, Ireland)
Sodium hydroxide	NaOH	€ 0.77/kg	[9]
Calcium hydroxide[2]	$Ca(OH)_2$	€ 0.20/kg	[10]
Polymers (acrylic acid)	variable	€ 5/kg	[11]
Calcium hypochlorite[3]	$Ca(OCl)_2$	€ 1.53 /kg	[12]
Ethanol	C_2H_6O	€ 0.65 /L	[13]

Chemical quantities

Polymers

Required polymer dosages for thickening and dewatering vary with treatment unit type and DS concentration. Polymer dosages for the Volute thickening and dewatering units were not available. A simplification was made whereby dosages were based on average values reported for centrifuge dewatering units as these are similar in design and operation. The dosages by reported Mamais et al [14] were 9.22 g polymer/kg dry solids.

[2] Estimated cost is based on U.S values adjusted from 2013 to 2017.

[3]Original price was quoted for 65% available chlorine; price presented here has been adjusted to represent 100% chlorine.

Alkaline stabilisation

There are two types of dry lime used for sludge stabilisation: hydrated lime or *slaked lime* [$Ca(OH)_2$] and quicklime (CaO). Although slaked lime is slightly more expensive than quicklime, it is more commonly used by plant operators. Slaked lime is available at values of 80% $Ca(OH)_2$. Dosage requirement are presented in Table 8. The specific cost of lime varies depending on location. The specific cost assumed for this study is €0.2/kg [10].

Table 9: Lime stabilisation dosage (Adapted from [2])

Sludge type	*Solids concentration (%)*	*$Ca(OH)_2$ dosage range (g/kg DS)*	*Model values (g/kg DS)*
Primary	4.3	60 - 170	120
Secondary	1.3	210 - 430	300
Mixed sludge (60:40) P&S	3.8		192

Ferric chloride for phosphorus removal

Quantities of Fe_3Cl required for phosphorus removal are determined by Figure 3 [2], whereby the molar ratio of iron to influent soluble phosphorus is given as a function of the required phosphorus effluent concentration (mg/l). It is assumed that the Fe_3Cl solution is available at 37% (~ 0.5 kg/L of solution).

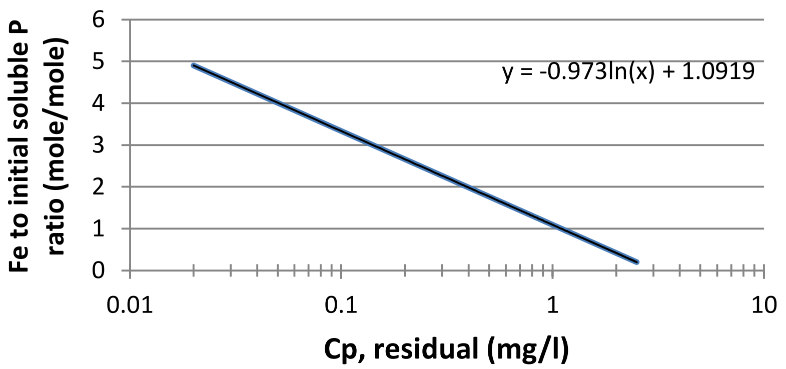


Figure 3: Required Fe as a function of influent phosphorus concentration. Adapted from [2]

Ethanol addition

An additional carbon source may be required for systems with low TN discharge limits. For systems with pre-anoxic tanks, the influent wastewater generally provides adequate substrate for denitrification; however, for systems with weak influent wastewater (low BOD/TKN ratio < 3) or where post-anoxic tanks are used an external carbon may be required. There are a number of organic compounds available that could be used as a carbon source. Inexpensive options used by small WWTPs included molasses and corn syrup. Two of the more widely used industrial compounds are methanol and ethanol [2]. Methanol is commonly chosen as an external carbon source because of its lower cost per g NO_x removed. However, ethanol has higher denitrification rates and is safer to handle. The following section presents the ethanol quantity calculation method for post-anoxic denitrification.

1) Select an anoxic volume and determine the required standard denitrification rate (SDNR) with ethanol from Eq. C.15 [2]

$$R_{NO_3} = SDNR(X_{VSS})(V) + \left(\frac{1.42}{2.86}\right)(b_H)(X_H)(V) \tag{C.15}$$

Where,

R_{NO_3} = amount of nitrate to be removed in the post anoxic tank (g/d)

X_{VSS} = mixed liquor volatile suspended solids (g/m^3)

V = anoxic tank volume (m^3)

b_H = endogenous decay coefficient (g/g.d)

X_H = biomass concentration (g/m^3)

2) Determine effluent ethanol concentration to achieve required SDNR with Eq. C.16 [2]

$$SDNR = \frac{1 - 1.42Y_H}{2.86}\left[\frac{\mu_{max}S_s}{Y_H(K_s + S_s)}\right]\left(\frac{S_{NO_3}}{K_{NO_3} + S_{NO_3}}\right)\left[\frac{(\eta)X_H}{X_{VSS}}\right] \tag{C.16}$$

Where,

Y_H = heterotrophic yield coefficient(g/g.d)

μ_{max}= maximum substrate utilisation rate (g/g.d)

S_s = effluent ethanol concentration (g/m^3)

K_s= ethanol half-velocity coefficient (5.0 g COD/m^3)

S_{NO_3} = required nitrate effluent concentration (g/m^3)

K_{NO_3} = nitrate velocity half constant (0.10 g/m^3)

η = ethanol degradable biomass fraction

3) Determine ethanol consumptive ratio, C_{R,NO_3} , with Eq. C.17

$$C_{R,NO_3} = \frac{2.86}{1 - 1.42Y_H}(NO_{3,in}^{-} - NO_{3,eff}^{-}) \quad \text{(C.17)}$$

4) Calculate ethanol dose (g ethanol/d) with Eq. 18

$$Dose = SDNR\ (X_{VSS})(X_H)(C_{R,NO_3} + Q(1 + R)(S_s) \quad \text{(C.18)}$$

Where R is the return activated sludge ratio (0.6 assumed)

Chlorination

Chlorination is provided as the option for disinfection. It is assumed that the influent flowrate is the quantity to be disinfected and that wasted sludge volumes are negligible. Chlorination values reported by [15] range from 5 mg to 20mg Cl_2/l of treated wastewater. This reflects seasonal variations where summer months and higher temperatures generally result in an increase in coliform concentration. As a simplification, the *breakpoint chlorination* process was used to calculate chlorine demand. A value of 10:1 is assumed for the chlorine - ammonia (NH_3) molecular weight ratio. It is assumed that the ammonia discharge limit is the residual ammonia quantity used to calculate chlorine demand. Calcium hypochlorite [$Ca(OCl)_2$] was chosen as the form of chlorine. Although [$Ca(OCl)_2$] is more expensive than liquid or gas forms of chlorine, it has been reported by [2] that this form is generally preferred by operators of small WWTPs because of handling and administration issues. A value of 70% Cl_2 is assumed for [$Ca(OCl)_2$].

Sludge disposal route

Table 9 presents details of the sludge disposal options available to small treatment plants operators. In the case of delivery to a parent plant, it is assumed that the cost is limited to the cost of transport only i.e. there is no gate fee involved. The option of sludge pumping from child to parent plant has been considered. However, it has been determined that the cost of pumping only becomes economically feasible when the sludge flowrate is greater than 300 m^3/d [16]. The cost of sludge disposal by contractor ranges from €45/m^3 to €75/m^3 for digested sludge, and €60/m^3 to €90/m^3 for undigested sludge. Anaerobic digestion is not an option for the plant scale range in question, and therefore, an average of €75/m^3 is assumed. The inclusion of the parent plant and external contractor with both treated and untreated sludge is primarily to assess the effect of volume reduction.

Table 10: Sludge disposal options

Sludge Type	*Disposal option*	*Specific costs*	*Source*
Untreated			
	Transport to parent plant	€0.66/m^3/km	(Mooney Transport, Birr Co. Offaly, sales representative, personal communication, Dec. 2016)
	External contractor	€75/m^3	(Enva Ireland[4], sales representative, personal communication, November 15, 2016)
Treated			
(D+S)[5]	Land spreading	€60/kg	
(D)[6]	Transport to parent plant		(as above)
(D)	External contractor		(as above)

Summary

The cost of sludge management is influenced by the quantity and quality of sludge produced, which is determined by the system type, scale, loading, and discharge limits. The decision to invest in on-site sludge treatment may also depend on available sludge disposal options. However, any decision relating to the economics of sludge management should not be made in isolation, but rather included in the entire LCCA of the entire wastewater treatment system.

[4] Enva is a waste management company in Ireland that provides sludge disposal services

[5] Dewatered and stabilised

[6] Dewatered only

Appendix C.5

Northeast labour hour per process data sheets [17].

The Northeast Guide for Estimating Staffing at Publicly and Privately Owned Wastewater Treatment Plants

CHART 1 (One Shift)
BASIC AND ADVANCED OPERATIONS AND PROCESSES

Process	Flow 0.25-0.5 mgd	0.5-1.0 mgd	1.0-5.0 mgd	5.0-10.0 mgd	10.0-20.0 mgd	>20 mgd	Total Hours for Plant
Preliminary Treatment	130	130	260	520	780	1040	
Primary Clarification (mult. by # of units)	130	130	130	260	260	260	
Activated Sludge	520	1040	1560	1560-2080	2080-2600	6240	
Activated Sludge w/BNR	780	1560	2080	2340-3120	3120-6240	7280	
Rotating Biological Contactor	260	390-780	780-1560	1560	X	X	
Sequencing Batch Reactor (per tank)	260	260	260	260	260	260	
Extended Aeration (w/o primary)	650	1300	2080	X	X	X	
Extended Aeration w/BNR	910	1820	2600	X	X	X	
Pure Oxygen Facility	X	X	X	2080-2600	2600	4680	
Pure Oxygen Facility w/BNR	X	X	X	2600-3900	3900	6240	
Trickling Filter	260	260	520	780	1040	2080	
Oxidation Ditch (w/o primary)	650	1300	2080	X	X	X	
Oxidation Ditch w/BNR	910	1820	2600	X	X	X	
Aeration Lagoon	390	390	390	X	X	X	
Stabilization Pond	260	260	260	X	X	X	
Innovative Alternative Technologies	520	780	X	X	X	X	
Nitrification	65	65	130	130	260	520	
Denitrification	65	65	130	130	260	520	
Phosphorus Removal (Biological)	65	65	130	130	260	520	
Phosphorus Removal (Chemical/Physical)	65	130	260	520	780	1040	
Membrane Processes	65	65	130	130	260	260	

continued on page 24

CHART 1 (One Shift) *continued*
BASIC AND ADVANCED OPERATIONS AND PROCESSES

	Flow						
Process	**0.25-0.5 mgd**	**0.5-1.0 mgd**	**1.0-5.0 mgd**	**5.0-10.0 mgd**	**10.0-20.0 mgd**	**>20 mgd**	**Total Hours for Plant**
Cloth Filtration	65	65	130	130	130	130	
Granular Media Filters (Carbon, sand, anthracite, garnet)	130	260	260	390	390	780	
Water Reuse	65	65	130	130	130	130	
Plant Reuse Water	26	26	26	39	65	65	
Chlorination	130	130	260	260	260	260	
Dechlorination	130	130	260	260	260	260	
Ultraviolet Disinfection	130	130	260	260	260	260	
Wet Odor Control (mult. by # of systems)	130	130	260	260	260	260	
Dry Odor Control (mult. by # of systems)	65	65	130	130	130	130	
Septage Handling	130	130	260	260	260	260	
TOTAL							

- Activated Sludge process includes RAS and WAS pumping.
- Secondary Clarification has been built into basic operations processes.

CHART 2 (One Shift) MAINTENANCE								
	Flow							
Activity	0.25-0.5 mgd	0.5-1.0 mgd	1.0-5.0 mgd	5.0-10.0 mgd	10.0-20.0 mgd	>20 mgd	Multiply by	Total Hours for Plant
Manually Cleaned Screens	65	65	65	65	130	260	# of screens	
Mechanically Cleaned Screens	65	65	65	260	780	1040	# of screens	
Mechanically Cleaned Screens with grinders/ washer/compactors	65	130	260	520	1040	1300	# of screens	
Comminutors/ Macerators	65	65	65	130	195	260	# of units	
Aerated Grit Chambers	26	26	65	130	195	260	# of chambers	
Vortex Grit Removal	26	26	65	130	195	260	# of units	
Gravity Grit Removal	26	26	39	52	104	130	# of units	
Additional Process Tanks	26	26	26	26	26	26	# of tanks	
Chemical Addition (varying dependent upon degree of treatment)	26	26	26	26-78	78-156	208	# of chemicals added for processes	
Circular Clarifiers	65	65	130	130	195	260	# of clarifiers	
Chain and Flight Clarifiers	65	65	130	130	195	260	# of clarifiers	
Traveling Bridge Clarifiers	X	X	X	X	195	260	# of clarifiers	
Squircle Clarifiers	65	65	130	130	195	260	# of clarifiers	
Pumps	100	100	250	500	750	1500	X	
Rotating Biological Contactor	39	39	65	65	X	X	# of trains	
Trickling Filters	39	39	39	65	104	130	# of TFs	
Sequencing Batch Reactor	39	39	39	65	104	130	# of tanks	
Mechanical Mixers	26	26	26	26	39	52	# of mixers	
Aeration Blowers	52	52	52	52	78	104	# of blowers	
Membrane Bioreactor	26	26	26	52	78	104	# of cartridges	

Continued on page 26

CHART 2 (One Shift) *continued*
MAINTENANCE

	Flow							
Activity	**0.25-0.5 mgd**	**0.5-1.0 mgd**	**1.0-5.0 mgd**	**5.0-10.0 mgd**	**10.0-20.0 mgd**	**>20 mgd**	**Multiply by**	**Total Hours for Plant**
Subsurface Disposal System	26	26	26	26	78	104	# of systems	
Groundwater Discharge	26	26	26	26	39	52	X	
Aerobic Digestion	26	26	26	26	39	52	# of digesters	
Anaerobic Digestion	X	52	52	78	156	260	# of digesters	
Gravity Thickening	26	26	26	26	78	104	# of basins	
Gravity Belt Thickening	39	39	39	65	104	130	# of belts	
Belt Filter Press	39	39	39	65	104	130	# of presses	
Mechanical Dewatering (Plate Frame and Centrifuges)	39	39	39	65	104	130	# of units	
Dissolved Air Floatation	X	26	26	26	78	104	# of units	
Chlorination (gas)	26	26	26	52	78	104	X	
Chlorination (liq.)	52	52	52	78	117	156	X	
Dechlorination (gas)	26	26	26	52	78	104	X	
Dechlorination (liq.)	52	52	52	78	117	156	X	
Ultraviolet	26	26	26	39	65	78	# of racks	
Biofilter	130	130	130	130	130	130	# of units	
Activated Carbon	130	130	130	195	195	260	# of units	
Wet Scrubbers	X	X	X	39	65	78	# of units	
Microscreens	26	26	26	39	65	78	# of screens	
Pure Oxygen	X	X	X	52	78	104	# of units	
Final Sand Filters	52	52	52	52	78	156	# of units	
Probes/ Instrumentation/ Calibration	26	26	26	26	26	26	# of probes in-line	
TOTAL								

CHART 3 (One Shift) LABORATORY OPERATIONS					
	How often are tests run?				
Test Required by Permit	Testing Time (hrs.)	Tested Weekly X 52	Tested Monthly X 12	Tested Quarterly X 4	Annual Hours
Acidity	0.75				
Alkalinity, total	0.75				
Biochemical Oxygen Demand (BOD)	2.5				
Chemical Oxygen Demand (COD)	2.5				
Chloride	0.5				
Chlorine, Total Residual	0.25				
Coliform, Total, Fecal, E.Coli	1.0				
Dissolved Oxygen (DO)	0.25				
Hydrogen Ion (pH)	0.25				
Metals	3.0				
Toxicity	2.0				
Ammonia	2.0				
Total Nitrogen	2.0				
Oil and Grease	3.0				
Total and Dissolved Phosphorus	2.0				
Solids, Total, Dissolved, and Suspended	3.0				
Specific Conductance	0.25				
Sulfate	1.0				
Surfactants	1.0				
Temperature	0.25				
Total Organic Carbon (TOC)	0.25				
Turbidity	0.25				
Bacteriological Enterococci	1.0				
Lab QA/QC Program	1.0				
Process Control Testing	3.0				
Sampling for Contracted Lab Services	0.25				
Sampling for Monitoring Groundwater Wells	0.5				
TOTAL					

• Sampling time is built into testing time estimates.

CHART 4 (One Shift)
BIOSOLIDS/SLUDGE HANDLING

	Flow					
Process	**0.25-0.5 mgd**	**0.5-1.0 mgd**	**1.0-5.0 mgd**	**5.0-10.0 mgd**	**10.0-20.0 mgd**	**>20 mgd**
Belt Filter Press	260	780	1560	2080	2080	2080/shift
Plate & Frame Press	260	390	780	2080	2080	2080
Gravity Thickening	65	65	130	130	260	260
Gravity Belt Thickening	65	65	130	130	260	520
Rotary Press	65	65	130	130	260	520
Dissolved Air Floatation	X	130	130	260	260	260
Alkaline Stabilization	65	65	65	65	65	65
Aerobic Digestion	130	130	130	260	390	520
Anaerobic Digestion	65	65	130	390	650	1040
Centrifuges	260	260	780	2080	2080	2080
Composting	260	520-780	1040	2080	2080	2080/shift
Incineration	X	X	X	X	6240	6240
Air Drying – Sand Beds	130	130	X	X	X	X
Land Application	65	130	130	X	X	X
Transported Off-site for Disposal	65	260	1040	2080	2080	2080
Static Dewatering	260	260	X	X	X	X
TOTAL						

Appendix D.1

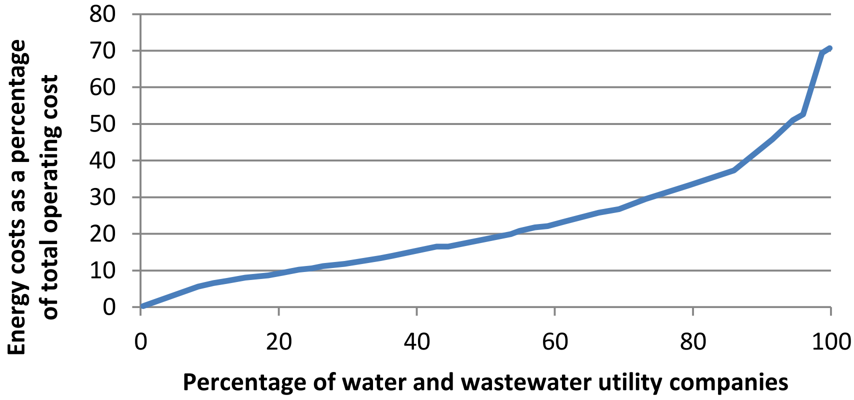


Figure 4: Energy costs as a percentage of total operating cost for Central and Eastern European waste and wastewater utility companies [18]

Appendix D.2

Aeration energy

Aeration energy is calculated in five stages:

- Determine O_2 demand
- Determine standard oxygen transfer rate (SOTR)
- Determine airflow
- Calculate inlet and outlet pressures
- Calculate blower power requirements

Calculate oxygen demand

Oxygen demand is calculated from the amount of bCOD (biodegradable chemical oxygen demand) oxidised per day (Eq. D.1). The assumption here is that all of the bCOD, except for the quantity that is removed with the wasted sludge is converted to end products (CO_2, H_2O) [2].

$$R_o = Q(S_o - S) - 1.42P_X + 4.57(Q)NO_x \qquad \text{(D.1)}$$

Where,

R_o = Required oxygen (kg O_2/d)

Q = Influent flowrate (m^3/d)

S_o = Influent substrate concentration (g bCOD/m^3)

S = Effluent substrate concentration (g bCOD/m^3)

P_X = Biomass production (kg VSS/d)

NO_x = Oxidised nitrogen produced (g /m^3)

Calculate standard oxygen transfer rate

The standard oxygen transfer rate is calculated using Eq. D.2. Adapted from [2].

$$SOTR = \left(\frac{OTR_f}{\alpha F}\right)\left[\frac{C^*_{\infty 20}}{\beta(C_{st}/C^*_{s20})(P_b/P_a)(C^*_{\infty 20}) - C}\right][(1.024)^{20-T}] \quad \text{(D.2)}$$

Where,

$SOTR$ = standard oxygen transfer rate (kg O_2/h)

OTR_f = actual oxygen transfer rate at site (kg/h)

α = relative transfer rate to clean water (unitless)

β = relative DO saturation to clean water (unitless)

F = diffuser fouling factor (unitless)

P_a = standard pressure at sea level (Pa)

P_b = pressure at plant site based on elevation (Pa)

C_{st} = saturated DO at sea level and operating temperature (mg/l)

C^*_{s20} = saturated DO value at sea level and operating temperature (mg/l)

$C^*_{\infty 20}$ = saturated DO value at sea level and 20°C for diffused aeration (mg/l)

$$C^*_{\infty 20} = C^*_{s20}\left[1 + d_e\left(\frac{D_f}{P_a}\right)\right] \quad \text{(D.3)}$$

Where,

D_f = depth of diffusers in basin (m)

C = operating DO in basin (mg/l)

D = aeration basin temperature (mg/l)

d_e = mid depth correction factor (unitless)

$$\frac{P_b}{P_a} = exp\left[-\frac{gM(z_b - z_a)}{RT}\right] \quad \text{(D.4)}$$

Where:

g = gravitational constant (9.81 m/s^2)

M = molecular weight of air (kg/kmol)

R = universal gas constant (8.314 J/mol·K)

T = temperature (K)

Calculate airflow

$$Af = \frac{SOTR}{(OTE) \times (60\ min/h)(\rho_{air.E})} \quad \text{(D.5)}$$

Where,

Af = airflow (m^3/min)

OTE = oxygen transfer efficiency (kg O_2/kWh) (assumed 35% for fine bubble diffusers)

$\rho_{air.E}$ = oxygen density of air at elevation E (kg O_2/m^3 air)

Piping and diffuser headloss are assumed negligable for the plant scale range in question. The tank depth model calculation is an iterative process that begins with a default depth of 6 m and reduces the value with discrete iterations of 0.1 m until the conditions of Eq.D.6 are met. This method does not specify tank surface area geometry.

$$D_{tank} \geq 1.2\sqrt{\frac{4A}{\pi}} \quad \text{(D.6)}$$

Calculate inlet and outlet pressures

$$P_1 = \rho g \Delta H \qquad \text{(D.7)}$$

Where,

P_1 = inlet pressure (kPa)

ρ = density of wastewater (kg/m^3)

g = gravity (9.81 m/s^2)

$\Delta H = D_{tank}$ – diffuser height (m)

$$P_2 = P_1 + P_b \qquad \text{(D.8)}$$

Calculate blower power requirement

Blower power is given by Eq. D.9

$$P_w = \frac{QP_1}{17.4(\eta_B \eta_M)}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right] \qquad \text{(D.9)}$$

Where,

P_w = blower power requirements (wire power) (kW)

Q = airflow rate (m^3/min)

η_B = blower efficiency

η_B = motor efficiency

Appendix D.3

Pumping energy modelling

The following sections provide details of the key parameters and components of the pumping models.

Pump type

A rotary-lobe positive displacement pump (PD) is the type specified for the models. Reported efficiencies range from 45 – 65% [2] (assumed average value of 55% used in the models). These types of pumps are commonly used in small WWTSs (capacities < 450 m^3/h [19]). Advantages of PD pumps are:

- Self-priming
- Space requirements are low
- High tolerance for rags and large solids
- Ability to handle a wide range of sludge viscosities
- Can pump sludges with up to 6% dry solids
- Can be run dry without damage
- Check valves are not required
- Although initial capital investment is relatively high, parts are inexpensive and easily replaced
- Efficiencies are not as effected by operating away from the system curve as other pumps

Pipe lengths and static head

Estimations of pipe lengths are based on calculated unit process surface area and the offset clearance buffer (see Appendix C.1). A value of 6 m has been assumed for inlet pumping static head. This is considered to be a reasonable estimation based on the scale range in question. It is assumed that the plants are designed with adequate fall to negate the need for pumping between unit processes in the water treatment line. Return activated sludge, and nitrate recycle lines have been assigned static head values of 4 m (sidewall depth of secondary settling tank) and 0 m respectively (assumed horizontal return line). A value of 7 m has been assumed for the sludge line static head. This is estimated as the height of the primary and secondary settling tank side walls plus the height of the sludge storage silo, or the mixing tank prior to sludge conditioning for scenarios that include sludge treatment. It is assumed that the output from the sludge thickening dewatering unit falls directly into a storage container without need for pumping.

Minor headloss coefficients

The minor headloss coefficient values are sourced from White [20] and Jones et al. [19].

Frictional headloss

According to Jones et al. [19], headloss for sludges with solid concentrations less than 2% can be modelled as water. For Newtonian fluids such as water, changes in pressure are directly proportional to changes in fluid velocity and viscosity. For sludge flows with higher solids concentrations such as primary and dewatered sludge the sludge flow is generally laminar, and for non-Newtonian fluids such as sludge, pressure variations are not proportional to flowrate during laminar flow, and therefore flowrate does not have a linear relationship with viscosity. This will effect estimations of frictional headloss within a system, which are dependent on rheological properties such as viscosity, elasticity and plasticity [19].

Thick sludge is considered to behave like a Bingham plastic during laminar flow conditions where a linear relationship exists between the fluid shear stress and shear rate once flow has begun. This relationship is referred to as the *coefficient of rigidity*. The earliest reported research in the area of sludge pumping was carried out by Babbitt and Caldwell [21], who, as well as developing methods to measure sludge characteristics, sought to determine and formulate the major factors influencing frictional loss through use of the Bingham equation (Eq. D.10).

$$\frac{H}{L} = \frac{16s_y}{3D\rho g} + \frac{32\eta\upsilon}{\rho g D^2} \tag{D.10}$$

Where,

H = frictional headloss (m)

L = pipe length (m)

s_y = shear stress (Pa)

η= coefficient of rigidity (kg/m.s)

υ = fluid velocity (m/s)

D = pipe diameter (m)

g = acceleration due to gravity (m/s^2)

ρ = fluid density (kg/m^3)

This method requires values for the coefficient of rigidity and yield stress, but is only applicable in laminar flow conditions. The application of the Bingham equation requires knowledge of sludge characteristics that may not be available to designers prior to plant design. Average yield stress and coefficient of rigidity data for sludge provided by Jones et al. [19] s are presented below (Figure 5 and Figure 6).

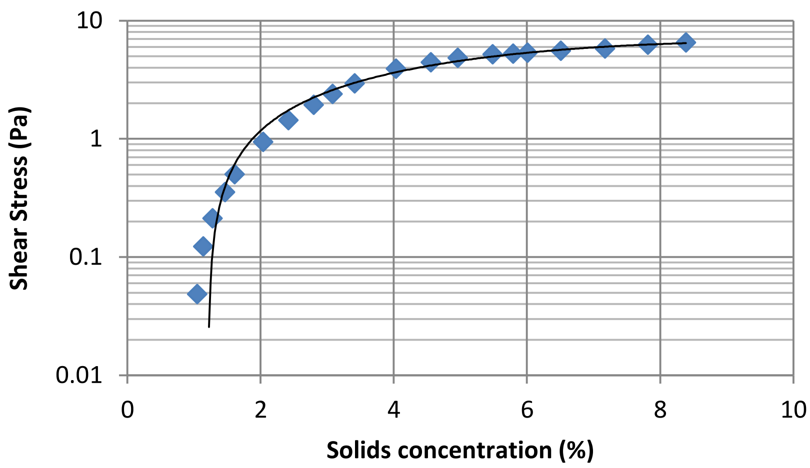


Figure 5: Shear stress as a function of solids concentration [19]

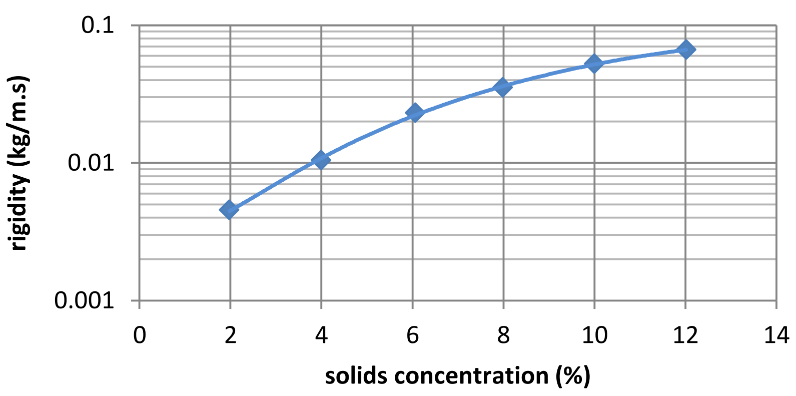


Figure 6: Coefficient of rigidity as a function of solids concentration [22]

More recent work carried out by Mulbarger et al. [23] used the shear stress and coefficient of rigidity constants to produce a relationship between sludge fluid velocity and sludge frictional headloss as a function of frictional headloss for water (Figure 7). It can be seen here that the frictional headloss is inversely proportional to the fluid velocity. It has been reported by Poloski et al. [24] that the minimum velocity required to avoid solids deposition is greater than 1.83 m/s. This suggests that at velocities above 1.83 m/s sludges with solids concentrations less than 5% can be considered to have flow characteristics similar to that of water for the purpose of modelling frictional headloss.

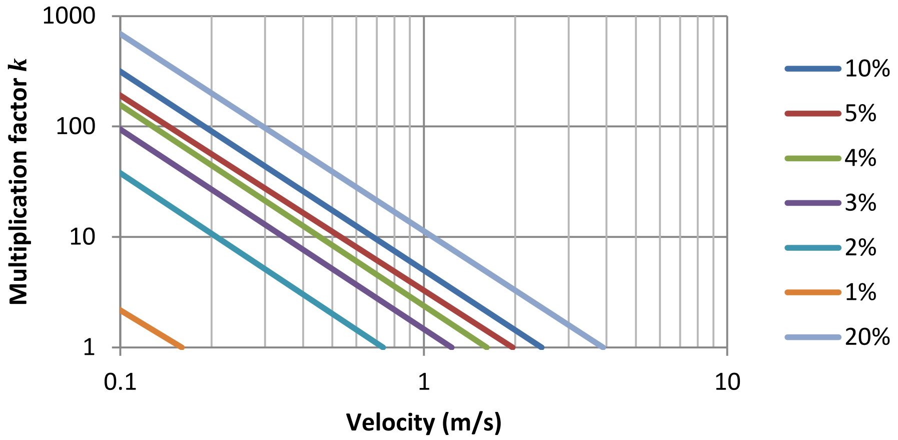


Figure 7: Frictional headloss prediction for routine operation adapted from Mulbarger [23]

A simple model for calculating headloss in sludge pumping was presented in Metcalf and Eddy [2]. The model provides a multiplication factor *k*, as a function of solids concentration (Figure 8). The factor *k*, is then multiplied by the frictional headloss for water to produce a value for sludge headloss.

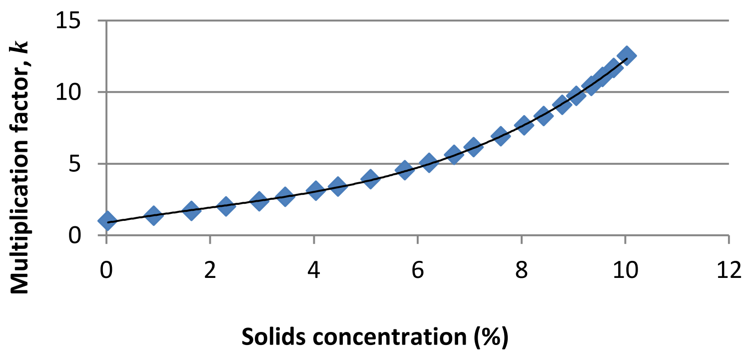


Figure 8: Frictional headloss model based on solids concentration [2]

Frictional headloss as a function of flowrate was calculated using the Mulbarger model, Bingham equation, and the simple model. Frictional headloss values as a function of flowrate are presented below (Figure 9). The sludge solids concentration was 3%. The fluid velocity was kept constant at 1.83 m/s by changing the value of the pipe diameter and pumping times. The temperature was set to 20 °C. Good agreement was observed between the Bingham equation and Mulbarger method. The simple model yielded values averaging 3 times higher. The simple model does not account for velocity and this could be responsible for the large difference in output between the calculated values.

Economies of scale were observed for all three models. From the Bingham equation, the decrease in headloss is a result of the two diameter terms in the denominators that increase with the flowrate in order to maintain minimum velocity. A similar trend occurs in the Mulbarger model due to the diameter increase in the Darcy-Weisbach equation. Either of these two models could be chosen as there was very little difference in output. The effect of the absence of a velocity parameter in the simple model removed is from consideration. Once the Reynolds number is known, the Mulbarger method requires only values for fluid velocity and solids concentration making calculations relatively straightforward. The application of the Bingham equation requires knowledge of sludge characteristics that may not be available to designers prior to plant design such as shear stress and coefficients of rigidity. Hence, the Mulbarger method is used in the pumping models.

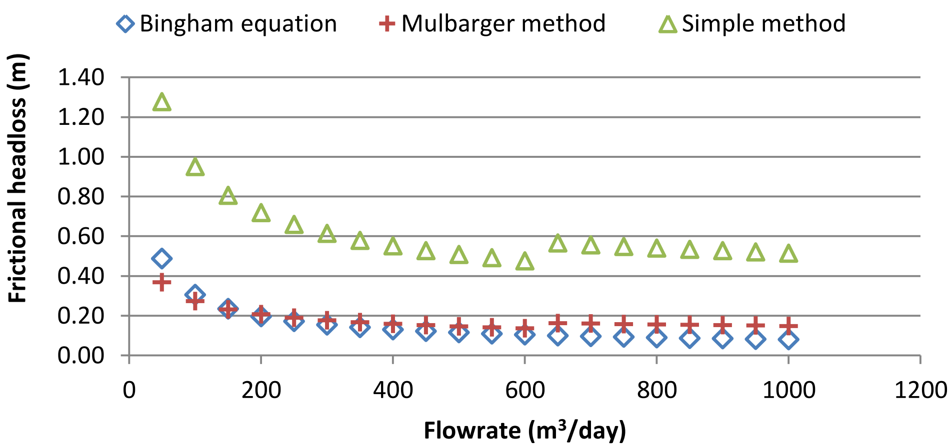


Figure 9: The Bingham and the Mulbarger frictional headloss models exhibit similar trends. The simple model given by [2] is much greater for smaller flowrate values

Pump model

Total dynamic head (TDH) is calculated as the sum of the static head ($\Delta\mathrm{H}$), friction losses (h_f) and minor losses (h_m) (Eq. D.11)

$$\mathrm{TDH} = \Delta\mathrm{H} + h_f + h_m \tag{D.11}$$

The static head is given by Eq. D.12. Where Z_0 is the elevation at the pipe inlet and Z_1 is the elevation at the point of fluid discharge.

$$\Delta\mathrm{H} = \mathrm{Z}_1 - \mathrm{Z}_0 \tag{D.12}$$

The minor losses account for bends in pipes, losses through valves and screens and other appurtenances, and are given by Eq. D.13.

$$\text{minor } losses = \sum k_i \frac{v^2}{2g} \tag{D.13}$$

Where k is the coefficient of the appurtenance i, v is the fluid velocity, and g is acceleration due to gravity. Frictional head loss for water is calculated with a modified version of the Darcy-Weisbach Equation (Eq. D.14). The Mulbarger multiplication factor m_f is included here, but is only applicable for primary sludge.

$$h_f = m_f f_f \frac{L}{D} \frac{v^2}{2g} \tag{D.14}$$

Where h_f is the frictional head loss, L is the pipe length, D is the pipe diameter, v is the fluid velocity, g is the acceleration due to gravity. In the laminar flow region where Re < 2000 the friction factor, f_f, is $\frac{64}{Re}$ and is independent of roughness, Re is the Reynolds number for fluid flow in a circular pipe (Eq. D.15), ρ is the density of the fluid, v is the fluid velocity, D_h is the pipe diameter and μ is the fluid viscosity.

$$\mathrm{Re} = \frac{\rho v \mathrm{D}_h}{\mu} \tag{D.15}$$

For 2000 < Re < 4000 the Reynolds number is in a transitional period and the friction factor is indeterminate [19]. For Re >> 4000 the friction factor can be calculated with the Colebrook-White equation [25] using an iterative method (Eq. D.16), where e is the absolute roughness in millimetres, and D is the inside diameter in millimeters. However, the iterative method can be cumbersome and time consuming. Haaland [26] developed an approximate explicit definition of the friction factor (Eq. D.17)

$$\frac{1}{\sqrt{f}} = -2log\left(\frac{e}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right) \tag{D.16}$$

$$\frac{1}{\sqrt{f}} = -1.8log\left[\left(\frac{69}{RE} + \frac{\varepsilon/D}{3.7}\right)^{1.11}\right] \tag{D.17}$$

Changes in viscosity with changes in temperature are modelled as presented in Figure 10.

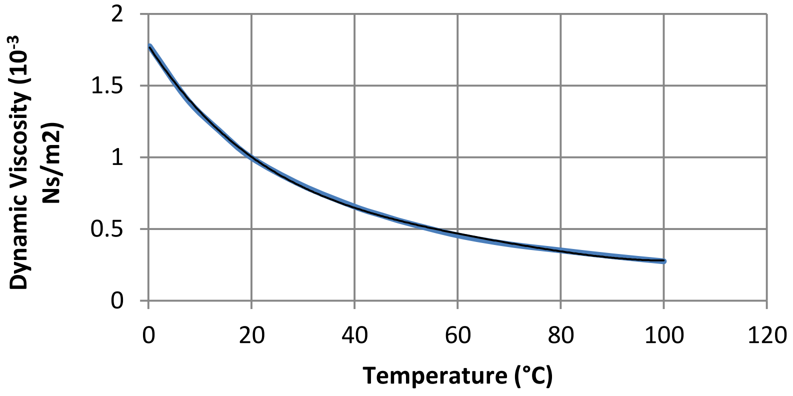


Figure 10: Temperature correction factor for viscosity

The individual headloss expressions combine to form Eq. D.18.

$$\text{TDH} = \text{Z}_1 - \text{Z}_0 + \frac{v^2}{2g}\left(\frac{64m_f\mu L}{\rho v D^2} + \sum_{i=1} k_i\right) \tag{D.18}$$

Converting to power, P, required and including motor (η_m) and pump (η_p) efficiencies yields Eq. D.19.

$$P = \left[Z_1 - Z_0 + \frac{v^2}{2g}\left(\frac{64 m_f \mu L}{\rho v D^2} + \sum_{i=1} k_i\right)\right]\left(\frac{\rho g Q}{\eta_m \eta_p}\right) \tag{D.19}$$

Appendix E.1

Complete mix activated sludge (single stage)

System description

The CMAS system layout is presented in below (Figure 11). The unit process descriptors are presented in Table 10.

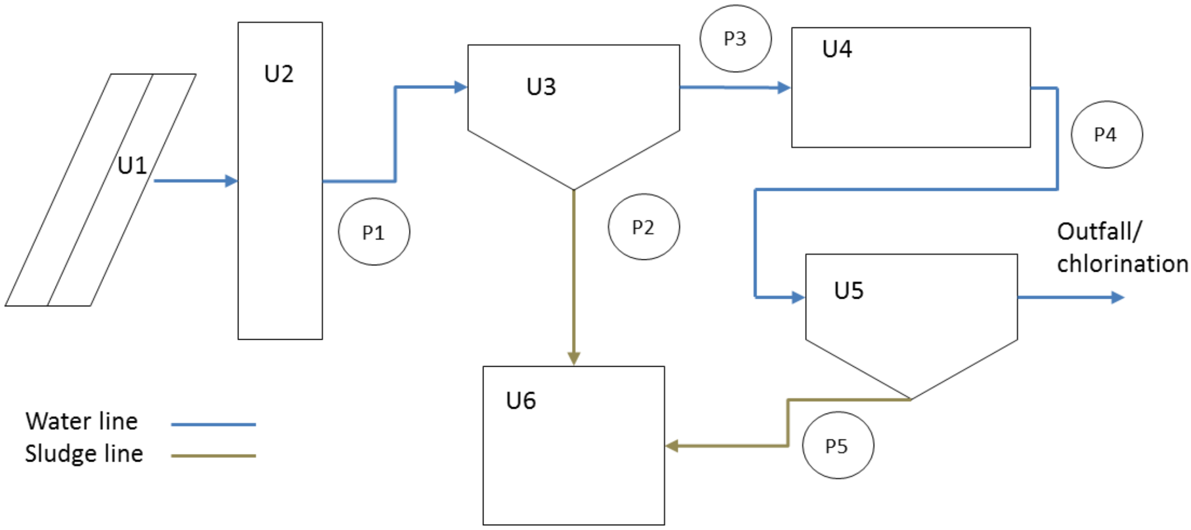


Figure 11: Complete mix activated sludge system schematic

Table 11: CMAS systems schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Primary settler
U5	Aerobic tank
U6	Secondary settler
U7	Sludge option
P 1 – 5	Indicates pumps

Model (BOD removal only)

Determine required influent characteristics

Assumptions

Initial SRT = 2 days

MLSS = 3,500 mg/l

bCOD = 1.6(BOD)

VSS = 0.85(TSS)

NH_3 = 0.75(TKN)

sCOD = 0.44($COD_{primary}$)

Peak to average TKN loading rate ratio = 1.5

Alkalinity = 200 mg/l as $CaCO_3$

Required equations for characteristics determination

nbCOD = COD-bCOD

nbsCOD = sCOD – 1.6sBOD

nbpCOD = TCOD – BCOD – nbsCOD

VSS_{COD} = (TCOD/sCOD)/VSS

nbVSS (non-biodegradable VSS) = (nbpCOD)/VSS_{COD}

iTSS (inert TSS) = TSS - VSS

Determine biomass production

Biomass production is determined by Eq. 9.4 and 9.5 (shown here for convenience) using coefficients presented in Table 11 and corrected for temperature.

$$A = \frac{QY_H(S_0 - S)}{1 + b_H(SRT)} \tag{9.4}$$

Where,

A = the concentration of heterotrophic biomass produced per day (g VSS/d)

Q = flowrate (m^3/d)

Y_H = yield coefficient (g VSS/g COD)

S_0 = concentration of bCOD (mg/l)

S = concentration of bCOD in effluent (mg/l) given by (Eq. E.1)

b_H = specific endogenous decay coefficient (g VSS/g VSS•d)

B represents the solids produced from cell debris and is given by (Eq. 9.5) [2].

$$B = \frac{(f_d)(b_H)QY_H(S_0 - S)SRT)}{1 + b_H(SRT)} \tag{9.5}$$

Where f_d is the fraction of biomass that remains as cell debris (g VSS/g biomass VSS depleted by decay).

$$S = \frac{K_s[1 + b_H(SRT)]}{SRT(\mu_{max} - b_H) - 1} \tag{E.1}$$

μ_{max} = maximum specific growth rate of heterotrophic bacteria (g VSS/g VSS•d)

Determine mass of VSS ($P_{X,VSS}$) produced per day (Eq. E.2) [2]

$$P_{X,VSS} = (A + B) + Q(nbVSS) \tag{E.2}$$

Determine mass of TSS ($P_{X,TSS}$) (Eq.E.3)

$$P_{X,TSS} = [(A + B)/0.85] + Q(nbVSS) + Q(TSS_o - VSS_o) \tag{E.3}$$

Where,

TSS_o = concentration of TSS in primary effluent

VSS_o = concentration of VSS in primary effluent

0.85 = VSS/TSS ratio

Equation E.1. is the controlling equation for the BOD removal. Using BOD kinetic coefficients, whereby Ks = 60 mg/l, Y = 0.6 g VSS/ g VSS oxidised, k = 6 d-1, μ_{max} = Yk, b_H= 0.1 g VSS/g VSS•d. The default SRT is 2 days, that is, the DST calculates the effluent BOD with an SRT of 2 days and through a series of iterations gradually increases the SRT until the effluent BOD (*S*) = BOD limit.

Table 2: Activated sludge design kinetic coefficients. (Adapted from [2])

Activated Sludge design kinetics @ 20°C				
Coefficient	Units	COD	NH_4	NO_2
μ_{max}	(g VSS/g VSS.d)	6	0.9	1
K_s, K_{NH4}, K_{NO2}	(mg/L)	8	0.5	0.2
Y	(g VSS/g substrate oxidised)	0.45	0.15	0.05
b	(g VSS/g VSS.d)	0.12	0.17	0.17
fd	unitless	0.15	0.15	0.15
K_{O2}	(mg/L)	0.2	0.5	0.9
θ Value				
μ_{max}	unitless	1.070	1.072	1.063
b	unitless	1.040	1.029	1.029
K_s, K_{NH4}, K_{NO2}	unitless	1.000	1.000	1.000

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BOD and ammonia removal

The procedure for calculating ammonia removal is the same as that for BOD removal. The main difference is the controlling bacteria. In these calculations it is the nitrifying organisms that control the SRT because of their reduced growth rate. The effluent NH_3 concentration (S_{NH_3}) is the controlling factor in determining the ammonia oxidising bacteria (AOB) growth rate, which determines the solids retention time. Additional steps involve firstly determining the design SRT by calculating the specific growth rate for AOB given by Eq. E.4 [2].

$$\mu_{AOB} = \mu_{max,AOB}\left[\frac{S_{NH_3}}{S_{NH_3}+K_{NH_3}}\right]\left[\frac{S_o}{S_o+K_{o,AOB}}\right]-b_{AOB} \quad \text{(E.4)}$$

Where,

μ_{AOB} = specific growth rate of ammonia oxidising bacteria (g VSS/g VSS.d)

$\mu_{max,AOB}$ = maximum specific growth rate of ammonia oxidising bacteria (g VSS/g VSS.d)

S_{NH_3} = ammonia concentration (mg/l)

K_{NH_3} = velocity half constant coefficient for NH_3(mg/l)

S_o = dissolved oxygen concentration (mg/l)

$K_{o,AOB}$ = half-velocity coefficient for DO for AOB (mg/l)

b_{AOB} = specific endogenous decay rate of AOB (g VSS/g VSS.d)

The SRT is then determined with Eq.E.5.

$$SRT = 1/\mu_{AOB} \tag{E.5}$$

The additional biomass from the nitrifying bacteria is given by Eq. E.6 (shown here for convenience).

$$C = \frac{QY_n(NO_x)}{1 + b_n(SRT)} \tag{E.6}$$

Where,

C= concentration of nitrogen biomass produced per day (g VSS/d)

NO_x = nitrogen concentration as a percentage of TKN (75% assumed for this study)

Y_n = yield coefficient for nitrifiers (g VSS/g COD)

b_n = endogenous decay coefficient for nitrifiers (g VSS/g VSS.d)

Total nitrogen removal

Total nitrogen removal with the CMAS system is achieved with a post-anoxic tank. The purpose of this choice is to examine the effect of the choice of the post-anoxic over the pre-anoxic process. The quantity of N removal (R_{NO_3}) is determined with Eq. E.7.

$$R_{NO_3} = Q(1 + R)(NO_x - N_{eff}) \tag{E.7}$$

Where,

NO_x= nitrogen concentration in aerobic effluent (mg/l)

N_{eff} = required effluent nitrogen concentration (mg/l)

Q = flowrate (m^3/d)

R = recycle ratio

The recycle ratio is given by Eq. E.8. [2]

$$R = \frac{X}{X_r - X} \tag{E.8}$$

Where, X_r and X are the return sludge and MLSS concentrations respectively (mg/l).

The tank volume is determined with Eq. E.9 [2].

$$R_{NO_3} = \left(\frac{1.42}{2.86}\right)(b_{H,T})(X_H)(V_{anox}) \tag{E.9}$$

Where,

$b_{H,T}$ = endogenous decay rate (g VSS/g VSS.d)

X_H= biomass concentration (g/m^3)

V_{anox} = post-anoxic tank volume (m^3)

Area calculations and assumptions

Aeration tank volume

The aeration tank volume is determined with the relationship given by Eq. E.11 [2].

$$(V) = \frac{(P_{X,TSS})SRT}{X_{TSS}} \tag{E.11}$$

Where,

X_{TSS} = MLSS (mg/l)

V = aeration tank volume (m^3)

The aeration tank surface area is then simply V/D, where D is the depth of the tank in meters. The default tank depth is 6 meters. This depth of the tank is determined through an iterative process that reduces tank depth until it meets the tank surface area to depth ratio.

Appendix E.2

Anoxic-oxic

System description

The AO system layout is presented in below (Figure 12). The unit process descriptors are presented in Table 12. The aerobic microbial activity, SRT calculations, volume and area calculations, and sludge volume calculations are as presented in the CMAS model with nitrification (Appendix E.1).

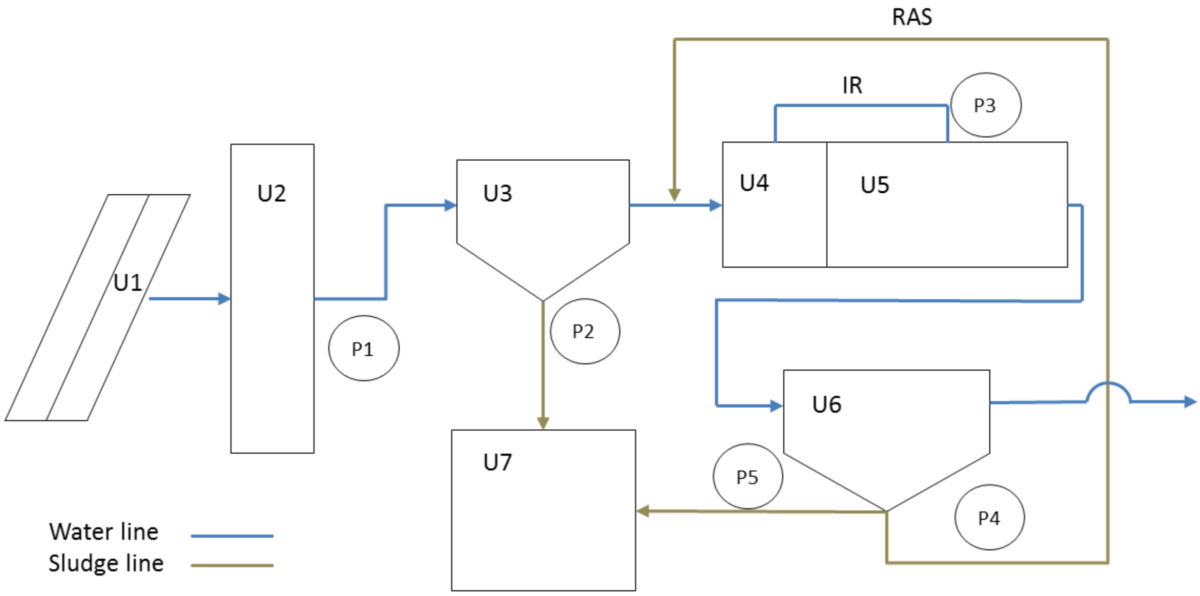

Figure 12: Anoxic-oxic system schematic

Table 3: Anoxic-oxic schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Primary settler
U4	Anoxic tank
U5	Aerobic tank
U6	Secondary settler
U7	Sludge option
P 1 – 5	Indicates pumps

Nitrate control

The nitrate effluent concentrations are controlled by the internal recycle ratio given by Eq. 13 [2].

$$IR = \frac{NO_x}{N_e} - 1.0 - R \tag{E.13}$$

Where,

IR is the internal recycle ratio

NO_x = nitrate produced in the aeration zone (mg/l)

N_e = effluent nitrate concentration (mg/l)

R = RAS ratio

To determine the mass of nitrate going to the anoxic tank the flowrate to the tank must be determined (Eq. E.14).

$$Q_{anoxic} = IRQ + RQ \tag{E.14}$$

$$\therefore NO_x = Q_{anoxic}N_e \tag{E.15}$$

The anoxic volume is calculated with Eq. E.16.

$$V_{nox} = (\tau Q) \tag{E.16}$$

Where:

τ = the anoxic tank hydraulic retention time (HRT) (d), assumed as 20% of the aerobic tank HRT.

Q = influent flowrate (m^3)

To determine the specific denitrification rate (SDNR) the biomass concentration must be first determined from Eq. E.17.

$$X_b = \left[\frac{Q(SRT)}{V_{aerobic}}\right]\left[\frac{Y_H(S_o - S)}{1 + b_H(SRT)}\right] \tag{E.17}$$

Where:

X_b= concentration of active biomass (g VSS/m^3)

S = aerobic tank effluent bCOD concentration (mg/l)

S_o = primary sedimentation effluent bCOD concentration (mg/l)

Y_H= yield coefficient (g VSS produced /g BOD removed) – (a value of 0.45 is assumed [2])

SRT = aerobic tank solid retention time (d)

b_H= specific endogenous decay coefficient for heterotrophic bacteria (g 0.088 VSS /g VSS.day)

$V_{aerobic}$ = volume of aerobic tank (m^3)

Q = influent flowrate (m^3/d)

The F/M ratio is then given by Eq. E.18.

$$\frac{F}{M} = \frac{QS_o}{V_{nox}(X_b)} \quad \text{(E.18)}$$

Where,

S_o = primary sedimentation effluent BOD concentration (mg/l)

The relationship between the SDNR and the F/M ratio is given by Eq. E.19 [2].

$$SDNR = b_o + b_1[ln(F/M)] \quad \text{(E.19)}$$

The fraction of rbCOD/bCOD is required to assess the biokinetic coefficients required for Eq. x. Linear and power law approximations of the relationship between the rbCOD fraction and the coefficients were developed from earlier work carried out by Grady et al. [27], and Stensel and Horne [28] (Figure 13).

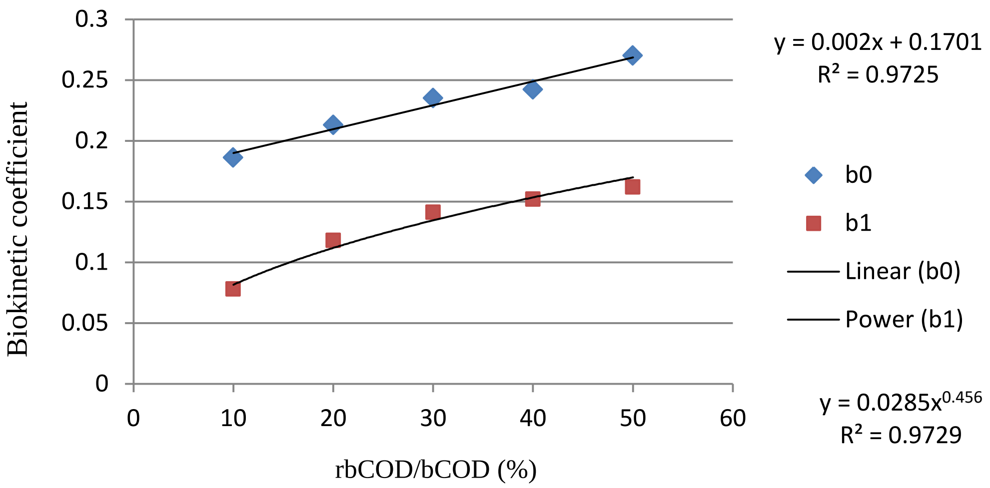

Figure 13: Specific denitrification rate biokinetic coefficient values as a function of rbCOD/bCOD percentage.

Apply temperature correction with Eq. E.20 ($\theta = 1.026$).

$$SDNR_T = SDNR_{20}\theta^{(T-20)} \qquad \text{(E.20)}$$

Determine overall SDNR using MLVSS (Eq. E.21).

$$SDNR = SDNR_b\left(\frac{MLVSS_b}{MLVSS}\right) \qquad \text{(E.21)}$$

Where;

$MLVSS_b$ = the active biomass concentration [X_b] calculated with Eq. E.17. (mg/l)

MLVSS = mixed liquor volatile suspended solids (mg/l), estimated as a percentage of MLSS – assumed MLVSS/MLSS = 0.8

Determine the nitrate that can be reduced $[NO_R]$ (g/day) with Eq. E.22.

$$NO_R = (V_{nox})(SDNR)(MLVSS) \qquad \text{(E.22)}$$

The value of NO_R can be compared with the NO_x feed value produced by Eq. E.15 and the anoxic HRT value can be adjusted iteratively until $NO_R = NO_x$.

Area calculations and assumptions

Additional area calculations for AO systems are limited to the anoxic tank. Aerobic tank calculations are presented in the CMAS with nitrification model (Appendix E.1).

Anoxic tank

The AO tank volume is determined by Eq. E.23 using the adjusted value for the anoxic hydraulic detention time.

$$V_{nox} = (\tau Q) \qquad \text{(E.23)}$$

The anoxic tank area calculation is then just volume/depth = area (m^2).

Energy

Aeration energy calculations are presented in Appendix D.2. There is a net reduction in required O_2 due to the release of O_2 during denitrification given by Eq. 24 [2].

$$Oxygen\ credit = \left(\frac{2.86\ g\ O_2}{g\ NO_3}\right)\left[\left(NO_x - NO_{eff}\right)g/m^3\right]\left(\frac{Q_{influent}}{day}\right)\left(1\,\frac{kg}{10^3}\,g\right) \qquad \text{(E.24)}$$

Additional pumping energy is required for the nitrate recycle line AO and AAO systems. This can range from 0.010 to 0.02 kWh/m^3 depending on flowrate and TN limit. As with the other system pumping lines, the minimum pipe diameter is 0.15 m and the velocity is maintained above 1.83 m/s to avoid solids deposition. For systems with low flowrates the pumping time is adjusted to allow higher intermediate flowrates (e.g. 12 h/day, 8 h/day). Nitrate recycle pipe lengths are estimated as the length of the aeration and anoxic tank plus 1 m. A value of 1 m is assumed for static head to account for plant slope. The RAS line length is estimated as the length of the aerobic and anoxic tank, plus the offset buffer x 2, plus half the length of the secondary settling tank, plus 1 m static head. A value of 5 kW/10^3 m3 is assumed for anoxic mixing [2].

Appendix E.3

Anaerobic anoxic oxic

System description

The anaerobic anoxic oxic (AAO) system facilitates enhanced biological phosphorus removal. With the exception of SBR systems and some configurations of extended aeration oxidation ditch systems, most systems can only achieve significant P removal with the addition of chemical such as alum or ferric chloride. The purpose of including the AAO system in the DST is to elucidate the life cycle costs of associated with the use of chemicals to precipitate phosphorus (both economic and environmental), by comparing the NPVs of the AO against the AAO system.

The AAO layout is presented in below (Figure 14). The unit process descriptors are presented in Table 12. Details of the mechanism of ammonia removal can be found in the AO system model (Appendix E.2). The aerobic microbial activity, SRT calculations, volume and area calculations, and sludge volume calculations are as presented in the CMAS model with nitrification (Appendix E.1).

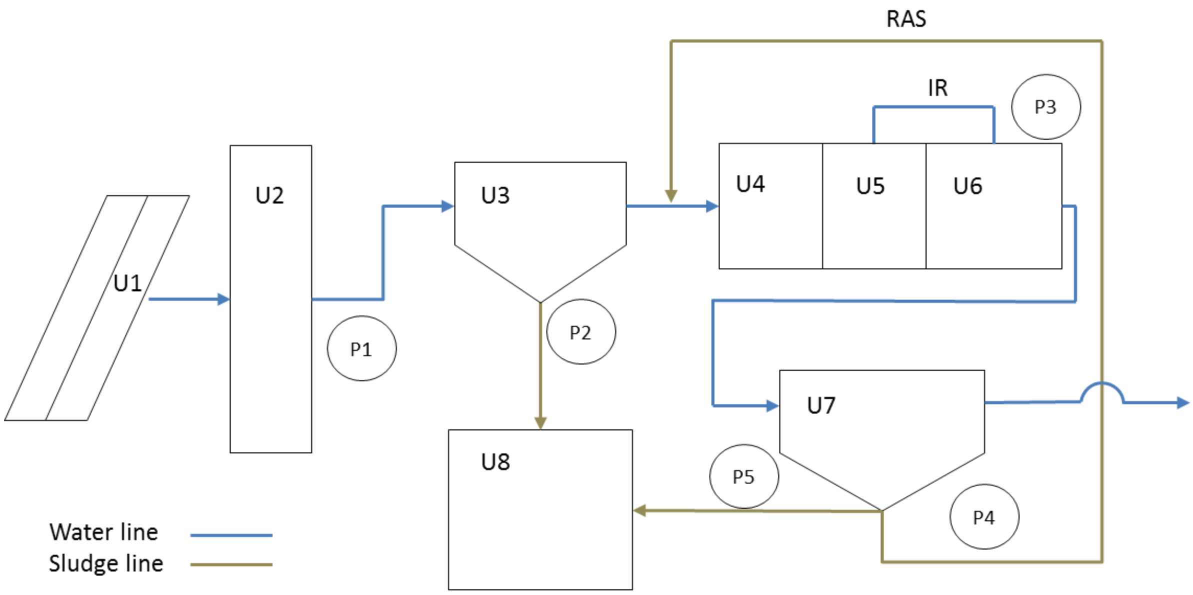

Figure 14: Anaerobic anoxic oxic system schematic

Table 14: Anaerobic anoxic oxic schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Primary settler
U4	Anaerobic tank
U5	Anoxic tank
U6	Aerobic tank
U7	Secondary settler
U8	Sludge option
P 1 – 5	Indicates pumps

Phosphorus removal

Assumptions

- The rbCOD fraction of COD ranges from 5 – 30% [29]. A value of 20 % is used here
- Phosphorus content of biomass = 0.015 g P/g biomass [2]
- Volatile fatty acids (VFA) concentrations of influent wastewater range from 28-69 mg CH_3 COOH/l [30]. An average value of 48 mg/l is assumed
- A value of 5 g rbCOD /g NO_3 is assumed [2]

Process

The rbCOD available for P removal is an important parameter in EBPR systems and must be determined in order to control P uptake. The mass of rbCOD in the influent wastewater is given by Eq. E.25

$$rbCOD_{mass} = Q(rbCOD_{conc.}) \tag{E.25}$$

Where,

Q = influent flowrate (m^3/d).

$rbCOD_{conc.}$ = concentration of rbCOD in the influent wastewater (mg/l).

The rbCOD consumed by nitrate is given by Eq. E.26.

$$rbCOD_{consumed} = \left(5\ g\frac{rbCOD}{NO_3}\right)(RQ)(NO_{3,eff}) \qquad \text{(E.26)}$$

Where,

R = the return activated sludge recycling ratio.

$NO_{3,eff}$ = the concentration of nitrate entering the anaerobic contact zone (mg/l).

Hence, the available rbCOD is given by Eq. E.27.

$$rbCOD_{available} = rbCOD_{mass} - rbCOD_{consumed} \qquad \text{(E.27)}$$

Figure 15 presents the rbCOD ratio as a function of the VFA/rbCOD ratio (adapted from [2]).

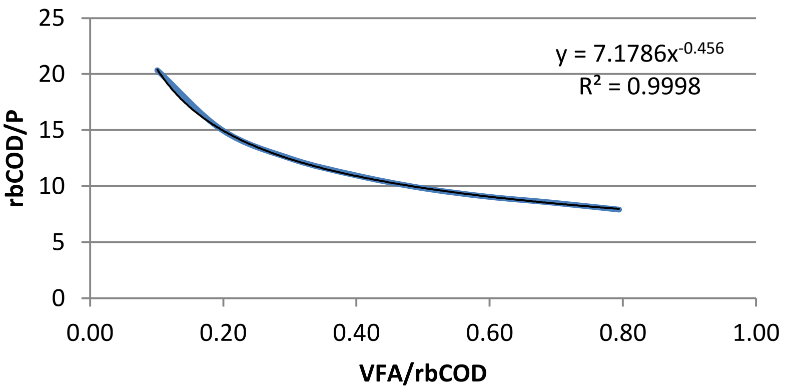

Figure 15: The rbCOD ratio as a function of VFA/rbCOD

The specific value for P removal can be determined by Eq. 28.

$$P\ removal = \frac{rbCOD}{rbCOD/P} \qquad \text{(E.28)}$$

The quantity of P removal for use in cell synthesis can be determined by calculating the mass of VSS produced per day. The quantity of P removal is then $0.015 \times (g\ VSS/d)$. The effluent P concentration can then be determined by Eq. E29.

$$P_{eff} = P_{in} - P_{EBPR} - P_{synthesis} \qquad \text{(E.29)}$$

Where,

P_{in} = primary effluent phosphorus concentration (mg/l)

P_{EBPR} = concentration of phosphorus removed by EBPR (mg/l)

$P_{synthesis}$ = concentration of phosphorus used for cell synthesis (mg/l)

Area calculations and assumptions

Area calculations for the AAO system are similar to those described in the AO system with the addition of the anaerobic tank. Aerobic and anoxic tank area calculations are presented in the CMAS with nitrification and AO model descriptions respectively (Appendix E.1 and E2). The other common area calculations are presented in the area section of the methodology chapter.

Anaerobic tank

The AAO tank volume is determined by Eq. E.30 using recommended HRT (τ) of between 0.5 and 1.5 h (average HRT_{an} value of 1 hour is assumed).

$$V_{nox} = (\tau Q) \tag{E.30}$$

The anaerobic tank area calculation is then just volume/depth = area (m^2).

Appendix E.4

Oxidation Ditch model – with nitrification

System description

The OD system layout is presented in below (Figure 12). The unit process descriptors are presented in Table 9. The OD type is the classic Pasveer design with horizontal shaft surface aerators (Figure 17). It was thought that the Pasveer model offered the advantage of being able to denitrify with the intermittent low and high DO zones. The trade-off with this system is the low OTE associated with the surface aeration systems. A rotary drum screen is used in place of primary treatment. As with all of the AS based systems, the removal rates are controlled by the SRT. The model presented here is based on the OD model presented by Davis [31].

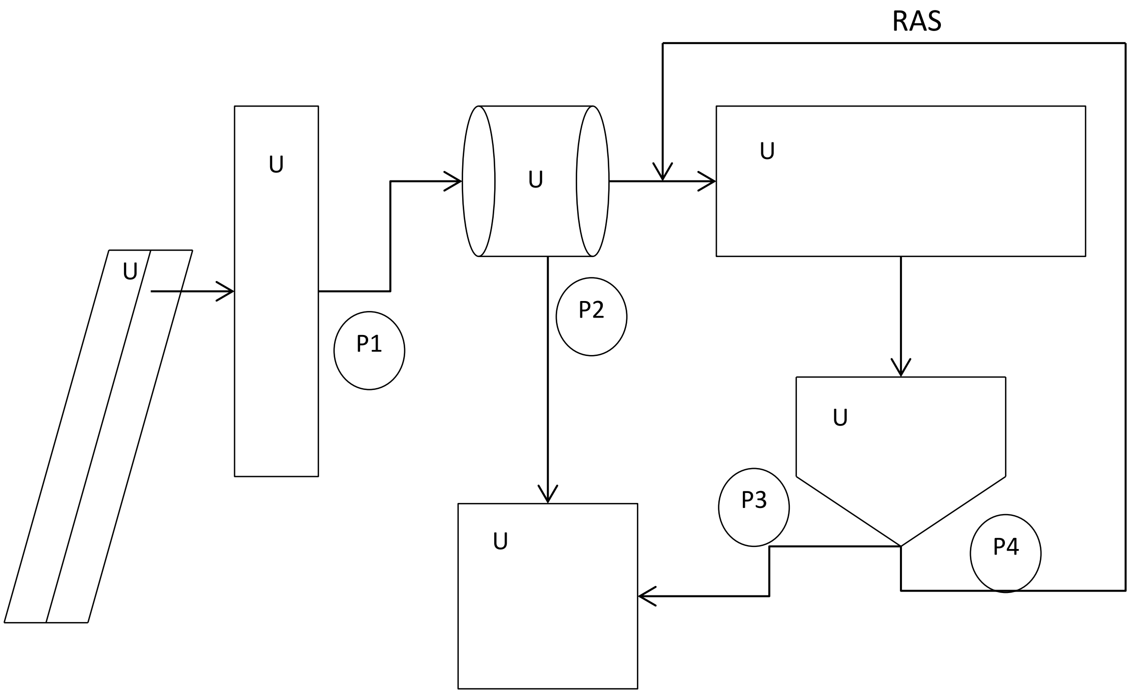

Figure 16: System schematic.

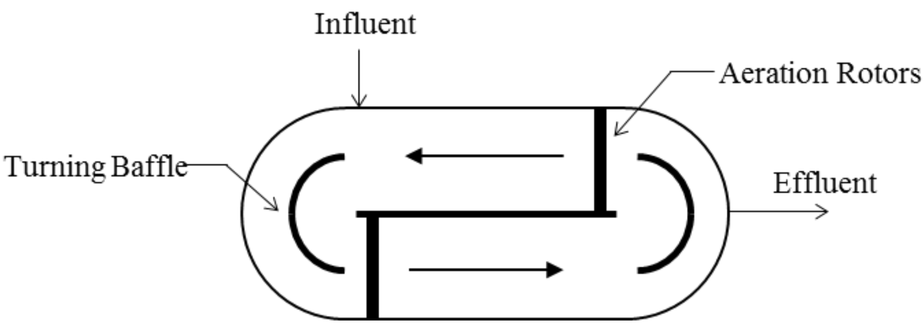

Figure 17: Pasveer type oxidation ditch.

Table 15: Oxidation ditch system schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Drum screen
U4	Oxidation ditch
U5	Secondary settler
U6	Sludge option
P 1 – 4	Indicates pumps

Tank volume calculation

Assumptions

Nitrification is the governing substrate

MLSS = 3,500 mg/l

MLVSS = (0.7) MLSS

VSS/TSS = 0.85

DO = 2.0 (mg/l))

FOS = 2.5

Tank depth = 4 m

Rotary drum screen removal rates:

- BOD 37%
- TSS 35%

1. Determine nitrification rates with Eq. E.31 using kinetic coefficients in Table 15 and correcting coefficients for temperature with Eq. E.32.

$$\mu_n = (\mu_{mn})\left(\frac{N}{K_n + N}\right)\left(\frac{DO}{K_o + DO}\right) - k_{dn} \quad \text{(E.31)}$$

Where,

μ_n = specific growth rate for nitrifying bacteria (g VSS/g VSS· d)

μ_{mn} = maximum specific growth rate for nitrifying bacteria (g VSS/g VSS· d)

N = nitrogen concentration (g /m^3)

K_n = velocity half constant (g NH_4/m^3)

DO = dissolved oxygen (g/m^3)

K_0 = half saturation constant (g/m^3)

k_{dn} =endogenous decay coefficient (g VSS/g VSS· d)

Y_n =yield coefficient (g VSS/g NH_4· d)

Table16: Nitrification kinetic coefficients

Coefficient	*Units*	*Typical values*	*Temperature correction factor* φ
μ_{mn}	g VSS/g VSS· d	0.75	1.07
K_n	g NH_4/m^3	0.74	1.053
Y_n	g VSS/g NH_4· d	0.12	
k_{dn}	g VSS/g VSS· d	0.08	1.04
K_0	g/m^3	0.5	

$$c_T = c_{20}\theta^{(T-20)} \quad \text{(E.32)}$$

Where,

c_T = coefficient at temperature T

c_{20} = coefficient at 20° C

θ = temperature correction factor

2. Determine mean cell residence time (θ_c) with Eq. E.33.

$$\frac{1}{\theta_{c\,min}} = \mu_n \tag{E.33}$$

3. Determine the BOD substrate utilisation rate (*U*) with Eq. 34 using kinetic coefficients from Table 16, and correct for temperature.

$$U = \left(\frac{1}{\theta_c} + k_d\right)\left(\frac{1}{Y}\right) \tag{E.34}$$

Table 17: Heterotrophic kinetic coefficients at 20 C

Coefficient	*Units*	*Typical values*	*Temperature correction factor φ*
μ_m	g VSS/g VSS· d	6.0	1.07
K_s	g bCOD/m^3	20	1.00
Y_n	g VSS/g bCOD· d	0.4	
k_d	g VSS/g VSS· d	0.12	1.04
f_d^a	g/g	0.15	

4. Determine the heterotrophic mean cell residence time (θ_{BOD}) with (Eq. E.35).

$$\theta_{BOD} = \frac{S_o - S}{UX} \tag{E.35}$$

Where,

S_o = influent bCOD (mg/l)

S = effluent bCOD (mg/l) (S=0, recommended)

X = MLSS (mg/l)

5. Determine substrate utilization rate (U) for nitrification using (Eq. 4) with coefficients adjusted for temperature (Table 14).

6. Determine the fraction of MLVSS (f_N) that is nitrifying organisms (Eq. E.36) [32].

$$f_N = \frac{0.16(NH_3 removed)}{0.6(BOD_5 removed) + 0.16(NH_3 removed)} \tag{E.36}$$

7. Determine the nitrosomas MLVSS (Eq. E.37)

$$MLVSS_N = f_N(0.7MLSS) \quad \text{(E.37)}$$

8. Determine θ_N with (Eq. E.35).
9. The OD volume can then be calculated with (Eq. E.38)

$$V = (Q)(\theta_N) \quad \text{(E.38)}$$

Area calculations

Oxidation ditch

The oxidation ditch area is given by (Eq. E.39).

$$OD\ tank\ area = V/D \quad \text{(E.39)}$$

Where,

V = the tank volume (m^3)

D = the tank depth (m)

Appendix E.5

Sequence batch reactor

System description

The sequence batch reactor system models are based on the calculation methods, assumptions and kinetic coefficients presented by Metcalf and Eddy [2]. The SBR layout is presented in below (Figure 18). As with the OD and EA systems, the SBR systems do not have a primary sedimentation stage. Considerations were given to the possibility of including an equalisation basin; however, it was determined that with the plant scale range in question, adequate wet well sizing may be able to provide enough of a buffer to accommodate hydraulic surge. The unit process descriptors are presented in Table 17.

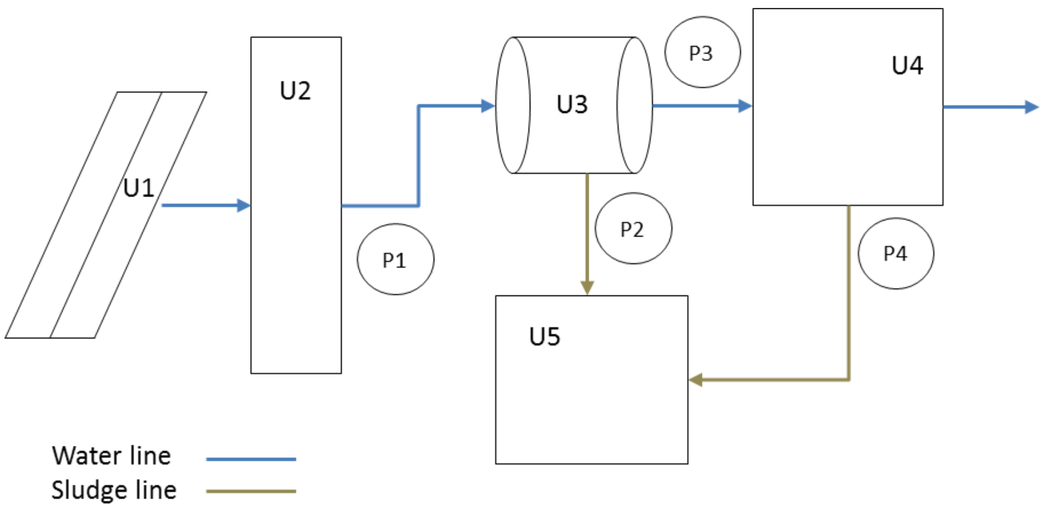

Figure 18: Sequence batch reactor system schematic.

Table 18: Sequence batch reactor schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Drum screen
U4	2 x SBR tank
U5	Sludge option
P 1 – 4	Indicates pumps

Process design

Assumptions

- Two SBR tanks to accommodate repair and maintenance.
- Tank height to diameter ratio ≤ 1.5.
- Decant depth = 20 % of tank depth.
- MLSS = 3500 mg/l
- MLVSS fraction of MLSS = 80%
- bCOD = 1.6BOD.
- SVI = 150 ml/g.
- Fine bubble diffusers.

Process

The wastewater characteristics that are required for system design are: bCOD, bsCOD, nbsCOD, nbpCOD, VSS, nbVSS and iTSS. Typical estimations of operation cycle intervals are firstly assumed, and then assessed for feasibility. Aeration times are adjusted for

- Fill time (t_F) = 3.0 h
- Aerate/react time (t_a) (BOD = 1.0 h) (NH_4 = 2.0 h)
- Settle time (t_s) (BOD = 0.5 h) (TN = 1.0 h)
- Decant time (t_d) = 0.5 h

While treatment of the wastewater is taking place in one tank, the other is filling. Therefore, the time to fill (t_f) is given by Eq. E.40, and the total cycle time is given by Eq. E.41.

$$t_f = t_a + t_s + t_d \qquad \text{(E.40)}$$

$$T_c = t_f + t_a + t_s + t_d \qquad \text{(E.41)}$$

The number of cycles per tank each day is (hours/day)/(hour/cycle), and the total number of cycles per day is then (tanks x cycles). The fill volume per cycle (V_F) can then be determined by Eq. E.42.

$$Fill\frac{volume}{cycle} = \frac{Q}{cycles/d} \qquad \text{(E.42)}$$

Where, Q is the flowrate in m^3/d.

The volume per tank (V_T) is given by Eq. E.43.

$$V_T = \frac{V_F/tank}{decant\ \%} \quad \text{(E.43)}$$

The HLR (τ) can then be determined by Eq. E.44.

$$\tau = \frac{(number\ of\ tanks)(V_T)(24h/d)}{Q} \quad \text{(E.44)}$$

The SRT is determined by the relationship between the biomass ($P_{X,TSS}$), SRT, MLSS, and tank volume (V) (Eq. E.45) [2].

$$P_{X,TSS}(SRT) = (V)(X_{MLSS}) \quad \text{(E.45)}$$

The $P_{X,TSS}(SRT)$ term can be calculated with Eq. E.46 using the kinetic coefficients presented in Appendix E.1.

$$P_{X,TSS}(SRT) = (A + B + C + D + \mathrm{E})(\mathrm{SRT}) \quad \text{(E.46)}$$

Where A represents the mass of sludge produced from heterotrophic biomass growth given by Eq. E.47 [2].

$$A = \frac{QY_H(S_0 - S)(SRT)}{1 + b_H(SRT)(0.85)} \quad \text{(E.47)}$$

Where,

Q = flowrate (m^3/d)

Y_H = yield coefficient (g VSS/g COD)

S_0 = concentration of bCOD (mg/l)

S = concentration of bCOD in effluent (mg/l)

b_H = specific endogenous decay coefficient (g VSS/g VSS•d)

B represents the solids produced from cell debris and are given by Eq. E.48 [2].

$$B = \frac{(f_d)(b_H)QY_H(S_o - S)(SRT)^2}{1 + b_H(SRT)(0.85)} \quad (E.48)$$

Where f_d is the fraction of biomass that remains as cell debris (g VSS/g biomass VSS depleted by decay).

C represents the nitrifying bacteria mass and is given by Eq. E.49 [2].

$$C = \frac{QY_n(NO_x)(SRT)}{1 + b_n(SRT)(0.85)} \quad (E.49)$$

Where NO_x = the nitrogen concentration.

D represents the non-biodegradable VSS in the influent given by Eq. E.50 [2].

$$D = Q(nbVSS)(SRT) \quad (E.50)$$

E represents the influent inert solids given by Eq. E.51 [2].

$$E = Q(TSS_o - VSS_0)(SRT) \quad (E.51)$$

An iteration process is then used to determine the SRT value. The MLVSS concentration can be determined with expressions A, B, C, and D, Eq. 9.3 – 9.7, Chapter 9. The amount of NO_x can be determined from the nitrogen balance given by Eq. E.52.

$$NO_x = TKN_o - N_e - 0.12P_x/Q \quad (E.52)$$

Where,

NO_x= the amount of oxidised ammonium (mg/l)

TKN_o =influent Kjeldahl nitrogen (mg/l)

N_e = desired effluent nitrogen concentration (mg/l)

P_x= biomass (kg/d) (determined from parts A, B, C, Eq. 9.3 – 9.7)

Q = flowrate (m^3/d)

To determine the required reaction (aeration) time, the oxidisable nitrogen must be determined. The total oxidisable N at the start of the cycle is given by Eq. E.53.

$$Total\ N = [N_e(V - V_F)] + [V_F(NO_x)] \quad (E.53)$$

Where,

V = tank volume (m^3)

V_r = the volume of fluid remaining in one tank after decanting (m^3)

V_F = the fill volume (m^3)

N_e = the disired effluent NH_4 concentration (mg/l)

Equation E.54 is used to determine the aeration time using the kinetic coefficients from Table 2.

$$K_{NH_4} ln\left(\frac{N_o}{N_t}\right) + N_o - N_t = X_n\left(\frac{\mu_{max,AOB}}{Y_n}\right)\left(\frac{S_o}{K_{o,AOB}+S_o}\right)t \tag{E.54}$$

Where,

K_{NH_4}= the velocity half constant for NH_4

N_t = NH_4 concentration at time t, (mg/l)

$\mu_{max,AOB}$ = the maximum substrate utilisation rate for ammonia oxidising bacteria (AOB)

S_o = dissolved oxygen concentration (mg/l)

Y_n = nitrifier yield coefficient

X_n= nitrifier concentration (mg/l) (given by Eq. E.55)

$$X_n = \frac{QY_n(NO_x)(SRT)}{1 + b_n(SRT)(V)} \tag{E.55}$$

Table 3: Activated sludge design kinetic coefficients. (Adapted from [2])

Energy

The energy sinks unique to the SBR system are limited to the fill time pumping. It is assumed that the static pumping height is equal to the tank height. The density of the fluid is assumed to be similar to that of the screened fluid (1010 kg/m^3). The results of test runs varying flowrate from 100 - 1,000 m^3/d found that the fill pumping energy varied from 0.031 - 0.035 kWh/m^3. Therefore, it was assumed that a constant average value of 0.033 kWh/m^3 would suffice. It is assumed that decanting occurs without pumping. The aeration energy is calculated as per the aeration model presented in Chapter 8. Mixing energy is assumed 0.005kW/m^3.

Appendix E.6

Extended Aeration

The EA system layout is presented below in Figure 19. The system schematic legend is presented in Table 19. Calculations for sludge production, oxygen demand, chemical demand and aeration tank volume are as presented in the CMAS nitrification model. The EA design specifications assumed for the study are outlined in Table 20. It is assumed that the EA system is only considered for scenarios that require ammonia removal. The EA system uses a fine drum screen in place of primary settling. Phosphorus removal is achieved with ferric chloride addition. Total nitrogen removal is achieved though cyclical aeration. An equalisation basin is provided to regulate flow, but also to provide additional capacity to facilitate influent flow during anoxic periods for denitrification.

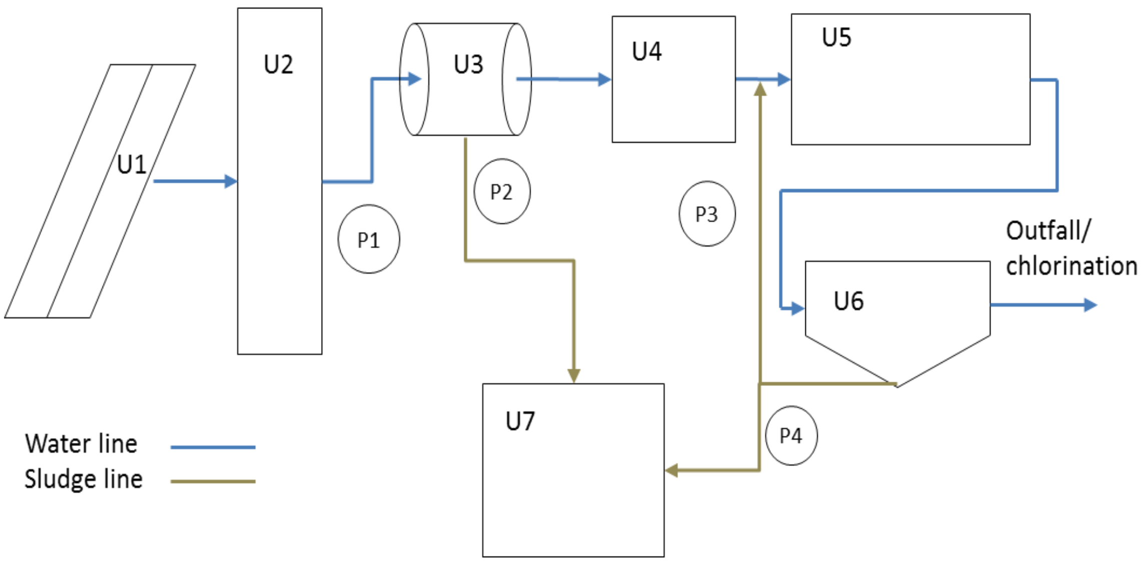

Figure 19: Extended aeration system layout

Table 20: Schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Drum screen
U4	Equalisation tank
U5	Aeration tank
U6	Secondary settler
U7	Sludge option

P 1 – 4	Indicates pumps

Table 41: Extended aeration design parameters

Parameter	*Value*
Design SRT (days)	20
HRT (hours)	25
MLSS (mg/l)	4,500
$MLSS_{return}$ (mg/l)	8,000
RAS (%)	128

Total nitrogen reduction

Total nitrogen removal in the EA model is achieved through a cyclical aeration process whereby the aeration tank is operated in anoxic conditions for a number of hours. A modified version of Stensel's method to calculate the average specific denitrification rate (SDNR) developed by [2] can be used to control TN effluent concentration and determine the anoxic time required (Eq. E.56).

$$SDNR = \frac{0.175A_n}{Y_{net}(SRT)} \tag{E.56}$$

Where,

A_n= net oxygen utilisation coefficient (g O2/g bCOD) (Eq. E.57)

Y_{net}= net yield (g VSS/g bCOD) (Eq. E.58)

$$A_n = 1.0 - 1.42Y_H + \frac{1.42(b_h)(Y_H)(SRT)}{1 + b_h(SRT)} \tag{E.57}$$

Where,

b_h = specific endogenous decay coefficient (g VSS/g VSS.d) (Table 10)

Y_H = heterotrophic yield coefficient (g VSS/g VSS.d) (Table 10)

$$Y_{net} = \frac{Y_H}{1 + b_h(SRT)} \tag{E.58}$$

Process

- Determine SDNR from Eq. E.55 – 58
- Determine biomass concentration in the mixed liquor
- Determine amount of NO_3 to be removed
- Determine NO_3 removal rate (g NO_3/d) with Eq. E.59

$$Anoxic\ NO_r = (SDNR)(X)(V) \tag{E.59}$$

Where,

X = biomass concentration (mg/l)

V = tank volume (m^3)

Appendix E.7

Integrated fixed-film activated sludge

The equivalent MLSS approach as presented in Metcalf and Eddy [2] is used to model the IFAS system. This method assumes a nominal MLSS value for the media fill fraction of the aeration tank, and then following the same procedure for the AO system design. The equivalent MLSS is determined with Eq. E.60 [2]. It is assumed that phosphorus removal is achieved with ferric chloride addition. System design parameters are presented in Table 21.

$$MLSS_{equiv.} = (V_{med})(X_{med.}) + (V_{sus.})(X_{sus.}) \quad \text{(E.60)}$$

Where,

$V_{med.}$ = media fraction of aeration tank volume

$X_{med.}$ = media volume solids concentration (g/m^3)

$V_{sus.}$ = activated sludge fraction of aeration tank volume

$X_{sus.}$= activated sludge volume solids concentration (g/m^3)

Table 22: Integrated fixed-film assumed design parameters

Parameter	***Value***
Design SRT (days)	Variable
HRT (hours)	variable
$X_{med.}$ (g/m^3)	18,000 [2]
$X_{sus.}$ (g/m^3)	3,500
$MLSS_{return}$ (mg/l)	8,000
DO concentration (mg/l)	4
Media fraction	0.4
Growth media specific surface area (m^2/m^3)	500

Appendix E.8

Trickling filter model

System description

The TF system layout is presented in below (Figure 20). The system schematic legend is presented in Table 22. The TF type is the single stage bio-tower design with rotating distributor arms. The bio-tower design was chosen for the higher organic loading rates that can be achieved with plastic media and reduced surface area. It is assumed that a single bio-tower will suffice for the scale range being considered.

The model provides the mechanism of BOD removal and the control parameters. It also includes TF bio-tower volume and surface area calculations, and energy use. Denitrification is achieved through a post anoxic process as presented in Appendix E.1 for the CMAS system. Phosphorus removal is achieved with ferric chloride addition.

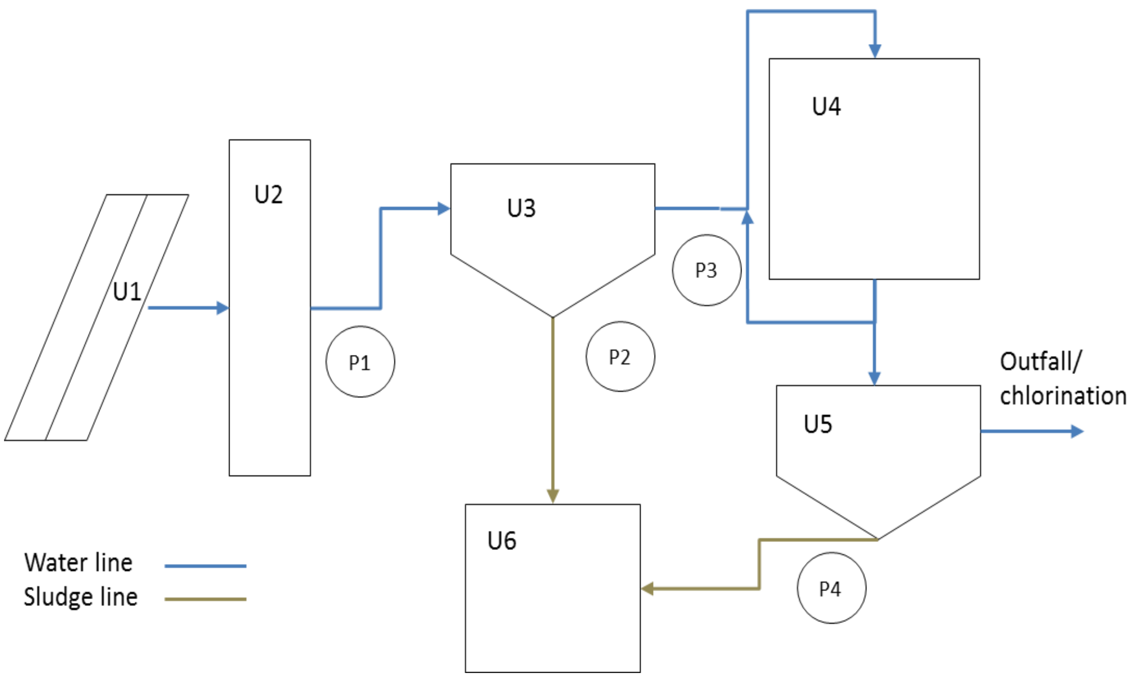

Figure 20: Trickling filter system schematic.

Table 23: Schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Primary settler
U4	Bio-tower
U5	Secondary settler
U6	Sludge option
P 1 – 4	Indicates pumps

Assumptions

Packing specific surface area = 150 m^2/m^3

Packing coefficient (*n*) = 0.5

Recirculation ratio (R) = 1.0

VSS/TSS = 0.85

sBOD/BOD = 0.75

BOD/UBOD = 1.6

Minimum wetting rate = 0.25 $L/m^2 \cdot s$

Hydrostatic pressures distributor system

FOS = 2.5

BOD removal

The methods for controlling BOD removal were based on the work of Velz [33] who determined that that BOD removal was related to the hydraulic loading rate (HLR). Building on the work of Velz, Schulze [34] proposed the contact time, HLR, and filter depth relationships given below (Eq. E.61 and E.62).

$$t = \frac{CD}{THL^n} \tag{E.61}$$

$$THL = \frac{Q(1+R)}{A} = (1+R)q \tag{E.62}$$

Where,

THL = total hydraulic loading rate ($m^3/m^2 \cdot d$)

t = liquid contact time (d)

C = packing constant

D = packing depth (m)

n = packing coefficient

R = recycling ratio

q = hydraulic loading rate ($m^3/m^2 \cdot d$)

The change in the filter BOD concentration w.r.t time is given by (Eq. E.63) [2].

$$\frac{dS}{dt} = -kS \tag{E.63}$$

Where,

k = rate constant

S = BOD concentration at time t (mg/l)

Schultz developed a relationship between influent and effluent BOD concentrations (Eq. E.64), which was later adapted by Germain [35] to account for plastic packing (Eq. E.65).

$$\frac{S_e}{S_i} = exp\frac{-kD}{(THL)^n} \tag{E.64}$$

Where,

S_e = effluent BOD concentration (mg/l)

S_i = influent BOD concentration (mg/l)

$$\frac{S_e}{S_i} = exp\left\{\frac{-kD}{[(1+R)q]^n}\right\} \tag{E.65}$$

Temperature correction for k is given by (Eq. E.66) [2].

$$k_T = k_{20}(1.035)^{T-20} \quad \text{(E.66)}$$

A further modification of the Velz equation (Eq. E.67) relates the BOD effluent concentrations to primary effluent BOD concentrations, temperature, recirculation ratio and packing constants. This is the governing equation that controls BOD in the TF model.

$$S_e = \frac{S_o}{(R+1)exp\left\{\frac{-k_{20}A_s D\theta^{T-20}}{[(1+R)q]^n}\right\}} \quad \text{(E.67)}$$

Where:

S_o = primary effluent BOD concentration (mg/l)

k_{20} = the filter treatability constant at 20° [(L/s)$^{0.5}$/m]

A_s = clean packing specific surface area (m^2/m^3)

Values for the term $k_{20}A_s$ in Eq. 6 have been determined for different types of wastewater by a number of pilot plant studies conducted by the Dow Chemicals Company. For domestic wastewater a value of 0.21 (L/s)$^{0.5}$/m^2 has been determined [2], and is used in this study.

Ammonia removal

Ammonia removal calculations and control are based on the model proposed by Pearce and Edwards [36] (Eq. E.68).

$$NH_{4,effluent} = 20.81\left(BOD_{loading}\right)^{1.03}(NH_{4,loading})^{1.52}(Iv)^{-0.36}(T)^{-0.12} \quad \text{(E.68)}$$

Where:

$NH_{4,effluent}$ = effluent ammonia concentration (mg/l)

$BOD_{loading}$ = specific BOD surface loading rate (g/m^2·d)

$NH_{4,loading}$ = specific surface area loading rate for ammonia (g/m^2·d)

Iv = specific hydraulic surface loading rate (L/ m^2·d)

T = effluent temperature °C

The specific surface area loading rate for ammonia is determined from the specific nitrification rate (R_n) (Eq. E.69), and the TKN influent concentration. The TKN/NH_4 is assumed to be 4/3.

$$R_n = \frac{(TKN\ removal\ percentage)(Q)(TKNconcentration)}{media\ surface\ area} \quad \text{(E.69)}$$

$$NH_{4,loading} = \frac{R_n}{TKN\ removal\ percentage} \quad \text{(E.70)}$$

The specific hydraulic surface loading rate (I_v) is given by (Eq. E.71)

$$I_v = \frac{Q}{surface\ area} \quad \text{(E.71)}$$

Sludge production

Trickling filter sludge production is determined with Eq. E.72.

$$X_{TF} = \frac{Y_H(S_o - S)}{1 + b_H(SRT)} \quad \text{(E.72)}$$

Where,

S = trickling filter effluent BOD concentration (mg/l)

S_o = primary sedimentation effluent BOD concentration (mg/l)

X_{TF}= concentration of volatile suspended solids (g VSS/m^3)

Y_H= yield coefficient (g VSS produced /g BOD removed) – (value of 0.6 is assumed [2])

SRT = solid retention time (d)

b_H= specific endogenous decay coefficient for heterotrophic bacteria (g VSS /g VSS.day)

To determine S, the sBOD and pBOD must be calculated. The sBOD concentration is given by (Eq. E.73) [2].

$$sBOD = BOD_{TF,effluent} - \left(\frac{0.6g\ BOD}{g\ UBOD}\right)\left(\frac{1.42\ g\ UBOD}{g\ VSS}\right)\left(\frac{0.85\ g\ VSS}{g\ TSS}\right)\left(\frac{30\ g\ TSS}{m^3}\right) \quad \text{(E.73)}$$

The pBOD influent concentration is given by (Eq. E.74).

$$pBOD_{in} = BOD_{in} - sBOD_{in} \quad \text{(E.74)}$$

The percentage of pBOD removed by the trickling filter is determined from Figure 21.

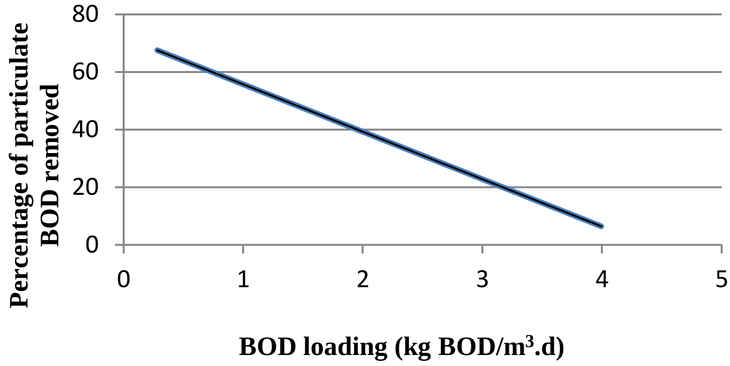

Figure 21: Percentage of pBOD removed as a function of the BOD loading rate (adapted from [2])

The total TF BOD effluent concentration is given by (Eq. E.75).

$$BOD_{effluent} = sBOD_{effluent} + pBOD_{effluent} \tag{E.75}$$

The TF SRT is calculated as a function of the organic loading rate (Figure 22).

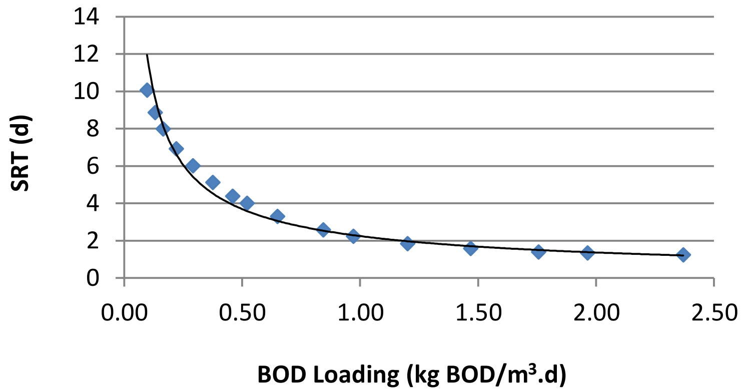

Figure 22: Trickling filter SRT as a function of the BOD loading rate (adapted from [2])

The total sludge production is then given by (Eq. E.76).

$$Sludge\ production = \left(X_{TF,VSS} + X_{iTSS}\right) * Q \tag{E.76}$$

Where,

X_{iTSS} = concentration of inert total suspended solids in TF effluent (assumed iTSS/TSS = 0.1)

Area calculations and assumptions

Trickling filter

The TF tank volume is determined with the relationship between volume, organic loading rate, flowrate, and substrate concentration as shown below (Eq. E.77) [2].

$$L = \frac{QS_o}{V} \tag{E.77}$$

Where:

L – organic loading rate (kg BOD/m^3.day)

Q = flowrate (m^3/day)

V = tank volume (m^3)

S_o= primary effluent BOD concentration (mg/l)

The required organic loading rate can be determined from Figure 23, where the loading rate is given as a function of the desired BOD removal percentage for plastic packing trickling filters [2]. It should be noted that the loading rates presented here are valid only up to 92% BOD removal.

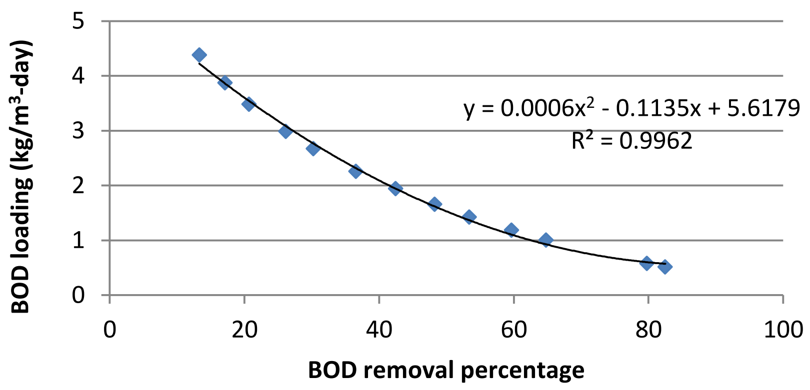

Figure 23: Trickling filter performance at 20 C (adapted from [2])

The TF surface area is then given by Eq. E.78.

$$TF\ tank\ area = V/D \tag{E.78}$$

Where, D is the depth of the bio-tower in meters.

Energy

Because the main energy sink in TF systems is the distribution pumping, a specific discussion and presentation of the calculation method is presented here. Headloss calculations are based on the model presented in Appendix D. The most significant parameter is the static head, which, for the 6 m bio-tower is estimated at 7 m so as to include distributor arm and underground channel clearance. There is little specific information regarding the distributor arm headloss value. Reports in the literature range from 0.6 to 2.5 m [37]. No data could be found with reference to m/flowrate or m/m of distributor arm length. A value of 1.5 m has been used here. It is recommended that pipe diameters should be adequately sized so as not to fall below 1.83 m/s fluid velocity at minimum flowrate, thus, the velocity has been set as a constraint during pumping energy calculations. The pipe roughness value ε, is considered to be similar to plastic ($0.0015 \pm 50\%$) [20].

Values reported in the literature for minimum wetting rates range from 0.25-0.5 $L/m^2{\cdot}s$ depending on the type of packing material used. The wetting rates are less significant for BOD removal only systems where the required HLRs are generally greater than required wetting rate. However, systems that require nitrification can have much lower HLRs and may require recirculation to maintain wetting rates. The recommended wetting rates for the plastic media here is 0.25 $L/m^2{\cdot}s$.

Appendix E.9

Rotating Biological Contactors

The RBC system layout is presented below (Figure 24), with schematic legend (Table 23).

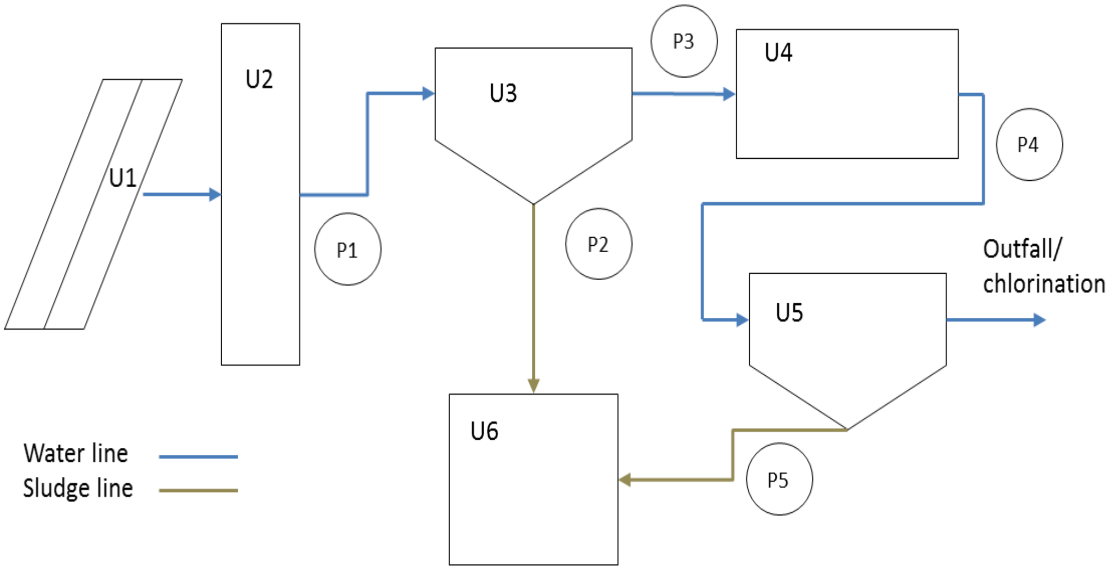

Figure 24: Rotating biological contactor system layout

Table 24: Rotating biological contactor schematic legend

Unit number	*Unit Process*
U1	Bar screen
U2	Wet well
U3	Primary settler
U4	RBC units
U6	Secondary settler
U7	Sludge option
P 1 – 5	Indicates pumps

Review of RBC system parameters

Operating parameters and other factors effecting RBC performance

Rotating biological contactor process performance is determined by a number of operating and design parameters including speed of disc rotation, tank volume-to surface area ratio, variation in hydraulic and organic load, and staging. Environmental factors such as temperature also effect performance and must be considered during the design phase so as to provide adequate sizing. The following is a discussion of some of the more significant factors influencing performance.

Variations in hydraulic and organic load

In most wastewater treatment systems a shock increase in hydraulic load will result in some level of reduction in removal efficiency. Suspended growth activated sludge systems may be better equipped to handle the immediate effects of the increase in hydraulic load but may suffer a degree of microbial wash-out not associated with attached growth systems, specifically in relation to AOBs. The effect of variation in flow is more severe on nitrifying bacteria than it is on heterotrophic microorganisms. There are two main reasons for this; firstly, AOBs have long generation times, and secondly because nitrifiers do not store substrate during delays in metabolism. In the study carried out by Antonie et al. [38] to examine the effects of changes in flowrate it was found that an increase in hydraulic load resulted in an almost proportional reduction in BOD removal efficiency, but further variations were evident with changes in influent BOD concentration. It was not until Stover and Kincannons study of RBC performance [39] that the importance of organic loading was realised. Organic loading combines hydraulic load and organic concentration (kg/m^3.d). Hydraulic load on its own is not sufficient to measure treatment performance because a high hydraulic load and low organic concentration can be treated to the same level as a low hydraulic load with a high organic concentration once the retention time is greater than 1 hour [40].

RBC manufacturers recommend using average flow conditions for sizing if the peak-to-average flow ratio is 2.5 or less [41]. If the peak-flow ratio is greater than 2.5 it is recommended to use the peak flow values. The additional CAPEX, OPEX and energy cost associated with increasing the size of the plant to over 2.5 times the average capacity can be significant. One option to mitigate the effects of shock loading without unnecessary oversizing of the system is to install an equalisation tank. The additional CAPEX will be small in relation to the savings made in OPEX over the lifetime of the system

Disc rotational speed

Early experimental work on RBC systems for wastewater treatment concluded that the optimum rotational speed for RBC discs was around 0.3 m/s [42], and subsequent RBC studies used this value as the benchmark without consideration for disc size or loading. In an experiment conducted by Bintanja et al. [43] the relationship between oxygen transfer and disc rotational speed was determined (Eq.E.79).

$$InK_L = a/ln\omega + b \tag{E.79}$$

Where K_L is the oxygen transfer efficiency coefficient, ω is the rotational speed in rev/m, a and b are empirical coefficients. It was suggested that because removal efficiency is related to oxygenation, rational disc speed could now be linked to removal efficiency. Friedman et al. [42] built on this work to examine changes in hydraulic loading and soluble COD concentration with variations in rotational speed. In the experiment, the rotational speed varied from 6 to 30 rev/min. It was found that regardless of variations in hydraulic load or influent concentration, there was an inverse relationship between disc rotational speed and substrate removal efficiency.

RBC Model

Assumptions

- It is assumed that because of the plant scale range in question, a single-train system is sufficient. It is also assumed that the train is a baffled configuration and the axis of rotation is parallel to the direction of flow (Figure 25).
- Disc diameter = 3 m
- Disc area per shaft length (BOD) = 1220 m^2/m
- Disc area per shaft length (N) = 1824 m^2/m
- Organic loading limit = 15 g sBOD/m^2.d
- NH_3 removal flux = 1.5 g sBOD/m^2.d
- NO_3 removal flux = 4.4 g sBOD/m^2.d

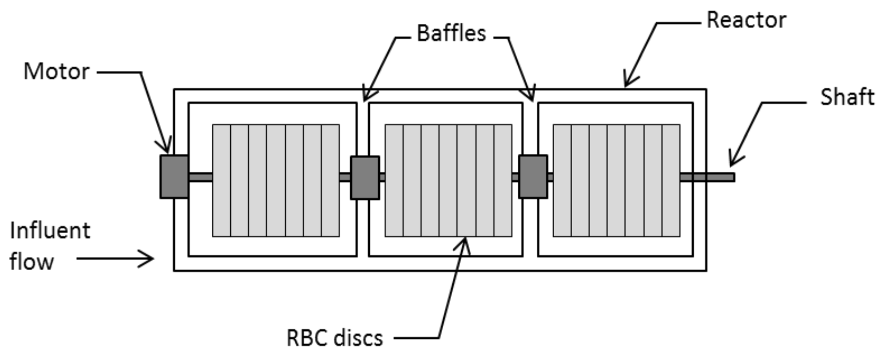

Figure 25: Rotating biological contactor configuration

The main controlling factor in the design of the RBC is the soluble BOD load per unit time per unit surface area of disc (g sBOD/m^2-d). If the sBOD value is not known a default value of sBOD = (0.5 BOD) is provided. There are varied reports in the literature for the optimal specific organic loading limit (g sBOD/m^2.d). Brenner et al. [41] have reported values between 12.32 and 12.69 g sBOD/m^2.d, while others have reported less conservative values of 18 – 34 g sBOD/m^2.d for hydraulic loads of 0.04 to 0.16 m^3/m^2.d to reach a target effluent of less than 30 mg BOD/l [40]. The sBOD loading in the first stage of the RBC train should not exceed 15 g sBOD/m^2.d [44]. The design model used for BOD removal in each stage is the second order model developed by Opatken [45] and later converted to SI units by Grady et al. [46] (E.q. E.80).

$$S_n = \frac{-1 + \sqrt{1 + (4)(0.00974)(A_s/Q)S_{n-1}}}{(2)(0.00974)(A_s/Q)} \tag{E.80}$$

Where,

S_n = sBOD concentration for stage *n* (mg/l)

A_s = disk surface area on stage *n* (m^2)

Q = flowrate (m^3/d).

Temperature correction for BOD removal only is determined with the polynomial developed from Figure 26 (adapted from [41]).

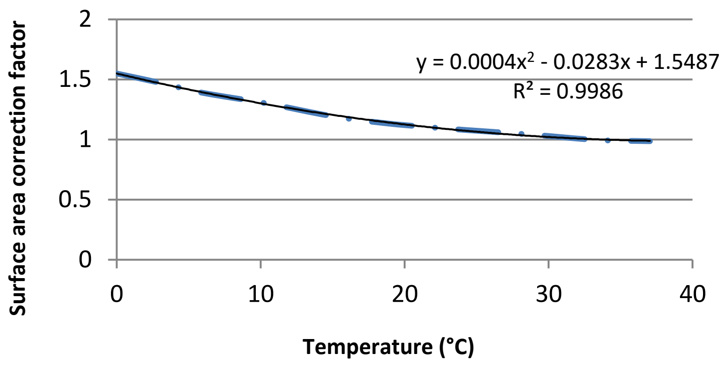

Figure 26: Surface area correction factor for RBC BOD removal

Nitrification

Weng et al. [47] concluded that of all of the process performance parameters such as disc rotational speed, submergence depth and influent flow rate, the only controlling factor in nitrification was the NH_3-N loading per unit time per disk surface area (g NH_3-N/m^2.d). It has been recommended that the loading should not exceed 1.5 g NH_3/m^2.d [41]. The rate of nitrification in each of the stages is calculated based on the relationship developed by Pano and Middlebrooks [48]. This relates the rate of nitrification to the concentration of sBOD in each stage (E.q. E.81)

$$\mathrm{F}_{r_n} = 1.00 - 0.1sBOD \tag{E.81}$$

Where F_{r_n} is the fraction of nitrification that can take place before the concentration of sBOD begins to impede the nitrifying reaction rate.

Temperature correction for nitrification is determined with as an average of values gathered from two separate empirical based studies (Figure 27) (adapted from [41]).

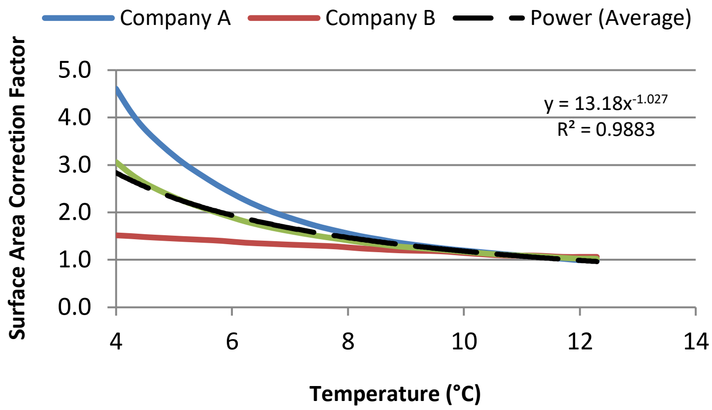

Figure 27: Temperature correction for RBC systems with nitrification requirements

Phosphorous removal

Phosphorus removal with attached growth systems has had limited success. It is possible to achieve phosphorous removal with RBC systems. Disc levels may be lowered to a point of full submergence to create an anaerobic zone in a stage. Removal rates of 70% in RBC systems have been reported by Hassard et al. [49], but also reported difficulties controlling oxic and anaerobic conditions. It has been assumed that this level of control may not be practical for small plants, and that chemical reduction is the preferred method of phosphorus removal.

Energy

RBC energy consumption varies significantly depending on whether or not nitrification is required. This is due to the large media surface area requirement s for nitrification. Energy calculations are based on a combination of empirical and first principle modelling. RBC shaft power is based on data collected by Gilbert et al. [50]. Power data was gathered from a number of RBC facilities and presented in terms of media surface area per shaft, and power per shaft. An average power value per media surface area was calculated to be 18.44 x 10^{-3} kW/m^2 disc, (n=105, S = 28%). This value accounts for both standard media density and high media density discs.

Area calculations

The RBC surface area calculations are based on the disc diameter and the required growth media surface area. The required growth media surface area will dictate the shaft length required. The calculation is then just the disc diameter times the shaft length. A simplification made in the area calculations is that the shaft lengths are tailored to the exact requirements. In reality, shafts come in standards lengths ranging from 1.52 – 7.62 m [44]. The exact discretions will depend on the RBC manufacturer.

Appendix E.10

Constructed Wetlands

Constructed wetlands system modelling is based on the findings from the studies conducted by Vymazal [51],[52] and Gkika et al. [4].

Capital expenditure

Linear regression models were developed from the CAPEX data compiled by Gkika et al. [4] for a survey of 7 CW treatment plants ranging in scale from 540 – 1200 PE (Figure 28). The CAPEX includes the cost of land (assumed to be similar to Ireland), civil works, construction, piping, electrical, mechanical and engineering.

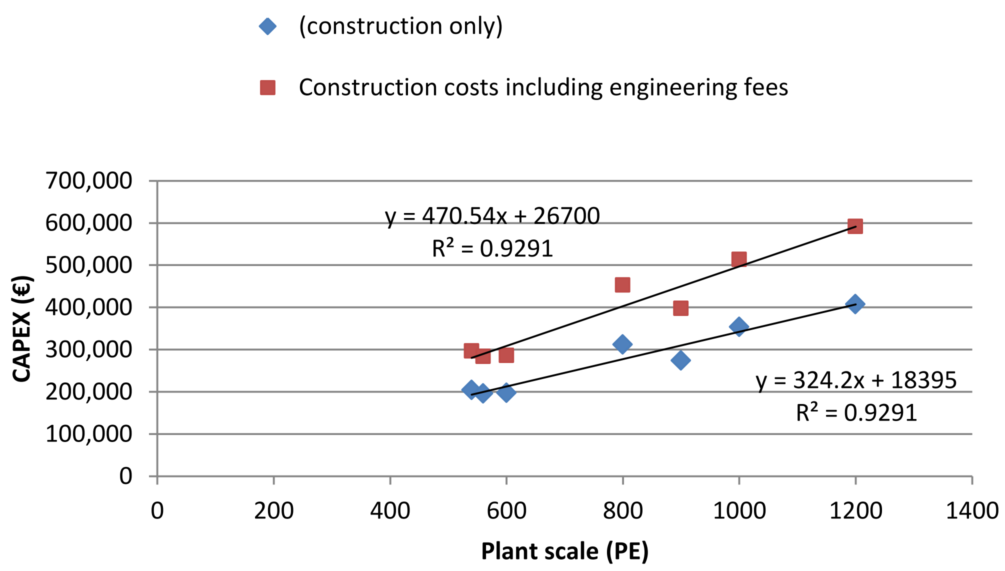

Figure 28: Linear regression model CW systems CAPEX (adapted from [4])

Vymazal reported that vertical flow constructed wetlands (VF-CW) provide the best option for nitrification and horizontal flow constructed wetlands (HF-CW) for denitrification (Table 24). Therefore, for systems required to reduce TN a hybrid VF-HF system is proposed. According to Gkika et al. [4], the inclusion of HF-CW cells for denitrification was found to account for an additional 10% of the total construction cost, and an additional 0.75m^2/PE in surface area requirements.

Table 25: CW nitrogen removal mechanisms

Nitrogen removal mechanism	Free surface water	Sub-surface horizontal flow	Sub-surface vertical flow
Ammonification	Medium	N/A	N/A
Volatilisation	High	High	High
Nitrification	Medium	Low	High
Denitrification	Medium	High	Low
Microbial uptake	Low	Low	Low
Plant uptake	Low	Low	Low

Vymazal also reported that CW TP removal efficiencies were low and averaged less than 40% in a survey of 386 CW systems, and that CW systems are generally not implemented with TP removal as target pollutant. In the study conducted by Gkika et al. [4], the problem of low TP removal at several CW plants was addressed by the inclusion of an anaerobic tank prior to the first stage VF cells at an average of 9.8% of the total construction cost and an additional 0.09 m^2/PE. The CW system layout is presented below (Figure 29).

Sludge production

As a simplification, the mass of solids deposited in the first VF stage is assumed to equal that of primary sedimentation in the electro-mechanical systems, and represents the bulk of the sludge to be disposed of. Sludge deposition in the remaining stages is assumed to be negligible. All CW systems are assumed to employ sludge drying beds.

Energy

Energy consumption in the CW system model is limited to influent rising at a static head height of 3 meters, inlet screening, and anaerobic tank mixing where phosphorus removal is required. There is also a negligible sink attributed to municiple energy (lighting). It is assumed that the systems layout has been designed to allow the water line to flow without any additional pumping.

Chemicals

It is assumed that chemical addition is limited to lime addition for sludge stailisation.

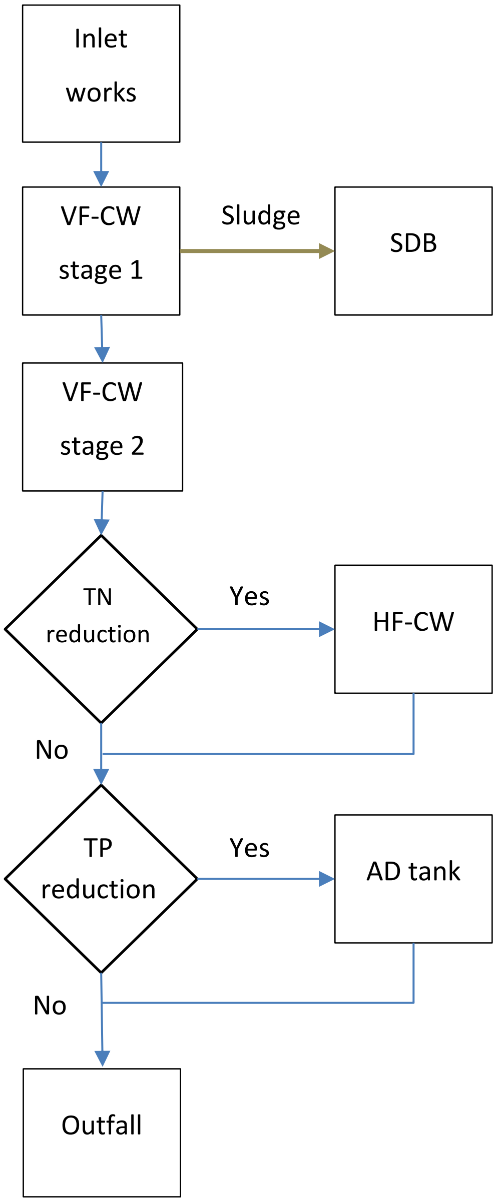

Figure 29: Constructed wetlands system layout and logic

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