Predictive Modelling of Hydraulic Flows to a WWTP based on Catchment Rainfall Data

A thesis submitted for the award of Master of Engineering (MEng)

by

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Candidate Declaration

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To Tiarnan and Claragh I offer thanks by way of advice: be careful what you wish for!

Finally I wish to dedicate this thesis to the memory of Mr. Eddie McCarney and Mr. Neil Corrigan. I'm grateful for the times we shared but wish we'd had time for more.

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AD – Anaerobic Digestion
bcm – Billion Cubic Metres
BOD – Biochemical Oxygen Demand
CAS – Conventional Activated Sludge
cBOD – Carbonaceous Biological Oxygen Demand
COD – Chemical Oxygen Demand
CSO – Combined Sewer Overflow
CSS – Combined Sewer System
DEA – Data Envelopment Analysis
DO – Dissolved Oxygen
DWF – Dry Weather Flow
EI – Energy Intensity
EPA – Environmental Protection Agency
ESI – Energy Systems Integration
ESIPP – Energy Systems Integration Partnership Programme
F/M – Feed to Micro-organism Ratio
GHG – Greenhouse Gas
IFAS – Integrated Film Activated Sludge
iRTC – Integrated Real Time Control
iUDS – Integrated Urban Drainage System
MTOE – Megatonne Oil Equivalent
nBOD – Nitrogenous Biochemical Oxygen Demand
NI Water – Northern Ireland Water
NIE – Northern Ireland Electricity
RAS – Return Activated Sludge
RDII – Rainfall Derived Inflow and Infiltration
RTC – Real Time Control
SDEA - Stochastic Data Envelopment Analysis
SSSs – Separate Sewer Systems
TIF – Total Inlet Flow
UDS – Urban Drainage Systems
UN SDGs – United Nations Sustainable Development Goals
UWWTD – Urban Wastewater Treatment Directive
WAS – Waste Activated Sludge
WFD – Water Framework Directive
WWFs – Wet Weather Flows
WWPS – Wastewater Pumping Station
WWTP – Wastewater Treatment Plant

Abstract

Predictive Modelling of Hydraulic Flows to a WWTP based on Catchment Rainfall Data

Eoin Daly

One of the greatest influences on the volume and quality of influent flows to wastewater treatment plants (WWTPs) is rainfall in the drainage area. These wet weather flows (WWFs) can lead to significant environmental impacts due to sewer overflows and affect the efficiency and resource cost of treatment in WWTPs. Previous studies have considered the impacts of WWFs on the WWTP and treatment efficiency, however few have considered the impacts of WWFs on the sewer network that serve these treatment plants. This thesis investigates the influence of drainage area rainfall on the hydraulic flows through an urban drainage system in Northern Ireland using available data. The contribution of the electricity used at pumping stations in two drainage areas is calculated, assessed and compared to the electricity consumed for treatment alone. A method of predicting the increased volumetric flows due to rainfall in the drainage area using the non-linear reservoir model is also presented. Using the developed model, it is possible to predict potential flows through the network, although the quality of the results will depend on the quality of the data available. Using single point measurements of rainfall in the catchment and only data already measured by the water utility, an interquartile range of percentage errors between -10% and 14% of inlet flows was achieved for a one-day time horizon. This range increases to 2% to 48% of annual inlet flows over a four-day horizon. While these error ranges are far from desirable, with better measurement regimes and data availability the method can be refined further, and increased accuracy provided.

Introduction

There is an interdependence between the water and energy sectors that has traditionally gone under-recognised, as both sectors have developed independent of each other (Gude, 2015; O'Doherty et al., 2014; U.S. Department of Energy, 2014). Water security and the impacts of climate change have both been identified by the World Economic Forum as major factors in international conflict (Kitty Van Der Heijden and Callie Stinson, 2019) and should be considered top priorities globally if further conflicts are to be avoided. The extraction, treatment and transport of both water and wastewater has been estimated to consume between 1.7% and 2.7% of the world's primary energy consumption (Liu et al., 2016) and more than 2% of the world's electrical energy (Vaccari et al., 2018). By 2014 the Energy Intensity (EI) of the global water sector stood at 120 million tonnes of oil equivalent (MTOE) most of which was consumed as electricity, representing 4% of global electricity consumption (OECD/IEA, 2016). Between 3 and 4% of total electricity used in the United States is used in the water and wastewater sector, representing 56 billion kW of electricity. The total energy consumption of the water and wastewater sectors can vary between 20% and 40% in different regions (Gude, 2015).

Of the 4% electricity used globally, about 25% is used in the treatment of wastewater (OECD/IEA, 2016). This figure also varies regionally, with some municipalities in Germany consuming 20% for wastewater treatment (Wang et al., 2016). In general 1% of total national energy consumption is considered a good estimate for the energy consumed in European countries for the treatment of wastewater (Longo et al. 2016). In return 10% of global water withdrawals go to meeting energy sector requirements with water being used at almost every stage of energy production (OECD/IEA, 2016), 660 million m3 of water globally going generate the energy required for wastewater treatment alone (O'Doherty et al., 2014). Energy production has also been identified as a major contributor to greenhouse gas emission (Wang et al., 2016) but a further 45 million tonnes of CO_2 is emitted by the processes used for wastewater treatment (Gude, 2015). With increasing intensification in both sectors expected over the next number of years driven by increases in populations and quality of life (Gude, 2015; OECD/IEA, 2016) proper co-ordinated planning across relevant sectors is required now to reduce global CO_2 emissions; mitigate the effects of climate change and the associated social conflicts that may arise.

If society is to continue to grow within the framework of the United Nation's Sustainable Development Goals (UN SDGs) (United Nations, 2015) then energy reductions within the water treatment sector and reductions in the water requirements for the energy sector should be sought (Liu et al., 2016; Molinos-Senante et al., 2018, 2014). The integrated solutions required to meet the challenges associated with this energy-water nexus will require an inter-disciplinary approach across traditional industrial and academic boundaries (Gude, 2015). To address this in an Irish context, the Energy Systems Integration Partnership Programme (ESIPP) was established in 2016. ESIPP brings together multi-disciplinary teams from universities and industry from across Ireland, with the aim of creating the environment necessary to build a knowledge base in the area of Energy Systems Integration (ESI). This work takes a holistic view of energy systems and combines it with knowledge from different areas, such as water management and social science. The objective is to monitor society's consumption of resources with a view to optimising their use. This thesis forms part of the work undertaken as part of the ESIPP work package EUI5: energy-water nexus and is focused specifically on the wastewater treatment sector.

The primary objective of this research is the energy benchmarking of wastewater treatment with due regard to end user behaviour. The over-arching goal is to address the energy/water nexus in an Irish context as well as to better understand end-user behaviour in relation to the water and energy sectors in Ireland. The following aims have been outlined in relation to the research undertaken here:

- To study and quantify the energy consumption of the processes used in wastewater treatment along with their environmental and social impacts.
- To evaluate holistic approaches to the assessment of energy consumption in wastewater treatment, representing as close to a "true" accounting as possible at a regional or municipal level.
- To consider the net energy consumption/production of the processes involved as current technologies develop.
- To contribute to the development of models and tool kits that can be used to quantify and improve awareness of the energy-water nexus.

With these aims and objectives in mind, several early research questions were formulated:

1. What is the relevance of the energy/water nexus, how has the nexus developed over time and what literature exists regarding the wastewater component of the nexus?

2. What is the energy intensity associated with the most common treatment options available to wastewater engineers and how do these treatment options work?
3. What are the sources of variability in wastewater treatment in terms of removal efficiency, hydraulic loading and the energy requirements?
4. What are the drivers of energy intensity in wastewater treatment?
5. What is considered "state of the art" in the field of energy benchmarking of wastewater treatment?

The literature review contextualises the relationship between society and water, outlining the development of modern urban drainage systems (UDSs) and investigating their associated resource cost. Integrated approaches to managing and modelling UDSs and a review of real time control (RTC) options for UDSs follows, with a view to the potential energy savings through better system management. Research objectives are developed based on the findings of the literature review and are then investigated. An analysis of the contribution of the drainage area to the electricity consumption for treatment at two locations in Northern Ireland is performed. A model for calculating and quantifying the increase in hydraulic loading due to rainfall events in the drainage area using available data is developed. The results of these analyses are then presented, evaluated and discussed. Finally the conclusions and suggestions for future work are given.

Literature Review

The Need for Water

It is hard to think of a resource society is more dependent on than water. It is unique among chemical compounds, necessary to sustain life on Earth, an important part of the environment's system for energy distribution around the planet and the means by which it does so are such important parts of global climate (Chocat et al., 2004; Edwards et al., 2015; Falkenmark, 2011, 1977). Approximately 70% of freshwater abstraction goes to agriculture and that value continues to increase, particularly in developing countries such as India, Africa, the Middle East and Latin America (Liu et al., 2016). The ability of water to act as a carrier of materials means that it acts not just as a nutrient carrier, but also poses a threat to environments as toxic materials can be carried just as easily (Falkenmark, 1997). Water is such a necessity that its location has influenced human settlement patterns throughout history and its supply is considered a limiting factor on human population (Falkenmark, 1997; Molinos-Senante et al., 2014). Water has been used as a measure of societal development as it was only when humans developed the means to manipulate and transport supplies that settlements could move away from their early water sources. Water is a part of what is referred to the "carrying capacity of a region" and technological developments can be used to modify this, thereby changing the environment (Falkenmark, 1997, 1977; Sivapalan et al., 2012).

As the population of the Earth continues to grow it tends to do so in urban areas. Since 2009 more than half the world's population have migrated to urban areas (Bach et al., 2014; Salvadore et al., 2015). By 2030 it is expected that 80% of the world's population will live in such areas, driven by development in developing countries (Salvadore et al., 2015). Global water extraction has already increased substantially in recent decades: since 1975 global water extraction has increased from 2,876 billion cubic metres (bcm) to 4,169 bcm in 2010 (Liu et al., 2016). The construction and operation of infrastructure required to increase a region's carrying capacity also comes at an economic and social cost. The economic cost of such infrastructure is perhaps the more obvious, but the social costs are not as clear, e.g. water transfers from areas of higher water availability and interference with waterbodies that cross national and international boundaries can be a source of conflict and tension. To date much of work done in the field of water management has been to balance these economic and social costs. These costs can be significant and water supply is considered a significant threat to regional and global security contributing to current and future conflict.

Traditional urban drainage practices considered the removal of waste from urban areas as the primary objective for collection systems. This focus remains primarily due to the economic costs associated with flooding as well as potential health hazards. Over time the need for treating wastewater prior to discharge became apparent for the protection of public health. More recently considerations of the impacts of design and operation of urban drainage systems has led to the implementation of sustainable drainage practices and a general acceptance of the need to consider ecological impacts at a catchment level and the emergence of techniques and frameworks to deal with systems in this way (Chocat et al., 2004).

The Hydrologic Cycle

To better understand the relationship between society, water consumption and the environment, it is important to first understand the hydrologic cycle. An overview can be seen in Figure 1 and it can be described as a description of how water is stored and flows through a closed system of processes on a large or global scale (Edwards et al., 2015; Viessman and Lewis, 2003). While there are no noticeable amounts of water gained or lost from the system, the distribution of the water does vary. The largest reservoir of water by far are the world's oceans with over 97% of the water on Earth (Gat, 2010; Viessman and Lewis, 2003). Fresh waters account for approximately 3.9 x 1016 m3 of the water. This is made up of the ice and glaciers on mountains or at the poles, amounting to 2.9 x 1016 m3; groundwater, which makes up 9.5 x 1015 m3; and surface waters account for 1.3 x 1014 m3. The biosphere is thought to take up 0.6 x 1012 m3 of the remaining water while the smallest portion is in the atmosphere with 0.13 x 1012 m3. This small amount belies the true importance of this portion of the cycle however, as the movement of water through the atmosphere is an important driving factor of the entire hydrologic cycle (Falkenmark, 2011; Gat, 2010; Viessman and Lewis, 2003). It is also worth noting that while this overview of the hydrologic system may be recognised by many, it often neglects crucial interactions that occur within and hides the unique role of humanity in having an influence over all aspects of the cycle (Sivapalan et al., 2012). Increases in surface temperatures as a result of climate change leading to increased water levels in the atmosphere is one modification to the global hydrologic cycle with serious implications for society (Falconer et al., 2009; Falkenmark, 2011, 1997, 1977).

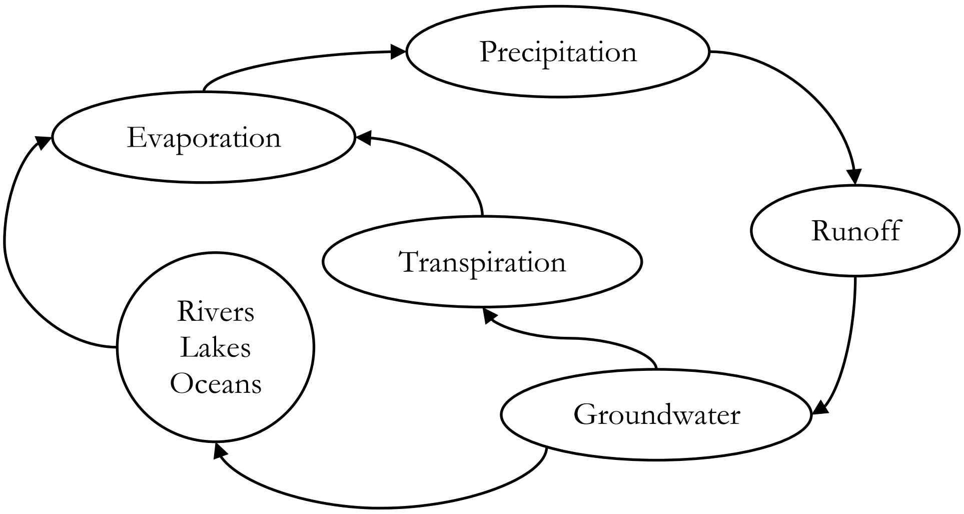


Figure 1: The Hydrologic Cycle

While the natural laws governing the various sub-systems of the hydrologic cycle are well understood, understanding the interactions between systems, particularly with social systems, is a more difficult task. Many of the challenges to research in the field of urban drainage are related to the complexity of the systems involved (Chocat et al., 2004). Unexpected consequences can develop over time when the dynamic nature of these interactions are not taken into account and the long term interactions in particular need to be better understood (Sivapalan et al., 2012). One example of such an unintended consequence that can create conflict are those that follow dredging upstream portions of rivers without consideration of the impacts downstream. While the dredging may prevent flooding in the upper reaches of the catchment, the subsequent increase in flows may cause flooding down river. If we are to realise society's potential for sustainable development, then holistic approaches will be needed that consider human-environmental interactions. Failing to accept this or society's role as environmental steward may see factors pushed beyond acceptable thresholds leading to black swan events without warnings or predictable frequencies (Falkenmark, 1997; Sivapalan et al., 2012; Viessman and Lewis, 2003).

Hydro-Sociology

Emerging in the 1970s the field of hydro-sociology is concerned with the study of the relationship between humans and water; it does this by trying to understand the interfaces between the two (Sivakumar, 2012; Sivapalan et al., 2012). Early work by Falkenmark (Falkenmark, 1977) classified the many interactions between society and hydrology as direct or indirect modifications and the various secondary effects of these. Over time feedbacks within or between systems also begin to emerge, which can complicate the issue further (Falkenmark, 1977). Population increases, urbanisation and increases in quality of life all have implications for societies use of water as well as wider resource consumption (Gude, 2015; Liu et al., 2016).

Urban areas are particularly important to the study of hydro-sociology as they are characterised by large populations in higher densities.

The built environment of urban areas also has direct implications on the hydrology of an area, as their many impervious surfaces prevent flow into soils beneath. The flow that these surfaces redirect are typically sent to drainage systems, in what amounts to a man-made modification of surface and ground-water flow patterns. These systems have the primary function of quickly removing storm water away from urban areas, resulting in relatively large volumes of waters being discharged into water bodies in a short period of time. Where these waterbodies are unable to handle the increased streamflow, or where the drainage system is inadequate to the needs of the urban area, flooding can still occur (Edwards et al., 2015). Urban areas are known to be vulnerable to flooding in the face of heavy rainfall, which comes with an economic and social cost, and could become more acute in the context of climate change. In Europe alone the cost of flooding is estimated to have been between 5 and 35 billion US$ per year between 1980 and 2010 and where drainage systems are inadequate the costs to public health could be innumerable (Falconer et al., 2008, 2009; García et al., 2015; Salvadore et al., 2015). While flooding and the potential for early warning systems have been the subject of research for some time (Falconer et al., 2008, 2009), recent evidence would suggest that there is little progress in these advanced warning systems, at least in an Irish context (Berry, 2019; McDermott, 2019).

The importance of hydro-sociology can also be seen in the food sector, which has led some to expand the energy-water nexus to include food (Bazilian et al., 2011; Endo et al., 2017; Smajgl et al., 2016). The nutrient carrying capacity of water is an important part of the food chains and the presence in wastewater of many of the nutrients necessary for growth has meant that wastewater irrigation is one end use for wastewater discharge that has long been employed (Anon., 1896, 1895; Chocat et al., 2004; Lens et al., 2001; Tilley, 2011; Wiesmann et al., 2006). The difficulty with this solution is that soil provides a ready sink for such contaminants as water filters through. Modern chemicals, pharmaceutical active compounds (PhACs) or other pollutants found in wastewater irrigation systems can pose a potential human health risk. They can be transferred to the soil by irrigation systems, from where they accumulate in edible parts of crops finding their way into the food chain in low concentrations (Christou et al., 2017). These compounds may also leach their way into groundwater posing potential risks further downstream in the hydrologic cycle. These interactions may mean there is a risk that continuous disposal of potential environmental contaminants may become a concern to the environment and public health over time. As society continues to develop increased population coupled with overall improvements in living standards will require improvements in wastewater water treatment or the overall quality of water circulating in this hydrologic cycle will suffer (Falkenmark, 1977; Gude, 2015; Liu et al., 2016; Molinos-Senante et al., 2018). These increasing demands will come with increased costs in terms of energy and will require innovative solutions to overcome challenges (Gude, 2015). The heterogeneous nature and regional specificity of water related problems may also mean that, with regard to water quality, universal solutions may not be the best ones (Falkenmark, 2011, 1977; Viessman and Lewis, 2003). This may also pose problems for traditional governance types.

The discharge of wastewater back into the hydrologic cycle poses obvious dangers to the environment as well as to public health. The specifics may change from place to place but water quality is an issue facing all regions and societies of the world. Coupled with the depletion of non-renewable energy sources and the impacts of climate change, these threats will have serious hydro-sociological influences (Liu et al., 2016). The quality of water will naturally be in flux as it moves through the hydrologic cycle, but contaminants can also be introduced by society. Modern household and industrial chemicals, as well as pharmaceuticals consumed by the water users and returned for wastewater treatment cannot be completely removed in many facilities. This leads to the addition of other treatment processes to ensure their removal which in turn increases the energy demand of treatment (Mousel et al., 2017) or an increase in the chemicals required for treatment. The energy-water nexus will be an important part of the hydro-sociological system going forward if future social developments are to be considered in the context of sustainability (Liu et al., 2016). In doing so the traditionally independent approaches of each sector will need to consider the implications of interdependence between the two (Gude, 2015; O'Doherty et al., 2014; U.S. Department of Energy, 2014). The energy consumed by the water sector has been increasing steadily over recent decades, almost doubling in the years between 1975 and 2010 (Liu et al., 2016), by 2014 the global water sector consumed 120 million tonnes of oil equivalent (MTOE) in energy which includes 4% of global electricity consumption.

For its part the global energy sector is one of the largest consumers of global water, with 10% of all the water withdrawals worldwide used to meet energy requirements (OECD/IEA, 2016). Wastewater treatment forms an important part of this nexus, consuming one quarter of the energy in the entire water sector (OECD/IEA, 2016) which amounts to an estimated 1% of total national energy in European countries (Longo et al. 2016). Additionally 660 million m3 of global water extraction is used to generate the energy required for wastewater treatment (O'Doherty et al., 2014). All of this means that wastewater treatment is an important component of the energy – water nexus and is due consideration in the context of future development.

Urban Drainage Systems

Urban Drainage Systems (UDSs) are systems responsible for the collection, transport, storage and treatment of sanitary and storm waters in urban areas (Chocat et al., 2004; García et al., 2015). They have traditionally been built to maintain public health, prevent flooding and to control pollution for the preservation of the environment and they are crucial for their social, economic and environmental benefits to society (Chocat et al., 2004; García et al., 2015; Młyński et al., 2016; Panepinto et al., 2016; Rauch et al., 2002). Meeting the requirements of the population for water and removal of wastewater means urbanisation, and the associated changes in land use can have significant effects on the local hydrology. Urban areas can be one of the main sources of diffuse water pollution and are major contributors to the contamination of ground and surface waters, with seepage from sewer networks being the most common source of released toxic substances into the environment. While sewers suffer less from leakages than water supplies they still represent leakage that accounts for between 5 and 20% of sewer fluxes (Chocat et al., 2004; Salvadore et al., 2015). Environmental burdens can be exacerbated further by the rate of population increases in urban areas, in particular where population increases at a rate greater than the development of the water and wastewater infrastructure to serve them (García et al., 2015; Młyński et al., 2016; Salvadore et al., 2015).

There are also the changes to land cover caused by urbanisation. Roads and buildings increase the impermeable areas increasing runoff volumes and often it falls to UDSs to deal with these volumes. These are examples of direct modifications to an area's hydrology but there are also the indirect influences of urban induced changes. Changing the energy balance with changes to land coverage and anthropogenic heat generation, modified surface roughness impacting wind patterns, thereby changing precipitation patterns, and pollution of the atmosphere having an impact on the chemical composition of both the atmosphere and rainfall. The effects of urbanisation on precipitation is particularly acute and has been an active area of research since the 1970s. The urban effects on evapotranspiration have also been studied (García et al., 2015; Salvadore et al., 2015). There has also been a recent focus on greenhouse gas (GHG) emissions associated with urban drainage systems, as emissions have grown in line with increasing inflows (Mizuta and Shimada, 2010). GHGs are produced in several ways during wastewater conveyance and treatment. The aerobic oxidation processes used in treatment produce CO2, embodied GHG emissions in chemicals used and transportation all contribute to GHG emissions from plants. CH4 production in sewer networks and released to the atmosphere can also contribute to the GHG footprint, all of which is in addition to those emissions from the electricity generation for treatment (Flores-Alsina et al., 2011).

UDSs are crucial to dealing with the difficulties posed by: increasing populations; increased standards of living; urbanisation; flooding and climate change impacts. The measures required to meet environmental requirements have been shown to be costly (Pleau et al., 2005), but the price of inaction could be a great deal more. Therefore governments have had to find new ways to adapt to problems that society faces in infrastructure and utility provision (Bach et al., 2014). A backlog of refurbishment work already deferred as well as growing emissions' charges, increased flood protection requirements, improved bathing water quality and more rigorous environmental protection legislation and EU Directives are all expected to leave municipalities and operators across Europe with large urban drainage investment bills (Breinholt et al., 2008). Recent events in Dublin, where the Ringsend treatment plant is over its design capacity, which has meant that even small rain events resulted in combined sewer overflows (CSOs) from the treatment plant. This problem has by no means been conclusively dealt with and remains a contemporary issue (Irish Water, 2019; Kevin O'Sullivan, 2019). UDS improvement can be broadly classified into two approaches: the network capacity approach and the network management approach. The former deals with increased volumes by expansion of the UDS: building more capacity in sewer networks with bigger tanks and larger pipes; increasing the capacity of WWTPs; catchment disconnections; and source control measures (Breinholt et al., 2008; Erbe et al., 2007; Schütze et al., 2008). This approach to solving the problem is not without difficulty: increasing network capacity can be economically and practically difficult to achieve, but nevertheless this is the traditional approach taken by operators and managers (Breinholt et al., 2008; García et al., 2015). The alternative approach of improving UDSs through improved management techniques, such as the integrated management of UDSs, can be a more cost-effective measure. This approach is at an early stage of development however and would benefit from interdisciplinary work across a range of fields, as will be discussed further in later sections.

Development of Wastewater Treatment

While settled societies have always had to deal with the problem of wastewater, the earliest solutions were to transport wastewater away from dwellings and urban areas. Often natural drainage ditches or channels were used but over time purpose-built networks were developed and maintained. This transportation was the solution to the problem of wastewater from antiquity through the Roman period and the Dark Ages that followed, though there were some limited examples of treatment also. Little developed in the field up until the Industrial Revolution when cities across Europe began to grow at a rate never seen before and with it came new problems. As these urban areas grew, the burden they placed on the environment grew too and conflicts between societies and the environment around them began to emerge in the form of various epidemics (Billings, 1885; Bingham et al., 2004; Budd, 2013; Chocat et al., 2004; Dobraszczyk, 2014; Dunnill, 2014; Gandy, 1999; Kesztenbaum and Rosenthal, 2017; Lens et al., 2001; Snow, 1849; Sunderland, 1999; Tilley, 2011; Wiesmann et al., 2006). The first wastewater treatment plants were developed to protect public health in urban areas, with environmental protection becoming a concern later in development. The processes used to treat wastewater have been developed from naturally occurring biological processes, with treatment plants creating an ideal environment for these processes to occur at a greater rate than would be naturally possible. Public health was the main driver of wastewater treatment developments, which translated into design objectives and operating procedures for UDSs. Plants designed and built from the early 20th century until the 1970s were designed to remove suspended and floating material, treat biodegradable organic material and eliminate pathogenic materials found in wastewater (Alleman and Prakasam, 1983; Burton et al., 2013; Lens et al., 2001; Longo et al., 2016; Lorenzo-Toja et al., 2016; Tilley, 2011).

Aside from the technical difficulties of meeting changing regulations and requirements of treatment, there is another problem in terms of wastewater treatment provision. Water and sanitation is one of the UN's sustainable development goals (United Nations, 2015) and wastewater treatment will be a crucial component in this. As of 2015 there was still 32% of the global population who are without wastewater treatment (Arroyo and Molinos-Senante, 2018), so achieving the sustainable development goals will mean this figure will need to be addressed using effective if not overly technical solutions.

Current Status of Wastewater Treatment

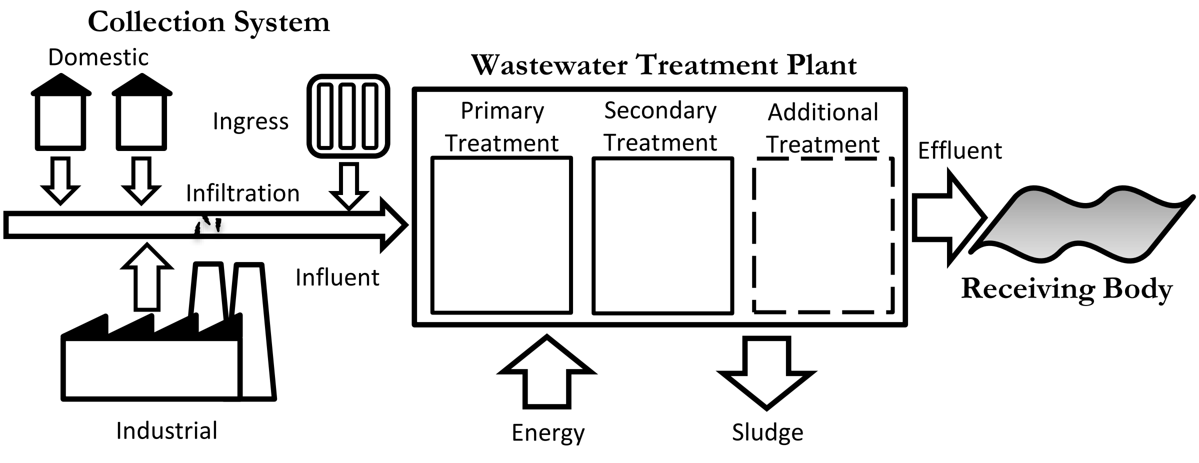


Figure 2: Outline of the Urban Drainage System

The processes within the wastewater treatment plant are typically broken down into three stages: primary, secondary and tertiary or additional treatment (Burton et al., 2013). The treatment processes can also be separated by the treatment type: physical treatment where the use of physical forces predominate e.g. screening, mixing or filtration; biological treatment makes use of biodegradation processes to treat wastewater; and chemical treatment processes are those that make use of chemical reactions to treat wastewater e.g. chemical precipitation or disinfection (Burton et al., 2013). As a broad generalisation it can be said that the primary and secondary treatment categories are predominantly physical and biological processes in a typical wastewater treatment plant. There are often exceptions to this rule however e.g. advanced primary treatment where precipitative chemicals may be added to enhance primary treatment (Burton et al., 2013).

Before the wastewater begins primary treatment it usually passes through preliminary treatment and a flow equalization stage. Both may occur at the headworks of the treatment plant but can also occur prior to the headworks throughout the catchment area. Preliminary treatment involves the removal of large objects or debris in the flow of wastewater that may cause problems with treatment processes or machinery (Burton et al., 2013). Flow equalisation usually takes place in offline storage tanks where water can be passed during periods of excess flow to dampen peak flows, thereby protecting treatment processes.

Primary settlement involves the removal of the easily removed portion of the suspended solids and floating materials (Burton et al., 2013). The tanks used for settlement can be of a circular or rectangular design, with the heavier materials sinking to the bottom and cleaner water being removed from the top of the tank using weirs. The sludge accumulations are typically mechanically removed from the bottom of the tank while the scum or floating material is similarly removed from the top of the tank.

Secondary treatment is usually a biological process and can be broken down into three broad categories (Burton et al., 2013):

- Suspended Growth Systems – where the micro-organisms responsible for treatment are suspended within the wastewater. Examples include activated sludge systems, aerated lagoons and membrane bio-reactors.
- Attached Growth Systems – where the micro-organisms are attached to an inert medium such as rocks or specially designed ceramic or polymer materials. Examples are the packed bed, rotating biological contactor (RBCs) and trickling filter processes.

- Hybrid Processes – These processes use some combination of the previous two. Examples include trickling filter/activated sludge systems and integrated film activated sludge (IFAS) systems.

The most commonly employed technologies of these plants in the US are activated sludge treatment; nitrification; biological nitrogen removal; enhanced nitrogen removal; and membrane bioreactors (Wang et al., 2016). The preference for activated sludge treatment processes is echoed in other parts of the world too (Vaccari et al., 2018).

Activated sludge, sometimes referred to as conventional activated sludge (CAS), is generally considered the norm for biological treatment since its emergence at the turn of the 20th century. Prior to this the technology used to treat wastewater, which was still in its infancy, had been based around physical or attached growth processes such as intermittent filtration, trickling filters or contact beds (Alleman and Prakasam, 1983). The reliance on attached growth systems was based on the limited knowledge of mechanisms that had been observed in nature for centuries. Early treatment systems usually distributing wastewater over large areas of land, on what would become known as sewage farms (Anon., 1896, 1895), where naturally occurring processes would treat the wastewater as it drained away. Early investigations considered the processes that were taking place in the soil, and an important discovery in the 1880s was the observance of biofilm growth on a wire mesh as untreated wastewater was sprayed onto it and the consequent removal of dissolved organics (Tilley, 2011; Wiesmann et al., 2006). As these early investigations developed it was discovered that the attached growth biofilm processes performed better with the presence of additional air (Alleman and Prakasam, 1983). During the late 1890s and early 1900s the concept of a suspended growth system was being investigated, where the biomass would be suspended within the wastewater into which air could be pumped to promote growth (Alleman and Prakasam, 1983; Tilley, 2011). In wastewater treatment, the biomass is the mass of solids in a reactor made up primarily of organic matter and micro-organisms (Burton et al., 2013). By 1910 suspended growth systems existed, however the biomass produced in these facilities was discarded after treatment. The crucial step in activated sludge systems came in 1914 with the discovery by Ardern and Lockett that when enough oxygen is passed through the suspended growth system there is an increase in the biomass within the system. If this biomass is then separated from the effluent and retained within the system to be mixed with the influent, then the amount of oxygen and the time required to treat wastewater can be reduced (Alleman and Prakasam, 1983; Tilley, 2011). This increase in the biomass used in treatment comes at a cost however: an increase in the oxygen required to sustain it and there is an associated energy consumed for aeration systems. It was the spread of this technology during the 1920s in the US and UK as treatment facilities began to be constructed that contributed to its widespread adoption (Alleman and Prakasam, 1983), with many facilities today being CAS or a derivative of its basic process.

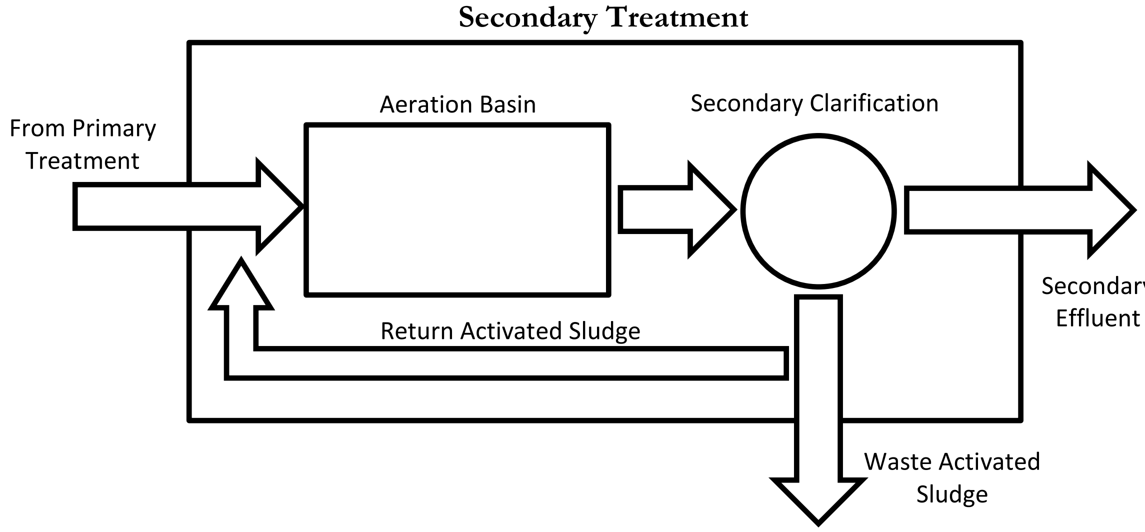


Figure 3: Simple Activated Sludge Process Flow (Adapted from (Burton et al., 2013))

As shown in Figure 3, at its simplest the activated sludge process consists of two processes, the aeration process and the clarification process. The term activated sludge refers to the activated biomass contained within the aeration basin that is used to stabilise wastewater under aerobic conditions. Wastewater arrives to the secondary treatment process having undergone primary treatment to remove easily removed solids and debris. The biological treatment begins in the aeration basin, where air is mixed through the wastewater, either by means of mechanical surface aerators or submerged diffusing aerators. From the aeration basin, wastewater then travels to the secondary clarifiers for settling where, like primary clarification, the heavier biomass settles to the bottom of the tank while the treated effluent is discharged from the top. The biomass at the bottom of the clarifiers is then separated from the wastewater and can travel down two streams. Return Activated Sludge (RAS) is returned to the influent stream prior to aeration basin where it is mixed with the incoming influent. The second pat removes the Waste Activated Sludge (WAS) from the treatment system as a waste by-product, commonly referred to as sludge. This sludge can be processed further, either on-site or at a separate location, depending on local standards.

Aerobic Oxidation

The biological processes that take place in the aeration basin is the removal of biochemical oxygen demand (BOD) through a process of aerobic oxidation. BOD is the most used measure of organic pollution and more specifically is a measure of the amount of oxygen consumed by micro-organisms within one litre of wastewater and is measured as mass per unit volume (g/L). The measurement of oxygen demand is time dependent and so BOD is measured over a specified time period, typically five days (BOD5) (Burton et al., 2013). The processes involved in primary treatment are relatively efficiently designed and operated, typically removing 50 to 70% of suspended solids and 25 to 40% of BOD (Burton et al., 2013). Typical municipal wastewater influent in the US lies in the range of 110 – 350 mg/L for BOD5 and 120 – 400 mg/L for total suspended solids (TSS), while typical effluent requirements are in the range of 2 – 20 mg/L and 2 – 10 mg/L for BOD5 and TSS respectively (Burton et al., 2013). Excessive BOD loading in receiving waters due to wastewater discharges can cause environmental problems due to the oxygen demand of the micro-organisms.

Aerobic oxidation is a conversion process where the organic matter contained in the wastewater is broken down by micro-organisms in the presence of oxygen and foodstuffs, in this case suspended in the wastewater. The first stage in this process are the oxidation and synthesis reactions and can be written as an unbalanced chemical equation, shown as equation (1) (Burton et al., 2013; Ovivo, 2016):

$$C, O, H, N, S + O_2 \rightarrow CO_2 + NH_3 + C_5H_7NO_2 \qquad (1)$$

An explanation of the components and processes involved in equation (1) is also shown in Figure 4.

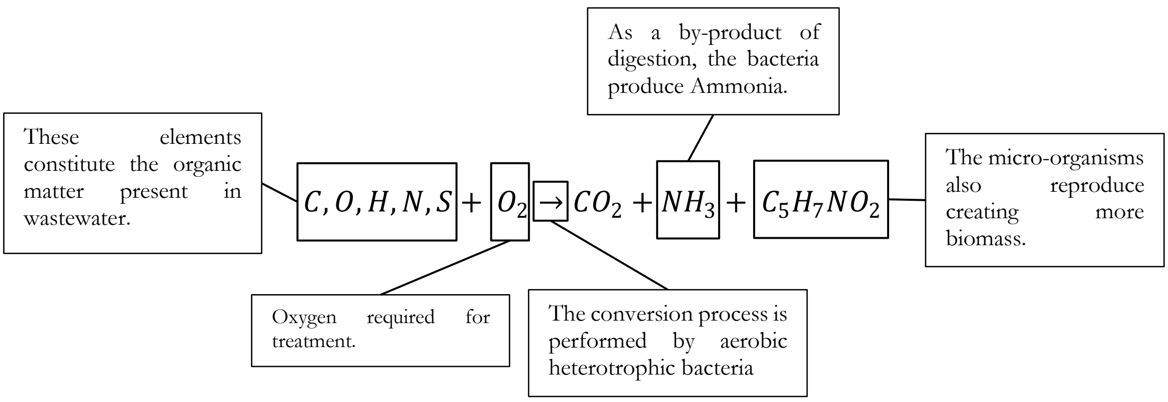


Figure 4: Oxidation & Synthesis Reactions

When this process has been completed i.e. the micro-organisms have consumed all the organic material available, a process of endogenous respiration begins. This is a process where the micro-organisms begin to consume their own protoplasm to fuel internal reactions and is shown in equation (2) and explained in Figure 5. This results in a reduction in the biomass as well as the production of a number of further by-products (Burton et al., 2013; Ovivo, 2016):

$$C_5H_7NO_2 + 5O_2 \rightarrow 4CO_2 + H_2O + NH_4HCO_3 \qquad (2)$$

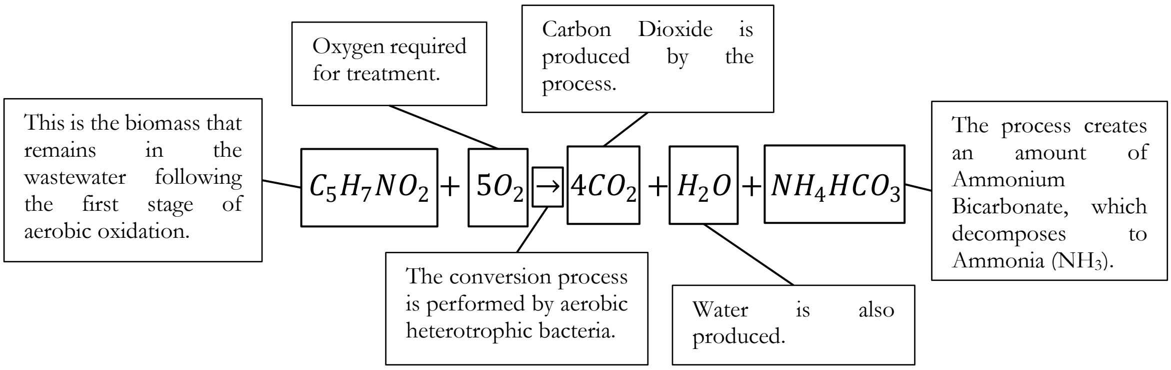


Figure 5: Endogenous Respiration

The amount of biomass returned or discarded as waste in CAS systems is governed by a variety of factors but is primarily based on the mixed liquor suspended solids (MLSS), which is the concentration of suspended solids in the aeration basin measured in g/m3, and the food to micro-organism ratio (F/M Ratio), which is the ratio of influent BOD or COD concentration to the volume of mixed liquor (Burton et al., 2013). Typical values for MLSS are in the range of 2,000 to 4,000 mg/L and is dependent on the design of the plant, specifically the aeration available and the capacity of the secondary clarifiers. The aeration capacity must be sufficient to provide for the concentration of biomass in the aeration basin, while the clarifiers should be able to settle out solids: more solids require more time to settle which will have an impact on the hydraulic retention time (HRT) of the plant. The HRT of the plant is the length of time liquid is retained in the treatment works and is typically 4 to 24 hours depending on the processes involved. A corresponding measure of the length of time the biomass is retained in the plant is known as solid retention time (SRT), which is typically 2 to 8 days for BOD removal and longer for nutrient removal.

If discharged to a water body the active micro-organisms in wastewater will continue the oxidation and synthesis reactions, consuming the dissolved oxygen within the water course. This results in hypoxic or anoxic conditions within the receiving waters, resulting in a “dead” watercourse unfit to sustain life. The legislated discharge requirements for effluent are implemented to prevent this, meaning the additional biological processes in secondary treatment step are necessary in modern treatment. The amount of oxygen within the aeration basin must also be sufficient for BOD removal and this is achieved by using DO probes within the aeration basin to continuously monitor oxygen levels and adjust aeration accordingly. If oxygen levels are not monitored it can result in several problems: if levels are too high energy requirements are increased and conditions become more favourable for filamentous bacteria causing problems for treatment, which can be difficult to settle. Reduced oxygen levels mean that there can be insufficient BOD removal and, if levels are allowed fall significantly, conditions can become anaerobic and can result in bad odours.

Nutrient Removal

After the BOD loading of wastewater has been reduced, the nitrogen and phosphorus within the wastewater remains to be removed. There are several reasons for the removal of these nutrients, not least because both act as fertilizers and promote the growth of algae in receiving waters, leading to eutrophication. Nitrogen also exerts a BOD on the receiving water known as the nitrogenous biological oxygen demand (nBOD) as distinct from the carbonaceous biological oxygen demand (cBOD) removed previously.

Three main types of each nutrient are present in wastewater: phosphorus is present as organic phosphorous (P); polyphosphate (P_2O_7); and orthophosphate (PO_4^{3-}). The latter two types of phosphorous come from the detergents that run into drains (Burton et al., 2013). The three types of nitrogen present in wastewater are: organic nitrogen (N), which comes from urea in wastewater; ammonia (gaseous) (NH_3) and ammonium (ionic) (NH_4^+), products of hydrolysed organic nitrogen which vary depending on the pH of the wastewater; and nitrate (NO_3^-) and nitrite (NO_2^-) which are formed by treatment processes and act as fertilisers (Myers, 2010). Due to these differing types of nitrogen present in wastewater, nitrogen is measured in three ways, all of which are measured in mg/L. The first is as total nitrogen (TN), which is the total of all types of nitrogen. The second measure is the total Kjeldahl nitrogen (TKN) which is a measure of the ammonia, ammonium and organic nitrogen present. Finally, there is the total inorganic nitrogen (TIN), which is the sum of the ammonia, ammonium, nitrite and nitrate present (Myers, 2010). TKN can be considered a measure of the nitrogen entering the plant and lies in the range of 25 to 45 mg/L (Burton et al., 2013). A typical value would be 40 mg/L TKN and most of this, approximately three quarters, is in the form of ammonia. This has been formed by hydrolysis of urea in the time taken to reach the inlet and is relatively easy to treat. Of the remaining 25% organic nitrogen (10 mg/L) 4.8 mg/L is biodegradable nitrogen, 4 mg/L is recalcitrant particulate nitrogen, which is difficult to treat. The remaining 1.2 mg/L is recalcitrant dissolved organic nitrogen (rDON) (Blacoh University, 2015).

The hydrolysis of organic nitrogen happens almost instantly in the presence of water and produces ammonia (NH_3) as a result. Gaseous ammonia, in the presence of water, is then almost entirely converted into ionised ammonia (i.e. ammonium NH_4^+), although the ratio of ammonia to ammonium is dependent on the pH level, with more acid producing more ammonium and more base producing more ammonia. Wastewater is typically in the range of 6 to 9 pH so most of the ammonia will be present as ammonium (NH_4^+) (Myers, 2010). There are a few ways that nitrogen can be removed from wastewater categorised as chemical and biologic methods. Chemical methods include air-stripping, chlorination and ion-exchange, but biologic methods are more commonly used (Blacoh University, 2015). The first step in this biologic removal of nitrogen from wastewater is the BOD removal process by aerobic oxidation previously outlined. During this process biomass is broken down into CO_2, H_2O and NH_3 by heterotrophic bacteria. Heterotrophic means these bacteria consume organic sources of carbon, similar to humans and other animals (Myers, 2010). The next step of the nitrogen removal process is the nitrification process which depends upon certain autotrophic bacteria, so-called nitrifiers or nitrifying bacteria. Autotrophic organisms are those that consume inorganic substances and transform them into suitable foodstuffs, such as plants (Myers, 2010). For nitrification to take place good BOD removal must first be established, otherwise the heterotrophic bacteria will compete with the autotrophs. Autotrophs require specific conditions and require longer to reproduce, which leads to longer for nitrification when compared with CAS systems. Autotrophs will grow faster in higher temperatures and with higher DO levels, leading to an SRT between five and ten days, while lower temperatures and DO levels lead to a higher SRT, between fifteen and twenty days. If the SRT is too low the autotrophic bacteria will not have sufficient time to reproduce, leading to their washout from the system and limited nitrogen removal (Myers, 2010).

A simplified version of the nitrification process is shown as equation (3) and explained in Figure 6, showing that much of the ammonia present is converted to ammonium, which is then converted to nitrite and nitrate by the autotrophic bacteria previously mentioned (Myers, 2010; Ovivo, 2016).

$$NH_3 \rightarrow NH_4^+ \rightarrow NO_2^- \rightarrow NO_3^- \tag{3}$$

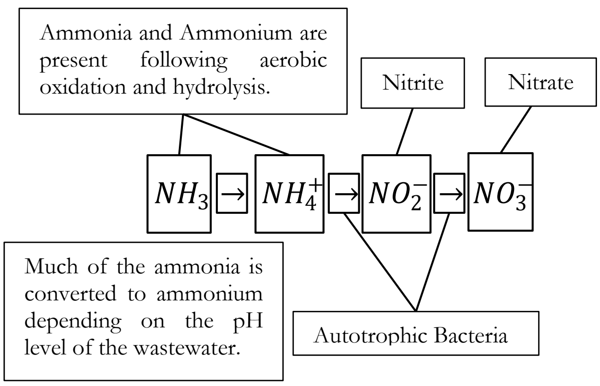


Figure 6: Simplified Nitrification Process

There are two distinct steps in the nitrification process however, the equations for which are shown here as equations (4) & (5) and explained further in Figure 7 & Figure 8 (Myers, 2010; Ovivo, 2016). Both steps require dissolved oxygen and this oxygen demand makes the process an energy intensive one. Aeration for the nitrification processes can be either a separate aeration basin from the BOD removal aeration, or a combined stage where aeration for both takes place in a larger, single aeration basin. Many aeration basins are combined BOD removal and nitrification systems and there are common examples of separate systems such as the trickling filter. In a trickling filter system water flows over a bacteria retaining media with BOD removal occurring at the top of the system and nitrification occurring at the bottom (Burton et al., 2013).

$$NH_4^+ + O_2 \rightarrow H^+ + H_2O + NO_2^- \tag{4}$$

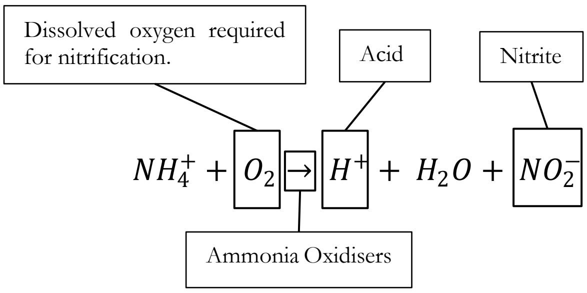


Figure 7: Part One of the Nitrification Process

$$NO_2^- + O_2 \rightarrow NO_3^- \tag{5}$$

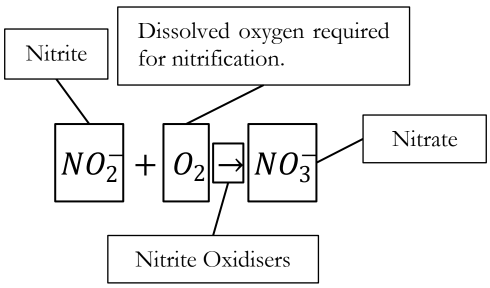


Figure 8: Part Two of the Nitrification Process

After the nitrification process is complete, there is still nitrogen present in the wastewater in the form of organisms and nitrate. This nitrogen loading can still contribute to eutrophication of receiving waters and cause other complications (Myers, 2010). The acid produced during the nitrification process is also toxic to the bacteria in the reactors and must be removed (Myers, 2010; Ovivo, 2016). For these reasons one final step in the nitrogen removal process is used: denitrification.

Denitrification is an anoxic process that uses heterotrophic bacteria under specific conditions to change the nitrate remaining in the wastewater to nitrogen gas (N_2), carbon dioxide (CO_2), hydroxide (OH^-) and some ammonia (NH_3) (Burton et al., 2013; Ovivo, 2016). Anoxic means that the bacteria require an environment where DO is not present, but O_2 is present in the form of nitrates. (Myers, 2010). As the denitrifying heterotrophic bacteria feed on biomass and all the DO has been consumed, the bacteria begin to use the nitrate as a source of oxygen (Myers, 2010). An unbalanced chemical equation for the process is shown as equation (6) and outlined in Figure 9 (Burton et al., 2013; Myers, 2010).

$$C,O,H,N \; + \; NO_3^- \rightarrow N_2 + CO_2 + NH_3 + OH^- \tag{6}$$

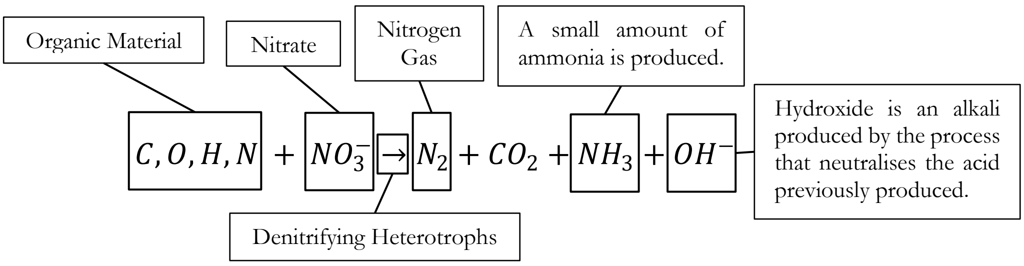


Figure 9: Denitrification Process

The anoxic areas required for denitrification can either be created as separate sections during construction or can be achieved by turning off aeration in sections of basins during treatment (Myers, 2010). The anoxic zone may also be included early in the secondary treatment phase of treatment because of the denitrification process' requirement for organic carbon to feed the heterotrophs. Two processes of denitrification that use this configuration are the modified Ludzack-Ettinger (MLE) process and the four stage Bardenpho process (Blacoh University, 2015).

Centralised or De-centralised Wastewater Treatment

New approaches to wastewater management are being discussed and reviewed in light of increasing populations in developing countries. These approaches are not tied to the centralised wastewater infrastructure that have become the norm for developed countries, particularly in Europe and the United States (Chocat et al., 2004). Centralised systems are far more prevalent in developed countries, in the US 22% of plants treat more than 1 million gallons per day (MGD), serving 85% of the population, while 70% of facilities are smaller and serve just 10% of the population (Gude, 2015). An ongoing discussion has emerged regarding the merits or otherwise of centralised and decentralised treatment facilities (Libralato et al., 2012; Sharma et al., 2010). Systems have been found to benefit from economies of scale in terms of energy consumed for treatment (Mizuta and Shimada, 2010), though this is dependent on conveyance and pumping along the connected sewer network (Vaccari et al., 2018). Venice serves as an example of a European city served by a largely decentralised system due to difficulties with conveyance. Wastewater is treated in over 140 decentralised treatment plants and additional septic tanks, since the high water table makes a large sewer network impractical (Libralato et al., 2012). It is generally accepted that centralised systems will continue to dominate where there is existing collection system infrastructure, whereas decentralised systems are preferred in areas where there is little to no existing network. However, as the economic viability of decentralised systems continues to improve and the maintenance and refurbishment of larger collection systems remains costly and disruptive there may be continued debate about the merits of both for some time to come.

Water Quality Regulation

Wastewater treatment has become a necessity of modern life if public health and environmental quality are to be maintained (Breinholt et al., 2008; Chocat et al., 2004; Panepinto et al., 2016). Increased awareness of the environmental burden of wastewater over time meant that WWTPs became ever more regulated in terms of the legislation and policies that govern them. As the field of wastewater management developed, so too did the objectives of treatment, while the regulations governing it were reactive in nature. Initially the regulatory requirements that emerged were the treatment of wastewater based on those pollutants that could be seen with the eye or under a microscope, broadly categorised as the removal of suspended solids. As the processes behind aerobic oxidation were first noticed, if not understood, minimum standards for COD and BOD were established. More recently, the contribution of nitrogen and phosphorus to the eutrophication and pollution of waters was understood and legislation that followed regulated their discharge (Lucas et al., 2014). As these requirements for facilities increased, it has been found to increase the chemical and energy resource requirements for treatment (Lorenzo-Toja et al., 2016). This resource consumption can be expected to increase as environmental regulations continue to develop. There will also be an additional capital or infrastructural cost as regulations change, as facilities need to change to meet these requirements (Arroyo and Molinos-Senante, 2018). It is expected that future developments in water quality regulation will seek to continue to meet previous requirements, detect and treat emerging contaminants and maintain consistent plant performance (Burton et al., 2013).

In Europe current regulations are driven in part by the EU's Water Framework Directive (WFD) which sets out the principles of treatment and discharge to be enacted by member states at a national legislative level (Pleau et al., 2005) and requires that member states ensure no deterioration in water quality as a minimum, except in cases of "force majeure" (Parliament and Council Directive 2000/60/EC). The WFD also establishes the river basin level as the administrative framework around which water resources should be managed and requires member states to do so, considering the impacts of human activity and the hydrologic cycle (Parliament and Council Directive 2000/60/EC). The WFD is very much an important part of the governance of wastewater treatment, but Annex VIII and X are particularly so. Annex VIII is the "Indicative list of the Main Pollutants" and this sets out the broad categories of pollutants considered. Annex X is a list of priority substances and has subsequently been amended by Directive 2008/105/EC and Directive 2013/39/EU. These amending directives dealt with "environmental quality standards in the field of water policy" and were in response to the growing awareness of emerging contaminants and their environmental impacts (Parliament and Council Directive 2000/60/EC, Parliament and Council Directive 2008/105/EC, Parliament and Council Directive 2013/39/EU). In the context of wastewater treatment Annex VIII is supplemented by Directive 91/271/EEC, the Urban Wastewater Treatment Directive (UWWTD). The UWWTD lists the specific constituents to be treated for by WWTPs as well as a number of other requirements (Council Directive 91/271/EEC) and is considered a driver of the employment of nutrient removal processes in Europe (Borzooei et al., 2019).

Developments in treatment are continuing, with wider environmental sustainability, energy reductions and resource recovery all now being considered as objectives for treatment (Gude, 2015; Panepinto et al., 2016; Quadros et al., 2010; Read, 2004; Tilley, 2011), and these additional aims can be expected to influence future regulations. There is currently little or no legislation around the world that uses degradation of the environment as a metric for consideration. As holistic and integrated approaches to management at a river basin level are developed, regulations can also be expected to take account of this and can be facilitated by integrated modelling of UDSs (Rauch et al., 2002).

Energy Use for Wastewater Treatment

The methods that have evolved for treating wastewater don't come without cost and modern WWTPs employ resource intensive processes in terms of water, chemical use and energy use (L. Fitzsimons et al., 2016; Gude, 2015; Longo et al., 2016; O'Doherty et al., 2014; Torregrossa et al., 2016). Traditionally the energy intensity (EI) of wastewater treatment has not been considered when designing wastewater treatment systems, as the efforts have been typically focussed on effluent quality (Panepinto et al., 2016; Vaccari et al., 2018) or on economic and technical aspects (Arroyo and Molinos-Senante, 2018). Energy costs account for 25 to 40% of the operating costs of conventional wastewater treatment plants (Panepinto et al., 2016). Energy and resource costs have been the subject of recent research, primarily because they have been the driver in decision making for water utilities as economic costs can be reduced and because reductions typically provide a good return on investment (Lorenzo-Toja et al., 2016; Molinos-Senante et al., 2016, 2014; Panepinto et al., 2016; Wang et al., 2016).

Overall the treatment of wastewater in European countries has been estimated to be 1% of the total energy consumption nationally, with compressors, pumps, valves and ancillary machinery used in treatment consuming energy (L. Fitzsimons et al., 2016; Lorna Fitzsimons et al., 2016; Longo et al., 2016; Wang et al., 2016). There are also environmental concerns in relation to greenhouse gas emissions from the wastewater treatment sector, though these have not traditionally been considered in decision making. While energy production contributes a significant amount to greenhouse gas emission (Wang et al., 2016), another 45 million tonnes of CO2 emissions are linked to the biological processes used in treatment and are not typically considered in emission accounting (Gude, 2015). With increasing intensification expected in both the energy and water sectors in the next number of years there has been increasing interest in the recovery of energy and other resources from wastewater in recent decades (Gude, 2015; OECD/IEA, 2016; Wang et al., 2016). It has also been concluded (Nakkasunchi et al., 2021) that the wastewater sector can be completely decarbonised by implementing combined energy saving and renewable energy production. The challenge identified in this regard is access to a decision support tool to help managers identify and prioritise best strategies (Nakkasunchi et al., 2021).

The Energy Intensity (EI) of wastewater treatment taken from the literature is shown here in Table 1. Global values for treatment, national breakdowns, various treatment technologies EIs', and process level values for a single plant in Germany are shown. These values show a broad range of EI values and the variation even with similar technologies, as well as the regionality of water and resource management. There are a number of factors that drive the EI of treatment including the type of treatment processes, the deployment of energy recovery methods, influent concentration and the location and scale of facility (Gude, 2015; Mizuta and Shimada, 2010; Molinos-Senante et al., 2018, 2014). For conventional activated sludge processes, EI is driven mainly by annual flow rate treated and BOD removal rate, but also by facility age and suspended solids removal (Molinos-Senante et al., 2018), and increased nutrient removal requirements brings additional resource costs (Gude, 2015). Energy requirements per unit volume treated also decrease with increasing plant size, though the variation in energy requirements even between similar plants can be great (Gude, 2015; Molinos-Senante et al., 2018) and the variation between high and low EI values is also greater for smaller plants than larger plants (Longo et al., 2016). Plant size has also been shown to have a greater influence on EI than process selection when conventional activated sludge and oxidation ditch plants of similar capacity were compared by (Mizuta and Shimada, 2010).

Table 1: Energy Intensity of WWT

Global Energy Intensity		
	Conventional Treatment	Between 0.13 and 2.1 kWh/ m^3 *
		Between 0.3 and 2.1 kWh/ m^3 **
	wo/ nutrient removal	< 0.5 kWh/ m^3 ***
	w/nutrient removal	Between 0.3 and 2.0 kWh/ m^3 ***
		Increase of 50% energy consumed compared with similar plant without **
	Municipal Wastewater	0.45 kWh/ m^3 *†
	Industrial Wastewater	0.65 kWh/ m^3 *†
	Additional energy for water recycling	Between 0.5 and 2.0 kWh/ m^3 ***
National Energy Intensity		
	USA	0 - 1.12 kWh/ m^3 ††
	China	0.13 – 0.5 kWh/ m^3 ††
	Germany	0.4 – 0.43 kWh/ m^3 ††
	South Africa	0.08 – 1.03 kWh/ m^3 ††
	Italy	0.29 – 0.96 kWh/ m^3 (AS Only) *†*
Treatment Technology		
	Lagoons	0.079 – 0.28 kWh/ m^3 ††
		0.09 – 0.29 kWh/ m^3 ***
		~48 – 66 kWh/PE_{BOD60}/yr ††*
	Trickling Filters	0.18 – 0.41 kWh/ m^3 ††
		0.18 – 0.42 kWh/ m^3 ***
		~18 – 152 kWh/PE_{BOD60}/yr ††*
	Activated Sludge	0.33 – 0.61 kWh/ m^3 ††
		0.33 – 0.6 kWh/ m^3 ***
		0.29 – 0.96 kWh/ m^3 *†*
	Activated Sludge w/ incineration	~34 – 118 kWh/PE_{BOD60}/yr ††*
		0.3 – 1.89 kWh/ m^3 †*
		0.38 – 1.49 kWh/ m^3 †*
	Oxidation Ditch/EA Plant	0.48 – 1.03 kWh/ m^3 ††
		0.44 – 2.07 kWh/ m^3 †*
	Advanced Treatment	0.31 – 0.4 kWh/ m^3 ***
		0.39 – 3.74 kWh/ m^3 †*
	MLE Activated Sludge System	~ 0.03 kWh/ m3 (calculated) *††
Process Energy		
	Pumping	0.005 kWh/ m^3 ††
		0.006 kWh/ m3 (~ 23%) *††
	Influent Pumping	0.022 – 0.042 kWh/ m^3 (5 – 18%) †*†
	Aeration	0.18 – 0.8 kWh/ m^3 (45 – 75%) †*†
		0.296 kWh/ m^3 ††
		0.02 kWh/ m^3 (~ 73%) *††
		50% of energy consumed ***
		55-70% of energy consumed **
	Sludge Thickening	0.001 kWh/ m^3 ††
	Digester Mixing and Sludge Pumping	0.002 kWh/ m^3 ††
	Sludge Pumping	15.6% of energy consumed **
	Sludge Aeration	0.00017 kWh/ m^3 ††
	Sludge Dewatering	0.0037 kWh/ m^3 ††
		7% of energy consumed **
	UV Disinfection	0.045 – 0.11 kWh/ m^3 †*†
	Chemical Dosing	0.009 – 0.015 kWh/ m^3 †*†

*(Molinos-Senante et al., 2018, 2014)
** (Panepinto et al., 2016)
*** (Gude, 2015)
*† (Liu et al., 2016)
†† (Wang et al., 2016)
†* (Mizuta and Shimada, 2010)
† (Vaccari et al., 2018)
*†† (Borzooei et al., 2019)
†*†(Longo et al., 2016)
††*(Krampe, 2013)

Even at an individual process level it can be difficult to compare the EI of similar treatments due to variability in reported EI. For example, while it has been noted in previous work (Foladori et al., 2015) that aeration and pumping are considered by many to be the greatest users of electricity consumption within WWTPs, they may account for less than 40% of energy consumed in some instances. Due to this variability any proper assessment of EI of a plant would require a thorough energy audit of the facility and such an audit would be a prerequisite for any process level benchmarking as is suggested by (Foladori et al., 2015) and shown in the work of (Panepinto et al., 2016). Greenhouse Gas (GHG) emissions are also becoming an important factor in assessing wastewater treatment (Flores-Alsina et al., 2011). Energy reductions in treatment may reduce that component of the GHG emissions related to treatment but can have detrimental impacts in other areas e.g. increased N2O emissions. As the requirements of wastewater treatment and the need for energy efficiency in treatment continues to grow, tools for benchmarking and auditing plants and processes are being investigated (Longo et al., 2016). The treatment of wastewater will become an ever more important balancing act between sustainable best practices and cost saving.

Variation in WWTP Influent

Wastewater treatment can be considered as a set of sequential and/or parallel processes that ultimately reduce the pollutant concentration in the effluent discharged. As mentioned previously these processes are subject to variation in several ways, influent variation being one that has many operational implications. Influent variance can come from a few sources and can be broken down into two categories: expected and unexpected. Expected variation can occur over periods of a day or more and are natural diurnal and seasonal changes in influent at a WWTP, in part due to societal activities but also because of other factors like the water table level in the catchment area (Młyński et al., 2016; Stricker et al., 2003). These variations are expected in so far as they can be estimated ahead of time based on previous experience and measurements and these methods have been used in the design and planning phase of WWTP construction for some time (Młyński et al., 2016; Munksgaard and Young, 1980). These predictions are still only estimates of expected flows and still subject to additional variability from unexpected sources. There are several sources for these unexpected variations in the flows to treatment, some of which are: industrial discharge; power losses; mechanical breakdown; recycled sludge; and rain events (Burton et al., 2013; Lucas et al., 2014). These types of variation are unexpected in as far as the timing of their occurrence may not be predictable, but efforts are made to account for them when designing facilities. This can lead to the infrastructure being oversized however which will have implications on efficiency (Longo et al., 2016).

Probably the most common cause for variability in WWTPs are periods of wet weather (Burton et al., 2013; Rouleau et al., 1997). If consistent plant performance is to be achieved, then variability due to such unexpected events should be minimised, particularly if these events are responsible for changes in flows, plant performance and operation or the ultimate failure of plant processes (Giokas et al., 2002; Rouleau et al., 1997). The problem of increased flows due to rainfall may become particularly acute in areas where rainfall may increase due to climate change. Local weather conditions are of such importance to WWTP operation that characterisation of flows to treatment are already based on them (Giokas et al., 2002). Flows during periods of low precipitation are described as Dry Weather Flows (DWFs), while periods of precipitation bring Wet Weather Flows (WWFs). WWFs are typically a combination of the base DWF diluted by the presence of increased ingress and infiltration. Combined and separated collection systems are influenced by these changes in flow as the intended and unintended flow of storm water, known as inflow and infiltration respectively, is typically unavoidable. The effects are typically more apparent in combined collection systems however as peak flows are greater as these systems combine storm and sanitary flows by design. Exfiltration, or the unintended leaking of water away from the network, is also common to both systems (Giokas et al., 2002; U.S. Environmental Protection Agency, 2007) and does pose problems for treatment also (Młyński et al., 2016).

Research has looked at the influence of WWF on wastewater stream, though there appears to be little specific focus on the energy requirements to treat these flows. Some of the results of this research has been shown here (Bertrand-Krajewski et al., 1995; Giokas et al., 2002; Lessard and Beck, 1990; Młyński et al., 2016; Nielsen et al., 1996; Panepinto et al., 2016; Rouleau et al., 1997; Stricker et al., 2003; U.S. Environmental Protection Agency, 2007):

- Increased Hydraulic Loading
- Changes in quantity and quality of sludge in primary and secondary treatment
- Changes to F/M ratio
- Sludge blanket overflows or washout from processes
- Sedimentation influence
- Decreased plant performance and removal efficiencies
- Increase in energy consumed in treatment plant
- Increase in untreated discharges to receiving waters

A summary of these points is that both rainwater ingress and infiltration impact the influent quantity and quality, both of which are critical to the effective operation of the treatment plant. Some of the changes in the influent and effluent quality that have been reported in the literature are shown in Table 2.

Even considering treatment without any information on the energy input for treating wastewater, the change in removal rates due to varying influent quality means that WWFs will have an impact on the benchmarking of WWTPs. Where it can be shown that decreasing removal rates, coupled with an increase in energy required for treatment during WWF periods, the effects could be significant. The release of biomass or sludge from the WWTP is of concern to operators as these events can have longer term effects in addition to the impacts immediately upon release. Increased hydraulic loading is typically dealt with by modifying the return sludge rates of processes and by employing step-feed systems, but there are limitations to these approaches. Reaction rates can be slow, sometimes taking hours to implement, and there can be detrimental impacts on removal rates (Nielsen et al., 1996). It has been suggested that the impacts of WWFs on WWTPs can be mitigated by the implementation of forecasting systems based on catchment area measurement of rainfall and flow coupled with suitable control strategies (Nielsen et al., 1996). So-called "first flush" events occur when materials that have been resting in sewer networks during dry periods are then flushed out during WWFs. These events could potentially be exploited with better information regarding the behaviour of collection systems during differing flows, although their effects may be sometimes overestimated (Nielsen et al., 1996). Differing reports of the impacts of first flush events could be indicative of the heterogeneous nature of UDSs and the catchments they serve. This would highlight the need for better understanding of catchment areas as well as the need for more bespoke solutions based on the needs of catchments as opposed to the more traditional standardised approaches to wastewater treatment based.

Table 2: Influent/Effluent characteristics during WWFs (Daly et al., 2018)

	Influent	Effluent
Flows	Increased flows (Richard O Mines Jr et al., 2007; Stricker et al., 2003) Increased variability (Stricker et al., 2003)	
BOD	Concentrations unchanged or decreased* (Bertrand-Krajewski et al., 1995; Richard O Mines Jr et al., 2007) Loads increased (Up to 3x Dry Load) (Bertrand-Krajewski et al., 1995)	Concentration increased** (Richard O Mines Jr et al., 2007)
COD	Increased variability (Up to 2x Dry Load) (Stricker et al., 2003) Average 200 kg d^{-1} (27%) increase (Stricker et al., 2003) Concentrations unchanged or decreased (Bertrand-Krajewski et al., 1995; Rouleau et al., 1997) Increased total COD concentration, little or no change to soluble COD (Borzooei et al., 2019)	Average 20 kg d^{-1} increase (Stricker et al., 2003) Increased concentration (Rouleau et al., 1997) Increased COD due to soluble COD (Stricker et al., 2003)
TSS	Increased loading (Up to 10x Dry Load) (Bertrand-Krajewski et al., 1995) Mean concentrations > 1000 mg l^{-1} (Bertrand-Krajewski et al., 1995) Decreased concentration* (Richard O Mines Jr et al., 2007; Rouleau et al., 1997)	Up to 7x Dry Loads (Bertrand-Krajewski et al., 1995) 10 kg TSS for 36 hour event (Bertrand-Krajewski et al., 1995) Average 6 kg d^{-1} increase (Stricker et al., 2003) Increased concentration*** (Richard O Mines Jr et al., 2007; Rouleau et al., 1997)
N	Decreased ammonia concentration (Bertrand-Krajewski et al., 1995) Increased ammonia loading (Up to 1.2x Dry Load) (Bertrand-Krajewski et al., 1995) Increased ammonium concentration (Borzooei et al., 2019) Decreased ammonium concentration (Rouleau et al., 1997) Increased TN concentration (Borzooei et al., 2019)	Ammonia barely affected (Stricker et al., 2003) Nitrate increased Decreased ammonium concentration (Rouleau et al., 1997) Decreased nitrate concentration (Rouleau et al., 1997)
TKN	Increased variability (Up to 2x Dry Load) (Rouleau et al., 1997; Stricker et al., 2003) Average 25% load increase (Stricker et al., 2003) Max. 105 kg d^{-1} (Stricker et al., 2003) Decreased concentration (Rouleau et al., 1997)	Increased concentration (Rouleau et al., 1997)
TP	Concentrations unchanged or increased (Borzooei et al., 2019)	

* (Richard O Mines Jr et al., 2007) Based on corresponding flow rate increase
**(Richard O Mines Jr et al., 2007) Based on corresponding BOD_{Inf} concentration increase
*** (Richard O Mines Jr et al., 2007) Based on correspjustonding TSS_{Inf} concentration increase

Energy Recovery

It has been known for some time that wastewater contains energy of different forms, so in the context of increasing awareness of the energy-water nexus, energy recovery technologies are also currently under investigation. It is thought that wastewater contains approximately nine times the energy required to treat it (Gude, 2015), which could make wastewater treatment a net energy producer (Wang et al., 2016). The potential for energy recovery again varies regionally and three basic categories have been outlined in Table 3, along with the potential energy availability.

Table 3: Energy Recovery Potential

<table>
<tr><td>Thermal Energy</td><td>Availability:
Up to 49,000 MJ/m^3 per 10 degree temperature difference (Gude, 2015).</td></tr>
<tr><td colspan="2">Notes:
In the US and Canada 400 billion kWh of thermal energy is estimated to be washed down the drains in one year as waste heat, far exceeding the energy required to treat wastewater in the US (Gude, 2015).The impacts of this method of recovery on treatment methods downstream needs to be assessed thoroughly, as well as the economic benefits (Gude, 2015). CHP systems can be coupled with the production of Biogas generated by anaerobic digestion processes in WWTP to generate heat and electricity (Gude, 2015).</td></tr>
<tr><td>Hydraulic Energy</td><td>Availability:
Location Dependent.</td></tr>
<tr><td colspan="2">Notes:
Depending on the height differential between the influent, treatment and discharge points there is the potential for energy recovery devices to be placed to capture kinetic energy from water. This has been shown to be a net energy producer over significant drop (400 ft) (Gude, 2015).</td></tr>
<tr><td>Chemical Energy</td><td>Availability:
Between 12 – 15 MJ/kg COD (organics)
Between 27.4 – 29.4 MJ/kg suspended solids (Gude, 2015).
32 to 36 L/PE/d Biogas produced (Krampe, 2013)
155 kWh/PE y (Krampe, 2013)
~0.1 kWh/m3 (Wang et al., 2016)</td></tr>
<tr><td colspan="2">Notes:
One example of exploiting the chemical energy of wastewater is through the use of Anaerobic digestion (AD). AD has been around for at least 20 years and plays a key part in energy recovery from wastewater, producing 25 to 50% of the energy required for aerobic treatment in the form of biogas (CH4) (Gude, 2015; Wang et al., 2016). AD processes have been reported to produce 65% methane (CH4), 30% CO2 and 5% hydrogen sulphide gas (H2S) (Wang et al., 2016). AD has been underutilised however because of poor effluent quality and nutrient removal (Gude, 2015). This biogas can then be used for heat or electricity generation or to power vehicles (Gude, 2015).</td></tr>
</table>

Energy Benchmarking

The energy benchmarking of wastewater treatment plants can be defined as being a continuous process of comparing the performance of a representative selection of plants or processes for the purposes of learning and identifying efficiencies and best practices for technologies or plants (Andersen, 1999; Krampe, 2013; Lindtner et al., 2008; Longo et al., 2016; Molinos-Senante et al., 2014; Torregrossa et al., 2016). If correctly implemented the benchmarking of wastewater treatment can account for the consumption of resources, the performance of the processes used and can also inform optimisation efforts (Doherty et al., 2017; Vaccari et al., 2018). Some methods of benchmarking are more relevant than others under different circumstances (Andersen, 1999), so two types of benchmarking are considered in this thesis: performance benchmarking, where key metrics and numbers are directly compared at a macroscopic level; and process benchmarking, where individual processes within an organisation can be examined (Andersen, 1999; Lindtner et al., 2008; Quadros et al., 2010).

Previous attempts at benchmarking have been reported as being "fragmented and piecemeal" (Vaccari et al., 2018) due to their broad nature, with the necessary data collection and analysis being a long and laborious task aggravated by difficulties with availability (Borzooei et al., 2019). Benchmarking is reliant upon readily available, accurate and reliable data for relevant metrics over suitable time scales, but there have also been problems associated with the availability of such data in wastewater treatment particularly with regard to EI data (Borzooei et al., 2019; Doherty et al., 2017; Lindtner et al., 2008; Molinos-Senante et al., 2018; Torregrossa et al., 2016). This lack of data may be for any number of reasons, such as (Borzooei et al., 2019; O'Doherty et al., 2014):

- Many plants can be small and unmanned or staffed intermittently
- There may be no monitoring of water or energy flows at a process level
- Lack of automated data collection
- Variations in the processes and technology employed make data difficult to gather
- Costs and effort of "unnecessary" data collection can dissuade utilities
- Unreliable data collection or sensors allowed fall to disrepair

Even within water utilities the necessary data may not be easily accessible or even measured. Wastewater treatment is a sector that can be characterised as conservative. As a result, change can be slow and operators can be wary of modifying processes that have a proven track record of achieving treatment objectives. Similarly, the measurements recorded by these utilities are chosen based on their relevance to the objectives of treatment and cost. Advances in on-line sensor and computing technology can be expected to overcome some of these difficulties as they reduce the cost of measurement. One way of dealing with these issues relating to data is to study only what is relevant in detail, thereby keeping the necessary data for analysis to a minimum (Doherty et al., 2017; O'Doherty et al., 2014).

A review of literature regarding the benchmarking of wastewater treatment has been summarised and tabulated here and is shown as Table 4.

Table 4: Energy Benchmarking Papers

Paper	Notes	PIs considered	Results
Benchmarking of municipal waste water treatment plants (an Austrian project) (Lindtner et al., 2004)	76 Plants across Austria were considered. Plants were broken down into PE groups based on COD loading: • < 5,000 • 5,000 to 12,000 • 12,000 to 25,000 • 25,000 to 50,000 • > 50,0000 Describes inter-organisational benchmarking but doesn't refer to process benchmarking. Develops a methodology of comparison across processes and "operational modes". 4 Processes considered: 1. Mechanical Pre. Treatment 2. Mechanical and Biological Treatment 3. Sludge Thickening & Stabilisation 4. Sludge Treatment & Disposal Benchmark bands are established based on experience. Benchmark plants must: • Comply with standards • Meet validity checks	Mean Yearly Load – COD (MYL-COD) described as best technical parameter for PIs related to cost. TN could also be used but data availability is a problem. Real Design Load (RDL-COD) used for process 1,3 & 4	Costs decreased with increasing scales. Capital costs drastically increased between group 1 & 2 due to scale and more reserve capacity. This seems consistent with yearly price drop between group 2 & 3, as larger plants tended to operate closer to design load. "Cost reduction potentials" were calculated. Staff quality was found to be the "most relevant parameter". Aeration control had little influence over the specific power consumption but there was a strong correlation with non-compliant discharges i.e. presence of aeration control reduced non-compliant discharge.

Paper	Notes	PIs considered	Results
	• Have no excessive industrial discharges • Have specific costs inside the benchmark band		
Energy Index Development for Benchmarking Water and Wastewater Utilities (Carlson et al., 2007)	Offers a benchmarking method for water and wastewater utilities. Surveyed 266 wastewater treatment plants in the US Plant dataset was noted to be skewed to the smaller size. Considered the contribution of the collection system to energy consumption, dealt with separately to the treatment plant assessment.	Wastewater treatment model related energy consumption to: • Average Influent Flow • Influent BOD • Effluent BOD • Ratio of influent flow to design flow • Use of trickle filtration • Nutrient Removal Collection system model related energy consumption to: • Average Influent Flow • Pumping Horsepower • No. of Pumps	Treatment Plant: • 1000 to 3000 kWh/MG • $75 to $200/MG Collection System electricity use below 400kWh/MG & $80/MG for most areas surveyed. Second peak in dataset above 1,000 kWh/MG Results of the collection system analysis were less robust than the treatment plant analysis, but reported collection system energy use was less than 25% of energy use in general, with most reporting less than 5%. Recommendation to collect energy data from water utilities on a regular basis (5+ year time frame). Research was to progress with the US EPA into the Energy Star Program.

Paper	Notes	PIs considered	Results
Benchmarking of large municipal wastewater treatment plants treating over 100,000 PE in Austria (Lindtner et al., 2008)	Considered 6 Plants in Austria with capacities over 100,000 PE. Looked at performance indicators (PIs) for one year. Benchmark plants were required to meet discharge requirements and there were to be no dominating industrial influent. Aimed to minimise operating costs based on COD removal.	€/PE_{COD110} €/PE_{Design} kWh/PE_{COD110}	Sludge treatment and disposal are responsible for 40% or more of the total operating costs. 50 to 65% of operating costs were found to be independent of COD loading, meaning utilisation factor is an important metric. No correlation was found between treatment efficiency and operating costs for plants with similar treatment requirements. "Excellent treatment efficiency often coincides with low specific costs…" which is considered an indicator of the importance of good quality staff. Specific costs increase with decreasing PE capacity.
Operational energy performance assessment system of municipal wastewater treatment plants (Yang et al., 2010)	Refers to Performance Assessment System (PAS). 559 WWTPs across China were used in the overall analysis, refined to 10 plants where process level data was available. Energy consumption means electricity consumption in the context of this study. Only plants with compliance of > 85% qualified.	Energy per unit volume treated (E_V) kWh/m^3 Energy per unit mass removed (E_M) kWh/kg* Energy per volume pumped per metre lift (E_P) kWh/m^3/m Aeration energy per volume treated (E_A) kWh/m^3 Sludge processing energy per kg sludge (E_S) kWh/kg Volume of biogas produced per kg sludge (G_R) m^3/kg	Benchmarks scores were all calculated based on the plant values compared with modelled or calculated best achievable or in best achieved in China. Paper only considers three plants in the final assessment of differing technologies and scales. [Unsure of relevance due to low numbers i.e. one plant for each technology.]

Paper	Notes	PIs considered	Results
	Industrial loads >30% or the presence of toxic loadings disqualified plants. Benchmark figures were established using models, calculated values or comparison with highest value achieved in Chinese plants.	Ratio of energy retrieved to energy consumed (E_R) *This metric used a weighted total pollutant mass removed: $(COD+2BOD+2SS+20TN+100TP)V_{Treated}$	Aeration identified as an area for targeting to achieve efficiencies at each plant. Variations in flow are also identified as a problem for all plants.
A performance indicators system for urban wastewater treatment plants (Quadros et al., 2010)	Refers to "Performance Assessment System" (PAS). Follows a "Plan-Do-Check" methodology. Broad Objectives Established: • Discharge Compliance • Minimise Cost Requirements • Ensure Environmental Sustainability Distinguishes between "Overall Performance Assessment" and "Operational Performance Assessment" with the former being an overview assessment (performance benchmarking) and the latter looking at process level (process benchmarking).	Number of PI categories considered: Treated Quality (8 PIs) Plant Efficiency & Reliability (50 PIs) Resource Use (6 PIs) By-Product Management (8 PIs) Financial (9 PIs) Planning & Design (4 PIs)	Proposal of a portfolio of benchmarking PIs.

Paper	Notes	PIs considered	Results
Benchmarking energy consumption in municipal wastewater treatment plants in Japan (Mizuta and Shimada, 2010)	Considered 985 plants in Japan, broken down into categories based on four types of treatment: • Advanced Wastewater Treatment (AWWT) • Oxidation Ditch (OD) • CAS w/ incineration • CAS w/o incineration	Volumetric loading m^3/d Process energy consumption MWh/yr. Influent concentration mg/L Effluent Concentration mg/L Volumetric specific power consumption (SPC) kWh/m^3 BOD loading SPC kWh/kg BOD m^3/d vs. kWh/m^3	Decrease in the specific power consumption ($kWh/m^3_{Influent}$) for increased flows. Energy consumption is closely linked to effluent quality and sludge reduction. Gas produced has a significant impact in reducing energy consumption. The SPC for OD and CAS systems was similar for similarly sized plants despite differing technologies. Implication that centralised systems offer greater energy savings. Negative GHG impacts for increasing effluent limits. Treatment cannot be easily changed, but pumping and sludge processing can. Future work: analysis of the relationship between influent loadings and inflow rate, as well as optimisation of effluent quality and electric power.
Energy benchmarking of South Australian WWTPs (Krampe, 2013)	Benchmarked 20 plants in South Australia. Looked at variety of technologies: • Aerated Lagoons • Trickling Filters • Activated Sludge	$kWh/PE_{BOD60}/yr.$ % Energy Self supplied External Heat ($kWh/PE_{BOD60}/yr.$) Biogas produced (L/PE/d)	Large variability in specific energy consumption (kWh/PE_{BOD60}) Showed that despite good biogas production, inefficient conversion process in turbines yield low energy. kWh/PE/yr. is preferred metric.

Paper	Notes	PIs considered	Results
	16 plants also featured tertiary treatment phases. Benchmarking was against other plants (performance benchmarking) and industry standards from literature.	Process Benchmarks: • Aeration - kWh/PE_{BOD60}/yr. • Pumps - kWh/PE_{BOD60}/yr.	
Benchmarking in wastewater treatment plants: a tool to save operational costs (Molinos-Senante et al., 2014)	A "Comparative Analysis" to identify strengths and weaknesses using simple "partial indicators". Aimed to increase environmental sustainability by reducing resource consumption, as well as financial cost minimisation. Looked at 192 WWTPs in the Valencia region of Spain. Used non-radial DEA analysis of the costs involved. Non-radial analysis gives aggregated information by identifying underperforming inputs. Considered several technology groups: • Activated Sludge (AS) • Peat Bed (PB) • Trickling Filter (TF) • Extended Aeration (EA) • Bio Disc/Rotating Biological Contactor (BD/RBC)	SS removed (g/m^3) COD removed (g/m^3) Total Costs (€/m^3) • Staff Costs (€/m^3) • Reagent Costs (€/m^3) • Energy Costs (€/m^3) • Waste Management Costs (€/m^3) • Maintenance Costs (€/m^3) • Other Costs (€/m^3)	Input efficiency determined by: • Plant Size • Plant Capacity • Technology used – Water & Sludge Lines • kWh/m^3 • Plant Age Greatest energy efficiency with anaerobic digestion due to biogas production. Larger plants are most efficient, medium are least efficient. Staffing costs are most affected by economies of scale. Deteriorating efficiency with plant age. Technology is a factor when considering efficiency in energy, staff maintenance and other costs. Technology Ranking for Energy Efficiency: 1. PB

Paper	Notes	PIs considered	Results
	Also considered sludge treatment categories: • Anaerobic Digestion (AD) • Solar Drying (SD) • Mechanical Dewatering (MD) • w/o treatment (WT) Also benchmarked and categorised based on size (volumetric and PE), energy consumption (based on kWh/m^3), sludge generation (kg/m^3) and age.		2. TF 3. EA 4. BD/RBC 5. AS Plants with good global efficiency also had the lowest energy consumption. Efficiency correlates with reduced costs. Found no evidence that sludge generation effected efficiency. Indicate energy efficiency usually correlates with high efficiency in maintenance and waste management areas. Younger plants showed a higher global efficiency.
Eco-efficiency analysis of Spanish WWTPs using the LCA + DEA method (Lorenzo-Toja et al., 2015)	Initially 470 plants across Spain were considered, before being narrowed down to 113 based on a range of issues (primarily data gaps and validity). Applied a combination of the LCA methodology to identify the main environmental impacts throughout life cycle. Combined with data envelopment analysis (DEA) to establish operational benchmarks through frontier analysis. Part of the EU funded AQUAENVEC project.	Inputs: • Electricity Use (kWh/m^3) • Environmental impacts of Chemical Consumption (Pt/m^3) • Sludge Production (kg/m^3) Outputs: • NEB of Eutrophication Potential (kgN$_{eq}$/m^3)	Tertiary treatment plants were removed from sample as the third step did not improve effluent and aggregated data was not available for process analysis. Small treatment plants had a higher variability in the LCI results due to heterogeneity and available data. Smaller plants also appear subject to a wider variety of operational factors, as opposed to the relatively more consistent medium and large plants (although these were still had high standard deviations).

Paper	Notes	PIs considered	Results
	Aim is to support decision making by providing target operational values, estimate environmental improvements and identify specific improvement actions. Used a Net Environmental Benefit (NEB) approach, basically comparing a lack of treatment with the treatment employed. Scale segregated: • < 20,000 • 20,000 to 50,000 • > 50,000		Most of the DMUs were in the 25 to 75% efficiency range. The efficiency was impacted by the size, influent characteristics, climate region, technology, age and operational practices. Average efficiency scores: • Small (77 Plants) – 31.6% • Medium (14 Plants) – 49.3% • Large (22 Plants) – 51.6% The environmental performance of a plant is probably impacted by a number of factors and not by any single one. No clear link between underperformance and over-capacity, except in more extreme cases (>300% capacity). Simpler technologies tended to be more efficient for smaller plants, while more complex technologies were more suited for medium and large plants. The operational management of smaller plants may have been a factor. Milder temperatures and low influent loads in the Atlantic regions appear to

Paper	Notes	PIs considered	Results
			be more favourable for efficiency than in other areas.
Energy performance indicators of wastewater treatment: a field study with 17 Portuguese plants (Silva and Rosa, 2015)	Looked at 17 plants in Portugal over 5 years. Refers to performance assessment system (PAS), using indicators from Quadros et. Al. (Quadros et al., 2010). While BOD and COD are used for assessments, volumetric is preferred because of data accuracy and availability because BOD and COD measurements are discontinuous. Treatment clustered based in scale and treatment type: • Activated Sludge (AS) w/ primary sedimentation • AS w/o primary sedimentation • Trickling Filters (TF) • Bio Filters (BF) • Volumes > 10,000 m^3/d	wtRU03.1 – Energy consumption (kWh/m^3) wtRU03.2 – Energy consumption (kWh/kg BOD_5 removed) wtRU03.3 – Energy consumption (kWh/COD removed) wtBP18.1 – Energy produced from biogas (kWh/m^3) wtBP18.2 – Energy production from biogas (%) wtER08 – Net energy from external (kWh/m^3) wtFi05 – Electric energy cost (€/m^3) wtEF03 – BOD_5 removed/volume (kg BOD_5/m^3) wtEF04 – COD removed/volume (kg COD/ m^3)	AS w/o primary sedimentation showed twice three times energy consumption of AS w/ primary sedimentation. Decrease in energy m^3 with increasing plant size. Increasing efficiency of BOD removal for increased BOD loading. Increase in energy m^3 with energy per kg BOD removed.

Paper	Notes	PIs considered	Results
	• Volumes < 10,000 m^3/d		
Assessing the efficiency of Chilean water and sewerage companies accounting for uncertainty (Molinos-Senante et al., 2016)	Considered data from Chilean water companies, looking at data from 23 of them. Most assessments use non-parametric digital envelopment analysis (DEA), but this type of analysis does not consider the "functional relationship" between inputs and outputs. It is deterministic which means that it is sensitive to "atypical observations". DEA is typically input orientated (minimisation of inputs) or output orientated (maximising outputs). Input oriented is typical choice based on the nature of WWTPs. There is a gap in the literature regarding data uncertainty in efficiency assessments.	Note: Considers the supply & distribution of water as well as wastewater. Inputs • Operating Costs • Labour (n of employees) • Network Length (km) Outputs • Volume of water distributed (m^3) • Wastewater customers (n) • Drinking water quality indicator • Wastewater treatment quality indicator	DEA does not allow for ranking by itself, as the efficiency is determined based on a number of inputs. It measures the relative inefficiency compared to best practice or benchmarks. It is essential that uncertainty is accounted for in the data for the benchmarking or assessment of water company efficiency.

Paper	Notes	PIs considered	Results
Energy saving in WWTP: Daily benchmarking under uncertainty and data availability limitations (Torregrossa et al., 2016)	Benchmarking as a comparison of PIs for a representative sample of plants for determining target values. Uses the Energy Online System (EOS) which is a system designed to save energy in wastewater treatment and features a number of KPIs. Lack of available data has been problem for operating EOS successfully. Considers COD one of the most important indicators of organic contamination in wastewater treatment. PE is calculated based on regional COD values (135 gCOD/PE/d is used here).	EOS has a set of 54 KPIs based on daily loads, not the plant under investigation. 20 of these are energy related and 34 are process related. Flow (m^3/day) COD (avg. mg/L/day) Daily Energy (kWh/d) PE_{Calc} (n) Energy (kWh/PE) Pump Specific Energy – kWh/m^3/m_{Lift}	Calculated or estimated values for influent characteristics can have advantages over lab based measurements e.g. time critical. Can be properly and precisely implemented when benchmark definitions are improved. Broad aggregations or clustering of data is needed.
Increasing Resource Efficiency in Wastewater Treatment Plants (Lorna Fitzsimons et al., 2016)	Assessed 10 WWTPs in Ireland for energy consumption and water quality using several methodologies: • Benchmarking • Plant Auditing • Life-Cycle Analysis (LCA) • Exergy Analysis	Benchmarking was done using key performance indicators based on the developed benchmarking system. This comprises two parts: KPI Advisor – assesses available data and provides relevant KPI recommendations based on availability as well as confidence intervals based on data accuracy. KPI Calculator – calculates, validates and reports the results based on the relevant KPIs.	Noted a lack of available data can hinder the process of benchmarking or auditing. Noted energy economies of scale with increasing plant size, although a larger plant sample size would produce better results. Two Plants of similar scale and technology (E & F) showed significant differences in perceived performance when compared. Identified a significant difference in the influent quality to both.

Paper	Notes	PIs considered	Results
			Influent composition can greatly affect the interpretation of results and the defined plant performance. For the plants using CAS system: • Higher carbon loading may have comparatively lower $kWh_{Consumed}/kg\ BOD_{Removed}$ reported. • Lower organic loadings and higher hydraulic loads may have comparatively lower $kWh_{Consumed}/m^3_{Treated\ wastewater}$. Accurate flow data and compliance monitoring are key to benchmarking operational performance or resource consumption. While purchase, calibration and maintenance of energy monitoring equipment may be obstacles, energy audits proved accurate and important baselines for energy use and management. This can lead to relatively short payback periods.

Paper	Notes	PIs considered	Results
Benchmarking wastewater treatment plants under an eco-efficiency perspective (Lorenzo-Toja et al., 2016)	Looked at 22 plants in Spain. WWTPs have developed over time to become resource intensive systems, typically following policy changes. This has led to increases in resource consumption (e.g. chemical and energy). Life cycle assessment (LCA) is the environmental assessment method of choice due to its flexibility. LCA (ISO 14045) is methodologically open to interpretation. Plants range in scale from 9,000 to 1,067,033 PE. Considers the eutrophication potential the most important impact category for wastewater treatment in LCA.	Eutrophication Net Environmental Impact (ENEI) kg PO_4-3 equiv./m^3 Global Warming Potential (GWP) kg CO_2 equiv./m^3 Cost €/m^3 ReCiPe Endpoint single score mPt/m^3 Graphed: • €/m^3 vs. kg PO_4-3 equiv./m^3 • €/m^3 vs. kg CO_2 equiv./m^3 • €/m^3 vs. mPt/m^3	Medium plants had higher operating costs compared to small and large plants. Facilities with nutrient removal performed better in terms of Eutrophication Potential (EP) removed. Those plants with anaerobic digestion as part of their sludge management plan scored lower than those without, possibly due to concentrated water from sludge line being returned to the headworks. ENEI is higher in medium plants compared with small and larger scale plants. There was no correlation between the ENEI and costs found. Plants with tertiary treatment did not score higher in terms of GWP impacts. Plants with AD as part of sludge management were better performing in terms of GWP when compared with those without. Large plants tended to perform better in terms of GWP. A strong correlation between those plants with low GHG emissions and operating costs was observed.

Paper	Notes	PIs considered	Results
Monitoring and diagnosis of energy consumption in wastewater treatment plants. A state of the art and proposals for improvement (Longo et al., 2016)	388 WWTPs in North America, Asia and Europe were analysed. Following Data gathered: • PE Loading (Design and Actual) • Flowrate (Design and Average) • Influent and Effluent Characteristics (i.e. COD,BOD,TSS,TN, TP) • Energy Data PE was calculated using one of the following methods: 1. Nitrogen – 12 gN/PE d 2. BOD – 60 gBOD/PE d 3. COD – 120 gCOD/PE d Data classified based on scale: • < 2,000 PE • 2,000 to 10,000 PE • 10,000 to 50,000 PE • 50,000 to 100,000 PE • > 100,000 PE Plants grouped under following headings: • Biological Nutrient Removal (BNR) • Membrane Bioreactor (MBR) • Aerated Ponds (AP)	KPI_1 – kWh/m^3 KPI_2 – kWh/PE_{Served} KPI_3 – kWh/kg $COD_{Removed}$ Dilution Factor (DF) – L/PE/d Load Factor (LF) – PE_{Served}/PE_{Design} (%)	Comparisons between kWh/m^3 and kWh/PE assume that the influent quality are broadly similar between plants. This is not always the case and it has been reported that volumetric measures are influenced by the dilution of wastewater. Measures of pollutant removal should be preferred, with N being favoured as COD and BOD can be influenced by combined sewer networks. COD, BOD, TSS, N and PO_4^{3-} removed have all been reported and used. There is a need for benchmarks that account for varying treatment types and WWTP configurations as well as future developments in treatment requirements. Energy consumption (kWh/kg $COD_{Removed}$) decreases as scale increases. Treatment type also had an influence, with CAS and AP having lower energy consumption while MBR systems had the highest. Reporting the energy based on COD removal alone does not reflect the additional energy required to treat for higher nutrient removal. Increased scale and corresponding decreasing energy consumption was

Paper	Notes	PIs considered	Results
	• Biodiscs (BD) • Conventional Activated Sludge (CAS) • Extended Aeration (EA) • Oxidation Ditch (OD) • Sequential Batch Reactor (SBR) • Unspecified Secondary Treatment (UST)		also reflected across the treatment technologies. Some countries show a better energy efficiency (Spain & Germany) when compared to others (France). This could be due to a number of factors including differing influent concentration, energy pricing and effluent regulation. Span & Germany showed lower dilution compared to French. Sewer system design, tertiary treatment and sludge treatment were not investigated due to a lack of data. Increased dilution factor shows increasing energy consumption (kWh/kg $COD_{Removed}$). Also shows an increase in energy efficiency for load factor, with 100% being optimal. Trend increases past 100% but at this point treatment is over capacity and other problems arise. Normalisation techniques are suitable for similar condition comparison, regression based techniques extend the validity and the use of techniques like DEA can allow for the use of multiple ins-and-outs for comparisons.

Paper	Notes	PIs considered	Results
			Most of the benchmarking done have been diagnostic in nature, offering no solutions for improvement. Data collection is a key part of work in coming years.
Comparative analysis of energy intensity and carbon emissions in wastewater treatment in USA, Germany, China and South Africa (Wang et al., 2016)	Looked at WWTPs in four countries: • USA – 15 plants • Germany • China – 5 plants + 2 Industrial WWTPs • South Africa Estimated carbon emissions from energy based on CO_2 emissions per kWh produced. Only appears to have considered GHG potential from energy production.	Unit electricity intensity – kWh/m^3 Green House Gas (GHG) emissions –CO_2 equiv/m^3 Pollutant Removal – kWh/kg $COD_{Removed}$ Nutrient Removal – kWh/kg NH_3 – $N_{Removed}$	Large scale treatment plants in China show a lower EI and GHG emissions. Scale is not the only factor in EI in China, the choice of technology is also a factor. Higher influent pollutant concentrations in China correspond to increase in EI. Lagoons in China had the lowest EI but required largest amount of land. Activated sludge and extended nutrient removal variants have better removal efficiency than other technologies but relatively higher GHG emissions. Volumetric energy measurements are not ideal because they don't account for removal efficiencies. Pollutant removal energy intensity is an option but no

Paper	Notes	PIs considered	Results
			single pollutant can fully account for water quality.
Benchmarking of energy consumption in municipal wastewater treatment plants – a survey of over 200 plants in Italy (Vaccari et al., 2018)	Looked at 267 activated sludge plants in Italy. This is because activated sludge were the most common treatment type. Study grouped plants based on scale: • <2,000 • 2,000 – 10,000 • 10,000 – 100,000 • > 100,000	Energy Consumption Indicator (ECI): • ECI_{m^3} – kWh/$m^3_{Treated}$ • ECI_{COD} – kWh/kg COD_{Rem} • $ECI_{PE(COD\ 120)}$ – kWh/PE_{COD120}/yr. Design Utilisation (%) Linear Regression: • Inlet COD vs. kWh/PE/yr. • L/PE/d vs. kWh/PE/yr.	Evidence of high variability across all plant scales in ECI_{m^3}. Found that there was a correlation between the PE and the COD ECIs, although this is linked to the PE calculation based on COD loading and may not be the case in all situations. More diluted wastewater, with low COD concentrations, have higher specific energy consumption (ECI_{COD}). $ECI_{PE(COD\ 120)}$ median is "substantially comparable" between combined and separate sewer networks, however also found that increased hydraulic flow also increases energy consumption, with a minimum threshold established for increasing flows ($ECI_{PE(COD\ 120)}$). The closer a plant operates to its design capacity the more efficiently it operates.

Paper	Notes	PIs considered	Results
			Correlation between increases in "specific organic load" in the bioreactors ($kgBOD_5/kgTSS/d$) and a decrease in the $ECI_{PE(COD\ 120)}$ ($kWh/PE_{COD120}/yr$). The presence of tertiary treatment did not result in significant increases in the specific energy consumption. This is possibly because the typical physico-chemical treatments used, such as filtration or precipitation, use relatively less energy than other treatment stages.

As can be seen from the data contained in Table 4, many methods have been outlined previously for the benchmarking of wastewater treatment. Many of these proposed methodologies are not necessarily all-encompassing or stand-alone proposals. Rather, they are differing methods of dealing with similar data using different gathering and analysis techniques. Figure 10 shows an outline of the process of performing a benchmark analysis based on examples in the literature. While it has been noted that no consistent or standardised approach to the benchmarking of treatment has been established thus far (Longo et al., 2016; Vaccari et al., 2018), efforts are ongoing.

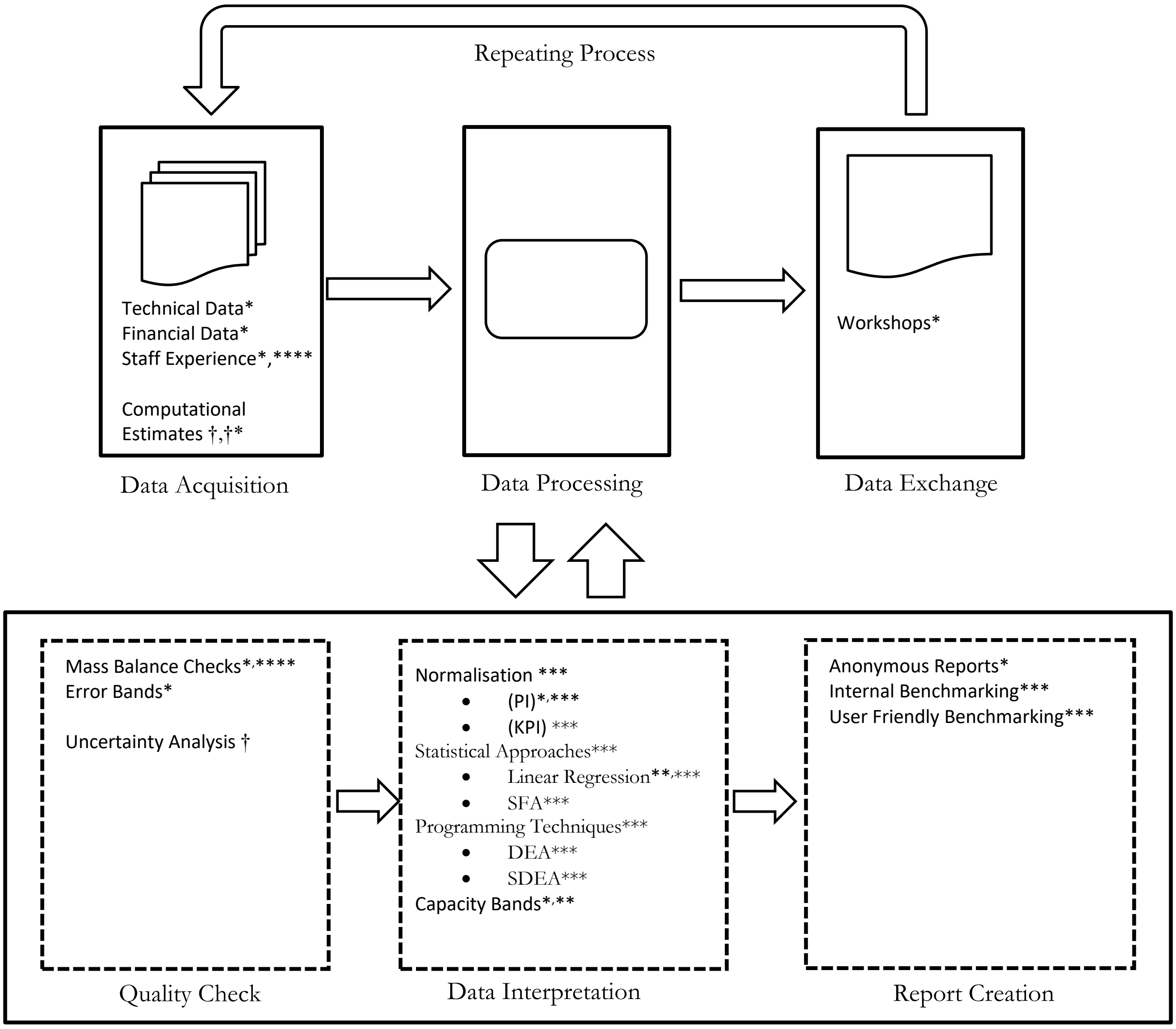


Figure 10: Approaches to Benchmarking

* (Lindtner et al., 2008)
** (Vaccari et al., 2018)
*** (Longo et al., 2016)
****(Lindtner et al., 2004)
†* (Yang et al., 2010)
† (Torregrossa et al., 2016)

Since 2007 the US Environmental Protection Agency's (EPA) Energy Star program has included a standardised method for calculating an Energy Star score for wastewater treatment plants under its commercial buildings and plants program (Energy Star, 2018a). This method uses data from 257 wastewater treatment plants collected by the EPA in the US to perform a comparative analysis and assigns a score from 1 to 100. An online tool, the ENERGY STAR Portfolio Manager, is provided for recording and tracking the calculated scores for commercial buildings (Energy Star, 2018a). One shortcoming of the method is that while the scoring method may be applicable in other regions, the data set the comparative analysis relies upon to assign a score is collected from treatment facilities within the United States. To assign a relevant score in other regions would require, at a minimum, the identification of and the collection of relevant data from suitable peer groups. Energy Star have noted this and offered an outline of the process of identifying, collecting and filtering suitable datasets (Energy Star, 2018b). A second potential shortcoming of the Energy Star system is that the information collected through its Portfolio Manager is not used as an additional source of data to update the calculation of other Energy Star scores (Energy Star, 2018b), a step that could provide a larger data set for comparison if implemented.

In Europe efforts have also been made to produce standardised methodologies for energy benchmarking in wastewater treatment, the most recent being the Enerwater Project (Enerwater, 2019), which has made a methodology proposal in May of 2019 (Longo et al., 2019). This proposal has been submitted to standardisation bodies at a national (Spain) and European level to produce a standardised technical document for the European Union. These efforts at formalising the benchmarking process are an important step, as to be considered truly comparable benchmarking approaches need to be standardised, using common, relevant measurements and using accurate data on resource use and treatment performance (Doherty et al., 2017; O'Doherty et al., 2014; Quadros et al., 2010).

Many of the listed benchmarking approaches don't allow for the hydro-sociological influence of differing regions, which prohibits a true or global comparative analysis of different treatment. Influent characteristics, for example, are important parts of modelling and benchmarking plants and are functions of the catchment area served by the treatment plant. This regionality in treatment is reminiscent of the regionality of the wider hydrologic cycle around the world. A wider array of benchmarking metrics, such as treatment technology; temperature range; and pumping heights in plant, have been suggested to overcome these issues (Borzooei et al., 2019; Quadros et al., 2010; Torregrossa et al., 2016).

The utilisation factor of a treatment plant, which is the utilised capacity of the treatment plant compared with the designed capacity, should also be considered in any analysis or benchmark. It has been shown that the closer a plant is to its design capacity, the more energy efficiently it operates (Krampe, 2013; Vaccari et al., 2018), making this a valuable metric for benchmarking.

To be truly comparable benchmarking approaches need to be standardised, using common, relevant measurements and using accurate data on resource use and treatment performance (Doherty et al., 2017; O'Doherty et al., 2014; Quadros et al., 2010). For benchmarking to be successful, particularly in a field that is slow to change, it will require the involvement of all relevant stakeholders. This includes everyone from the management team within the water utilities, the operational staff and on to the end-user. Their involvement in the benchmarking process serves two primary purposes: the first is to ensure a continuous stream of reliable data necessary for such an ongoing process; the second is to ensure that the necessary steps are taken for improvements if actionable results are produced by the benchmarking process. In return for their participation stakeholders should also be a part of the benchmarking process, having a significant input into the chosen methodology as well as the KPIs that may be chosen. This could be in the form of direct input e.g. involvement in the selection of KPIs but is more likely to be in the form of indirect involvement, such as where operational staff are required to record and distribute operational measurements and data for the purposes of benchmarking.

The choice of whether stakeholder involvement should be direct or indirect is dependent on the stakeholder group being considered: for operational staff to see the benefit of the process they must have a sense of "ownership" over it, which can be achieved by a balance of direct and indirect involvement. For the end-user group however, there should be a preference toward indirect participation, where open dialogue and feedback is sought, but they have little direct input in the methodologies employed e.g. the KPI selection or measurement regimes. The reason for this is simple: if benchmarking efforts are to be useful on a scale greater than that of a regional or national level, then clearly defined methodologies will need to be implemented requiring a level of knowledge and understanding of the processes involved that the public may not have. Benchmarking should not be used as a metric by which to judge operational staff either. Performance benchmarks may be heavily influenced by decisions made prior to the operational staff being involved. Judging their performance based on such metrics could be unreasonable and could result in decreasing participation in the process over time.

Collection Systems

The collection system within UDSs is the part that deals with the collection and transportation of wastewater away from urban areas to the WWTP and receiving body. Collection systems have developed over time to prevent flooding by channelling excess surface water away and to improve public health by removing wastewater from urban areas safely. It can already be seen that flooding has become more frequent and disruptive in urban areas. As mentioned in previous sections there are economic costs associated with urban flooding (García et al., 2015; Salvadore et al., 2015) but there are also significant social and environmental costs. Combined sewer systems (CSSs) are used in many urban areas in the industrialised world (Chocat et al., 2004) and combine storm and sanitary waste into a single pipe. The benefit of such as system is that it can remove domestic, industrial and storm water away from urban areas within a single infrastructure (Chocat et al., 2004). In contrast separated sewer systems (SSSs) keep these storm and sanitary waste separated in independent sub-systems. Therefore, any floods caused by excess flows into a CSS can be contaminated by pollutants that would not be ordinarily present in storm water posing significant health risks.

Long collection system networks are to be avoided due to the disruption and costs associated with maintenance; infiltration of water along the network can dilute wastewater and longer networks can result in longer retention times within the network affecting treatment (Borzooei et al., 2019; Libralato et al., 2012). This may lead to CH_4 production within sewer networks, which impacts on treatment processes within the treatment plant (Flores-Alsina et al., 2011) or can increase EI values due to influent dilution (Vaccari et al., 2018). Such influences are typically neglected when analysing the overall UDS, in part due to the consideration of time and costs when specifying model or analysis boundaries (Borzooei et al., 2019). Future assessments of the performance of UDSs should consider the influence such factors have on the overall efficiency of the systems, to include the energy consumption of the wider collection system.

To reduce the risk to public health wastewater managers use combined sewer overflows (CSOs) to regulate the flow of water through the network (García et al., 2015; Salvadore et al., 2015). CSOs occur where the volume of water in the drainage system exceeds the capacity of the system transporting and treating the wastewater and so operators will release excess waters at specific points in the catchment area. These events increase the pollutant loads in receiving waters and although the type of pollutants released from the sewer networks can depend on the location and density of urbanisation (Salvadore et al., 2015), elevated levels of suspended solids; fecal coliforms; phosphorus; ammonia-nitrogen; and heavy metals are generally found in receiving bodies as a result and can persist for several days after the initial rainfall event (Pleau et al., 2005). With proper integrated management across a UDS pollutant discharges and flooding can be prevented, particularly by managing the hydraulic loads that can overload treatment and cause CSOs. Even today, however, traditional static control systems are often preferred when trying to solve flooding or pollution problems, despite more dynamic solutions such as Real Time Control (RTC) of CSOs being shown to be a cost effective solution (Pleau et al., 2005). As climate change leads to more frequent and intense rain fall events, it is inevitable that inaction in this area will lead to greater difficulties in the future for urban area dwellers, managers and policy makers (García et al., 2015).

Integration, UDSs and RTC

Traditionally the different aspects of water management have been dealt with in isolation to one another. In part due to the successes of improved water supply and public health over many years a reluctance to new approaches in the field has emerged (Bach et al., 2014). This conservatism in water management along with the vast scale and complexity of hydrologic systems has meant that the relationship between the urban environment and its water dynamics is not particularly well understood. As technological developments continue models can be developed that take account of these factors when addressing the needs of these complex systems, and can go some way in alleviating the deficit of knowledge in relation to them (Salvadore et al., 2015). The potential complexity of these systems can be seen in Figure 11, which is a development of the Hydrologic Cycle from Figure 1, and shows some of the complexity when we include the social interactions with the hydrologic cycle.

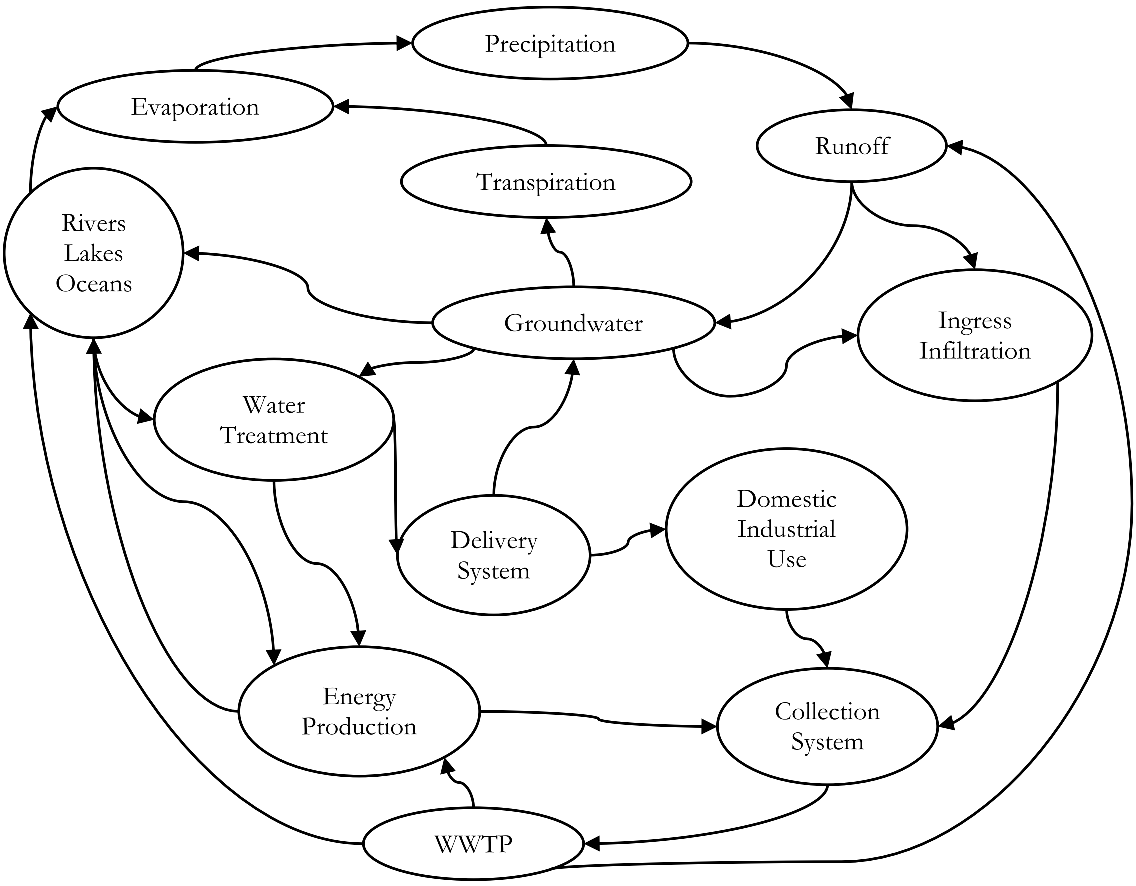


Figure 11: The Hydro-Socio Cycle

The challenge now is to move towards the integrated management of water and wastewater systems and the development of integrated modelling options to inform design and operational decisions. Despite improvements in the field, developing these models is still a challenge (Rauch et al., 2002). Solving the problems facing society such as mitigating the impacts of climate change and more efficient management of resources requires that society accept and adapt to the integrated nature of the problems. Resource conservation objectives; consideration of the systems holistically; consideration of the systems across a range of temporal and spatial scales; and the establishment of links between stakeholders and environmental cycles will be required as part of any multi-disciplinary approaches to water management (Bach et al., 2014). In this context, future urban drainage will require a more integrated approach. This will require an integration of UDSs in terms of physical infrastructures at the planning stages as well as their integrated control during operation. Water treatment; distribution; wastewater collection; storm water drainage; treatment of these flows; and their discharge to the environment will all have to be considered holistically, in a way that considers the interactions and feedbacks that occur between all of them (Bach et al., 2014).

Urban Drainage System Modelling

The search for new and practical solutions to rapid urbanisation; required infrastructural rehabilitation; and the impacts of climate change coupled with improvements in computational sciences have driven the development of new urban water modelling technologies (Bach et al., 2014; Flores-Alsina et al., 2011). Even with these developments there is still more work, particularly in the area of urban drainage modelling. The development of these models is complicated by the lack of understanding around the complex dynamics at play but also by the heterogeneous nature of urban environments and the multitude of hydrologic processes within these environments (Chocat et al., 2004; Salvadore et al., 2015). Of particular interest in any aquatic system model, but particularly in UDS modelling, are the fluid motion processes through the urban environment; the transport processes; and matter conversions because these processes are of key importance in the accurate description of aquatic system behaviour (Rauch et al., 2002). The energy consumption of processes in UDSs have seen little in the way of modelling efforts, as effluent quality has been main focus (Panepinto et al., 2016).

Urban hydrological models are already used and serve a number of purposes: evaluation of the impacts of urbanisation on the hydrologic system; to overcome a lack of suitable data regarding urban environments; and the prediction of future outcomes for planning and management such as flood forecasting and land use (Salvadore et al., 2015). Due to the complexity of the systems, the interactions they are intended to simulate, and the fine temporal and spatial resolution requirements, typical urban hydrological models are complex in nature and are difficult to model holistically. A large amount of data is needed, with a number of parameters over wide temporal and spatial ranges and significant accuracies, although data estimation with suitable uncertainty analysis has been shown to aid daily WWTP benchmarking (Torregrossa et al., 2016). These requirements are driven by the need for these models to account for the interactions that occur across temporal and spatial ranges (García et al., 2015; Salvadore et al., 2015). The reliability of these models and their results as well as their validation and calibration, which is also reliant on the availability of data.

To overcome these difficulties, models used for hydraulic planning are usually simplified, in the sense that they do not model the entire hydrologic cycle, but still require important input parameters and well defined boundary conditions to be considered effective (Salvadore et al., 2015). A number of ways to reduce the complexity of models have been noted: selection of meaningful parameters and processes that represent as closely as possible the criteria required of the model; reducing the number of parameters needed for meaningful results while maintaining strong, relevant and representative parameters for the system; well-defined system boundary conditions; and suitable calibration and validation methods for models (Borzooei et al., 2019; Rauch et al., 2002; Salvadore et al., 2015).

Collection System Modelling

Sewer models have been developing since the 1980s when hydraulic and pollutant loads were calculated using simple models. Pollutant behaviour has been particularly problematic for model development because of the complexity of interactions that can take place within them. Models have gone from considering pollutants simplistically in terms of suspended solid transportation to models considering the sewer as a physical, chemical and biologic reactor, which gives a more realistic accounting of sewer behaviour. Sewer models are still less developed in terms of quality than in other areas, with pollutant concentrations typically being assumed to be constant from event to event and throughout individual events, which neglects certain relevant processes (Rauch et al., 2002). This is despite influent quantity and quality being important parts of the overall modelling of wastewater treatment, particularly if GHG emissions are to be included, and few models have traditionally considered this (Borzooei et al., 2019; Flores-Alsina et al., 2011). There are a number of methods of simulating flow (Salvadore et al., 2015): linear reservoirs; wave approximation; shallow water equations; and Saint-Venant equations. Each method has its benefits and drawbacks and so typically packages modelling collection systems and their catchments will use a variety based on specific requirements.

Several commercially available software packages that model integrated urban catchments that include sewer modelling are available: MUSIC; InfoWorks ICM; MOUSE; CANOE; and MIKE are a few examples. These packages can be prohibitively expensive and as commercial packages their original source code is not easily accessible or available. There are also several freely available software packages that model sewer systems, two of which were investigated further: the US EPA's SWMM and CityDrain II. CityDrain II is an open source urban drainage toolkit available for

MATLAB/Simulink, developed by Hydro-IT (Achleitner et al., 2007; Hydro-IT, 2007). SWMM, like CityDrain, is an urban drainage software package, the source code for which is openly available. Unlike CityDrain however, SWMM can be operated as a standalone package without the need for additional software.

SWMM is a rainfall – runoff model, originally developed in the 1970s as a tool for studying the flow of water through different environments, the analysis of combined sewer overflows (CSOs) and as a catchment drainage tool for flood analysis (Rossman, 2017, 2015). Over time, the software has been used extensively for drainage analysis in urban areas, with functionality for modelling sewer systems under flood and surcharge conditions (Burger et al., 2014; Gironás et al., 2010; Rossman, 2004; Singh and Frevert, 2005).

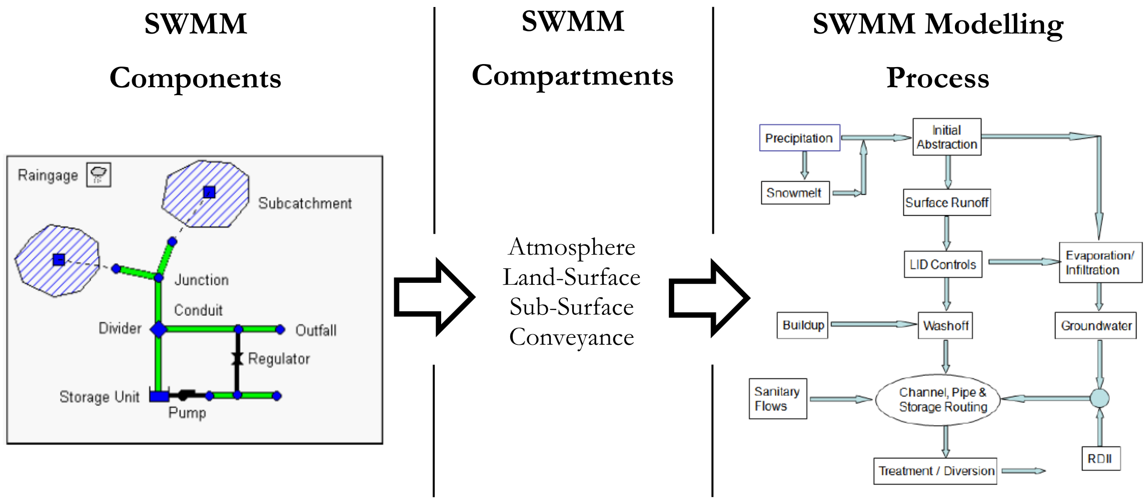


Figure 12: Work flow for SWMM (Adapted from Rossman (Rossman, 2017))

Figure 12 shows the basic workflow for SWMM. The user inputs and designates various components from the catchment area being considered within the user interface and then creates links between them e.g. the sub-catchment component represents an area of land which is then connected to a junction component into which runoff flows. When the user is finished inputting the required data, SWMM then divides the different components into compartments used for the subsequent calculations. The four compartment are (Rossman, 2017):

- Atmosphere Compartment – This compartment deals with precipitation
- Land-Surface Compartment – Represents the catchments being studied. Inflow comes as precipitation while outflow can be evaporation, surface runoff to the conveyance compartment via a junction component or as infiltration to the sub-surface compartment.

- Sub-Surface Compartment – This compartment assesses infiltration from the surface flows.
- Conveyance Compartment – This is the compartment containing network elements such as nodes and conduits that represent sewer network components.

Once all the required data has been input into SWMM, the calculations are performed by the software following the modelling process flow shown in Figure 12.

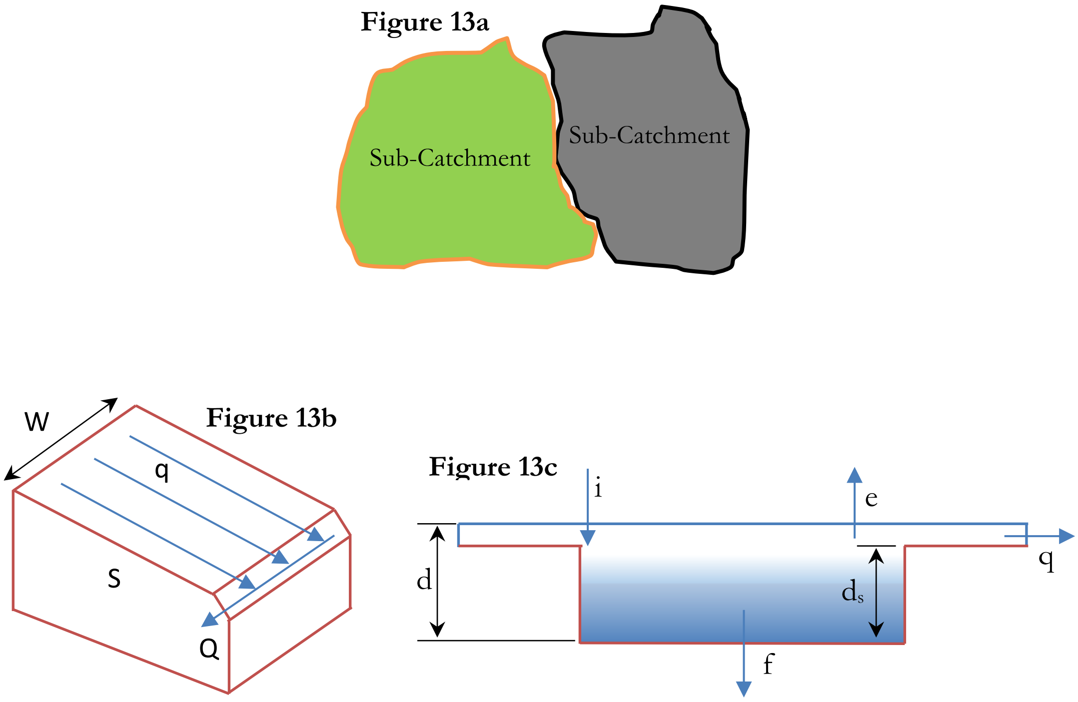


Figure 13: SWMM Model

Where:
W is the width of the sub-catchment (m)
q is the run-off rate (m/s)
S is the average slope of the area
Q is the Volumetric flow rate (m^3/s)
d is the depth of water (m)
d_s is the storage depth of water in the catchment area (m)
i is precipitation rate (m/s)
e is evaporation rate (m/s)
f is the infiltration rate (m/s)

The first step in this process is to conceptualise the irregular and heterogeneous sub-catchments, such as those shown in Figure 13a, as uniform, rectangular areas shown in Figure 13b. Once this is done the volume of rainfall that falls on the sub-catchment, as well as the subsequent overland flows, can be calculated much more easily. Methods such as this, where runoff is calculated based on the surface area and precipitation data, date back to the 1800s (Chocat et al., 2004). When using these methods a reduction factor to account for losses due to ponding, evaporation and infiltration is often applied and the outline of how SWMM takes this into account is shown in Figure 13c.

Evaporation rates (e) are given to the system in millimetres per second (mm/s), as are the infiltration rates (f) for the catchment area. The depth of storage (d_s) for the catchment area is also a known or estimated parameter. As the rain falls, surface runoff does not begin until such time as the depth of storage in the catchment area has been exceeded. The volume of runoff water is then equal to the width of the conceptual channel (W), shown in Figure 13b, multiplied by the length of the slope (S) and the total depth of water (d) minus the storage depth (d_s). The catchment width parameter (W) can be estimated by dividing the catchment area (m^2) by the maximum length of the flow within the catchment (m) i.e. the length of the longest line from the outlet junction of the catchment to the boundary of the catchment (Rossman, 2015). The length of the slope (S) can then be calculated by dividing the area of the catchment by the catchment width (W).

The change in the depth of water (d) with respect to time (t) in the catchment is calculated using the partial derivative shown as equation (7).

$$\frac{\partial d}{\partial t} = i - e - f - q \tag{7}$$

Where:
i is precipitation rate (m/s)
e is evaporation rate (m/s)
f is the infiltration rate (m/s)
q is the run-off rate (m/s)

The precipitation (i), evaporation (e) and infiltration (f) rates are typically known values based on the weather and catchment area information. The run-off rate (q) from a sub-catchment is calculated for each time step as an overland flow into an assigned junction by the SWMM software using a using a variation of Manning's equation, shown as equation (8). To do this it is assumed that the sub-catchment overland flow behaves as a uniform flow in a rectangular channel of width (W), a depth of $d - d_s$ and a slope (S) as per Figure 13.

$$Q = \frac{k}{n} S^{\frac{1}{2}} R_x^{\frac{2}{3}} A_x \tag{8}$$

Where:
Q is the Volumetric flow rate (m^3/s)
k is a unit conversion factor (1 for SI Units, 1.49 for Imperial Units)
n is a surface roughness coefficient ($s/m^{1/3}$)
S is the average slope of the area
R_x is the hydraulic radius of the area (m)
A_x is the cross-sectional area through which runoff flows (m^2)

Note: n is often listed without units or unit less, this is not the case.

In calculating the overland flow SWMM assumes that :

$$|W| \gg |d|$$

Therefore:

$$A_x = W(d - d_s)$$

$$R_x = d - d_s$$

So equation (8) becomes:

$$Q = \frac{k}{n} W S^{\frac{1}{2}} (d - d_s)^{\frac{5}{3}} \tag{9}$$

Where:
W is the width of the sub-catchment (m)
d is the depth of water (m)
d_s is the storage depth of water in the catchment area (m)

Finally, to solve for the run-off rate (q) equation (9) becomes:

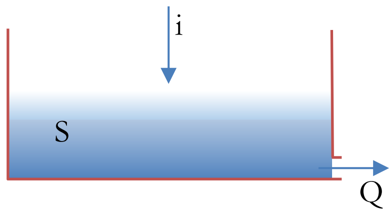
$$q = \frac{kW S^{\frac{1}{2}}}{An} (d - d_s)^{\frac{5}{3}} \tag{10}$$

Where:
q is the surface run-off rate per unit time (m/t)
A is the surface area of the catchment area

The surface run-off can then be passed across the interface between the conveyance and land-surface compartments through an assigned node or junction component. It is then used to calculate the flow through the sewer system within this conveyance compartment using one dimensional flow analysis, calculating the flows through conduits and the water depths at nodes.

Linear and Non-Linear Reservoir Models

As stated previously many methods for calculating the runoff from an area due to rainfall exist. The simplest of these is the linear reservoir model, shown here in Figure 14 (Pedersen et al., 1980). In the case of a linear reservoir as shown, the flowrate (Q) can be calculated using equation (11).



Where:
Q is the discharge flow rate (m^3/s)
S is the volume of stored water (m^3)
i is the rainfall excess (m^3)

Figure 14: Linear Reservoir Model

$$Q = KS \tag{11}$$

Where:
Q is the discharge flow rate (m^3/s)
K is the constant response factor (1/s)
S is the volume of stored water (m^3)

The constant response factor (K) in equation (11) is a function of the reservoir being considered. The value of the stored volume of water (S) can be calculated by multiplying the area of the catchment by the rainfall excess (i).

$$i = Q + \frac{dS}{dt} \tag{12}$$

Where:
i is the rainfall excess (m^3)
Q is the discharge rate (m^3/s)
S is the volume of stored water (m^3)

By combining equation (11) & (12) and rearranging the linear reservoir equation can be derived, shown as equation (13).

$$Q_2 = Q_1 e^{-K(t_2-t_1)} + i\left(1 - e^{-K(t_2-t_1)}\right) \tag{13}$$

Where:
Q_1 is the discharge rate at time t_1 (m^3/s)
Q_2 is the discharge rate at time t_2 (m^3/s)
K is the drainage area response factor (1/s)
i is the rainfall excess (m^3)

It can be seen that equation (13) is the sum of two terms: the first term deals with the flows based on the continuous drainage of the stored water in the reservoir while the second deals with the recharging of the water in the reservoir due to rainfall. When there is no rainfall occurring (i = 0) the second term goes to zero, thus the first term in equation (13) can be re-arranged and used to solve for K, shown in equation (14).

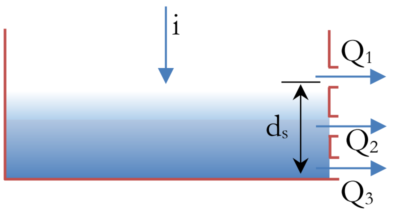
$$Q_2 = Q_1 e^{-K(t_2 - t_1)} \tag{14}$$

If $\Delta t = 1$ (i.e. $t_2 - t_1 = 1$) then equation (14) can be re-arranged as:

$$K = -\ln\left(\frac{Q_2}{Q_1}\right) \tag{15}$$

Where:
Q_1 is the discharge rate at time t_1 (m^3/s)
Q_2 is the discharge rate at time t_2 (m^3/s)
K is the drainage area response factor (s^{-1})

The non-linear reservoir model, shown as Figure 15, is a furtherance of the linear method that assumes the response factor (K) is no longer constant as the number of flow paths (Q_n) increases. This is in fact a truer representation of the flow paths from a catchment area as flow out of a catchment can travel overland, through sewer networks and into the ground as infiltration.



Where:
Q_n is the discharge rate from the nth outlet (m^3/s)
i is the rainfall excess (m^3)
d_s is the depth of water stored in the reservoir (m)

Figure 15: Non Linear Reservoir

WWTP and Discharge Modelling

WWTP modelling is an area that has seen much work over recent decades and differs from sewer network modelling or river modelling in two ways: the hydraulic models can be crude approximations, with little variation to flow and outflows equal to inflows, and the models are built around individual treatment processes or technologies. For example, a standard suite of Activated Sludge Models (ASMs) have been developed and used since the 1980s to model the biological processes in activated sludge treatment(Gujer et al., 1999; Henze et al., 1987; Koch et al., 2000). The Benchmark Simulation Model 1 (BSM1) was an attempt to standardise the modelling of wastewater treatment against a standard plant for the evaluation of control strategies and still uses the ASM models to model the biological processes in treatment (Alex et al., 2008; Maere et al., 2011). These models are often limited however by the data required to calibrate parameters to resemble plant behaviour and by the difficulty in using them to model plants where a significant portion of influent is comprised of industrial waste (Rauch et al., 2002).

Discharge from treatment forms part of wider river quality modelling. This has been considered and the IWA formed a task group to look at creating a technical basis from which to formulate standard, consistent river water quality models. The difficulty in modelling river quality is exacerbated by the temporal and spatial scales across which it needs to be considered and the difficulties that arise at the interfaces between different sub-systems and systems involved. This is an example of the type of problem experienced more generally in the integration of models and to overcome such issues interfaces need to be identified and a set of consistent model parameters in sub-system models will need to be developed (Rauch et al., 2002). These models can be applied to estuaries, coastal waters or lakes, once the differing spatial dimensions and their effects on hydrodynamics are taken into account (Rauch et al., 2002).

Integrated UDS Modelling

Integrated UDS (iUDS) modelling is a field of research that seeks to integrate models of the various sub-systems, e.g. collection system, treatment plant and receiving waters, that have in the past been treated in isolation. Such an approach is in alignment with the integrated approach to hydro-sociology that has been mentioned in previous sections and will be a requirement of future developments in wastewater treatment (Gude, 2015). In this context it is perhaps one of the most important areas of research, as the environmental burden and resource intensity of collecting, storing, treating and discharge of urban wastewater is one of the crucial interfaces in the hydro-sociological relationship. In particular this integration work is important in wastewater management, where reducing the overall impact of treatment is the goal (Lorenzo-Toja et al., 2016). Uptake of these integrated modelling approaches has been slow to date, possibly due to the fragmented approaches that have been taken in managing the relevant infrastructures; differing views, backgrounds, terminology and methods of those involved in relevant fields; as well as the previously mentioned complexities within systems (Bach et al., 2014; Rauch et al., 2002). This is despite the potential financial returns of implementing such integrated modelling approaches and, as has been mentioned previously, iUDS modelling may provide for improved planning of collection systems and intelligent network management options in a more cost-effective way than traditional methods. For example, the implementation of such modelling would mean that more costly network capacity options could be avoided in favour of optimised control of existing drainage systems (García et al., 2015).

Integrated hydrological model development has to date focused on specialised tools for specific areas of the hydrological processes or on the coupling of a few semi-specialised models to give an overview of the overall urban hydrological environment. Such an approach may at times be flawed, for example where the receiving water is being modelled it is often done to represent surface waters such as rivers and coastal waters, while ground water modelling suffers as a result (Rauch et al., 2002; Salvadore et al., 2015). It is also important to note that for many reasons there is no agreement on a universal concept or methodology for simulation of the urban hydrological environment at a catchment scale (Salvadore et al., 2015). One of the potential improvements allowed for by integrated modelling of urban drainage systems will be to allow managers to model the impact of discharges on receiving waters more holistically. Discharges to receiving water can be categorised based on the type of impact: bio-chemical, physical, hygienic, aesthetic, hydraulic, etc. and in terms of the temporal impact of discharges: acute, delayed, and accumulative. Links have been made between the costs associated with the impacts of these temporal delays in wastewater management and the benchmarking of facilities (Torregrossa et al., 2016).

The emergence of integrated real time control (iRTC) systems may also be another driver in the development of integrated models. Such models will have specific requirements in terms of computational time and efficiency as they simultaneously model the complete system in a minimal amount of time (Rauch et al., 2002). Meeting the requirements of such technology may mean abandoning, instead of adapting, the modelling approaches taken in the past, and the piecemeal approach integrating of one or two specialised models to achieve the level of integration required (Bach et al., 2014). Problems in integrating models typically occur at the interfaces between the various sub-systems and models, due to inconsistencies in measurements or parameters between the sub-models across a range of scales (Rauch et al., 2002). Overcoming this problem will require a level of understanding of multiple components within a model, over a range of fields and this will require a multi-disciplinary approach (Bach et al., 2014; García et al., 2015). Developments in computing have reduced the computational burden of these difficult problems and the detailed and consistent measurements that can be provided with readily available remote sensing has meant distributed hydrological models are becoming more commonplace (Bach et al., 2014; García et al., 2015). The goal of ever more integrated modelling of UDSs is becoming more of a technical reality but will require a move away from traditional norms. This will require a change in institutional attitude towards data sharing and move away from traditional single discipline education and research.

Real-Time Control and Control Systems

In the 100 years since the development of the activated sludge process many of the developments in wastewater treatment have been in optimising the performance of processes for a range of conditions. This is a difficult task considering the complexity but seen as necessary as a result of rising costs (Borzooei et al., 2019; Chocat et al., 2004). One such development has been the integrated, real time control of urban drainage systems where collection systems and WWTPs are managed and optimised simultaneously. Such a system not only includes the interactions and feedback between all of its parts but also considers the system with respect to the rest of the hydrologic cycle as well as (Bach et al., 2014; Breinholt et al., 2008). Traditionally two types of control strategies have been available to wastewater managers: static or off-line control, which are difficult to adapt to changing conditions; and real-time control (RTC) or on-line strategies that monitor the system through real time sensor data acquisition and actively interacts with it, making modifications to the processes that can optimise system operation by means of a control strategy. This is achieved by comparing the present state of the UDS to established rules based on pre-defined control objectives (Campisano et al., 2013; Erbe et al., 2007; García et al., 2015; Schütze et al., 2008).

As the complexity and dynamic nature of UDSs has been recognised, it has been suggested that RTC is the more suitable control strategy for the management of UDSs. While static controls have been widely implemented, they cannot typically achieve the same level of optimisation as RTCs. RTC is usually one of a suite of measures considered to improve water resource management, along with sewer infrastructural improvements; catchment disconnection; and source control measures (Erbe et al., 2007; Schütze et al., 2008). RTC allows more flexible options for dealing with disturbances to the system, and such strategies can introduce time as a factor in making key decisions (Schütze et al., 2008). Two basic categories for RTC strategies have emerged over time: volume based and pollution based. Volume based strategies are typically employed, where control decisions are made based on volumetric measurements (García et al., 2015). Having been at the concept stage for some time recent developments have seen pollution based RTC strategies emerge, where the control decisions are made based on pollutant levels in wastewater (Campisano et al., 2013). The use of RTC strategies have shown it is possible to improve UDS performance; improve the environment by improving CSO performance; reduce greenhouse gas emissions; and minimise the capital and operational expenditure for UDSs, particularly when compared to expensive infrastructural upgrades (Breinholt et al., 2008; Campisano et al., 2013; Erbe et al., 2007; Flores-Alsina et al., 2011; García et al., 2015; Nielsen et al., 1996; Pleau et al., 2005; Schütze et al., 2008).

There are difficulties with implementing the integrated real time control (iRTC) of UDSs which has resulted in slow uptake even with the proof of more efficient or optimal results: institutional wariness of making changes to proven systems; the approach also requires management practices that take a complicated and holistic approach to managing the hydrologic needs of the area, which can be difficult to implement. Concerns raised about the ability of such a system to react to emergencies; limitations in computing or a lack of suitable models and the reliability of sensor, control and actuator equipment has, in the past, led to difficulties and negative opinions in the industry have been formed about the approach. Where equipment fails, or is otherwise allowed fall into disuse, it can further reinforce negative opinions about the overall system implementation rather than its operation (Breinholt et al., 2008; Campisano et al., 2013; Erbe et al., 2007; García et al., 2015; Pleau et al., 2005; Schütze et al., 2008). Many of these technological issues have been overcome more recently however, many having been addressed with technology reaching mature stage in many instances and so the integration and RTC of systems has become an ever more viable option for managers and operators in the right context. It may not always be the case that RTC strategies are necessary or cost effective and they tend to suit larger urban areas more so than smaller networks and treatments systems (Pleau et al., 2005). To determine their viability, several areas for consideration have been identified to investigate the benefits and potential objectives of RTC for a system before investment is necessary (García et al., 2015):

- Hydraulics
- Instrumentation
- Remote Monitoring
- Process Control
- Software Development
- Mathematical modelling
- Organisational issues
- Forecasting Rainfall/flows

There also other considerations, such as the level at which control of the system will be maintained. The options for control are broken down into two scales: local level; and global level. These levels of control can be broadly distinguished relative to the location of actuation and control equipment: where control of the system is retained on site the system can be said to be locally controlled, as opposed to instances where the actuators are controlled from a location off site, in a centrally located control room being the most common example of global level control systems. It is usually the case that more localised systems are used where the numbers of actuators controlled is quite small, whereas global controls are more appropriate with larger numbers of actuators. The decision making process should also take these two levels into consideration: globally controlled systems have more monitored parameters, allowing strategies that take a more holistic view of the system when making decisions, although this benefit is somewhat offset by the need for more intensive communications infrastructure in global systems (García et al., 2015).

While the technical feasibility may no longer be in doubt there are few successful examples of implementation and there are some areas that still need attention. The quality of sensor and measurement equipment; the resilience of such equipment in dealing with harsh environments in wastewater systems, fault detection, fault prevention and redundancy methods should be considered. The terms of maintenance and operation, the RTC of solids settling in the sewer network (Campisano et al., 2013) and appropriate staff training are also areas that remain to be addressed. Improved water management requirements, climate change and the mounting costs of infrastructural solutions are however driving iRTC as a possible solution to network capacity constraints, but are not necessarily a replacement for traditional infrastructural projects and should be used to supplement traditional measures also (Breinholt et al., 2008; Erbe et al., 2007; García et al., 2015). Despite these remaining limitations, a new push should be made for iRTC solutions with multiple objectives, that have been outlined throughout the literature (Breinholt et al., 2008; Campisano et al., 2013; Chocat et al., 2004; Erbe et al., 2007; Gude, 2015; Pleau et al., 2005; Schütze et al., 2008; Wang et al., 2016):

- Maintain Public Hygiene
- Avoid flooding and surcharges in urban area
- Avoid sediment build up in the sewer network
- Maximise storage capacity before overflows occur
- Store the most polluted water (first flush) before overflows occur
- Reduce pollution in receiving waters
- Direct overflows to least sensitive receiving waters
- Maximise the treatment of wastewater during rainfall events
- Ensure treatment of wastewater to regulatory standards
- Improve or enhance the processes of the WWTP
- Reduced operational cost in terms of energy and chemical use in treatment
- Maximise the recovery of resources during treatment

Many of these objectives can only be achieved through inter-disciplinary work that crosses traditional boundaries (Breinholt et al., 2008; García et al., 2015; Gude, 2015; Liu et al., 2016; Pleau et al., 2005; Salvadore et al., 2015). The benchmarking of UDSs and WWTPs will also benefit from these integrations, as the real-time collection and assessment of data would be beneficial to performance assessment (Doherty et al., 2017). The application of any RTC strategy relies heavily on the availability of suitable data which, as has been previously mentioned, can be a difficulty in wastewater engineering. In the absence of suitable data sets from which a strategy can be derived, models would provide a suitable proxy when correctly implemented. Minimisation strategies that reduce the number of parameters necessary for iRTC systems along with data quality checks and fault detection will also be critical (Campisano et al., 2013). The future development of RTC systems may also benefit from the inclusion of forecasting solutions, such as forecasting the amount of rain that may fall into an area, and thereby becoming a load on UDSs. These forecasts will require cross disciplinary work and may not be useful over longer prediction horizons and so a trade-off should be considered regarding computational requirements, complexity and usefulness of any such models (García et al., 2015). The implementation of such an iRTC strategy would, for example, take data over time of WWTP influent loadings during a variety of situations. This could then be combined with data taken from upstream in the collection system to allow for a correlation study of the two. This system would in theory then be able to estimate the concentration levels in collection networks based on the time between wet weather events from measured flow data throughout the network. Control parameters could be established that retain concentrated wastewaters and prioritise the discharge through CSOs based on receiving water sensitivity (Nielsen et al., 1996) and when coupled with weather forecasting could allow for WWTP optimisation over an extended time horizon.

Summary of Literature Review

The hydrologic cycle, while simple in concept, is made more complicated when the various interactions within it are considered. Society is in a critical role because of its ability to manipulate this cycle and consequently should take account of and minimise its impact on the cycle. One size fits all solutions may not always be suited to water management due to the regional differences in water availability, or use, and so solutions should be sought that can provide a range of suitable options. Wastewater treatment represents a significant modification of the hydrologic cycle, both in terms of quality and quantity of water, and should be considered holistically and in the context of the wider socio-hydrologic cycle. Such complex systems and interactions can be overcome by interdisciplinary work that crosses traditional boundaries. The integration of wastewater treatment across several traditionally segregated topics is an area of ongoing research and this will require an interdisciplinary approach in several areas. Where water management has typically been dealt with as isolated sub-systems, this will need to be overcome to facilitate integration and meet new challenges. UDSs are an interface of importance in the field of hydro-sociology as they represent a threat to both public health and the environment. As such their efficient operation is crucial to ensuring a sustainable urban area and their consideration within the energy-water nexus is vital.

Broadly speaking wastewater treatment energy use is dependent on the quantity and quality of influent to treatment and regulations regarding effluent quality and is expected to rise in coming years. Energy audits and assessments have already been performed and consistently report highly variable energy intensities based on several factors. Specific comparisons are not yet possible due to the nature of differing catchment areas, regulations and approaches to wastewater treatment across different boundaries. A regional assessment of treatment should therefore be carried out before any inference is made regarding the energy intensity of treatment.

The greatest source of variation to influent is due to wet weather flows and consequently it is expected that these flows will have an impact on the energy intensity of treatment that has yet to be investigated fully. If there is a decrease in removal efficiency due to these flows and a parallel increase in energy consumed this is expected to impact any energy benchmarking efforts. Typical system boundaries have often neglected the energy consumed in the collection system of a given UDS. Where data is lacking to conduct such an analysis, it may be overcome by the responsible use of simplified modelling options, conducted with an understanding of the limitations of the results of such models. Calibration and validation of these models will also require significant amounts of data and so models should be derived that minimise the numbers of parameters to be included while utilising what data is readily available.

Energy benchmarking of wastewater treatment has been a relatively recent development in the field and studies to date have considered the energy intensity of wastewater treatment in regional or fragmented ways. Benchmarking and KPI selection tools have been developed, however these typically rely upon regionally specific datasets or provide a large selection of KPIs. Performance benchmarks should be simple and uniformly specified to allow ease of use and for the comparison of the greatest number of plants. Despite this no consistent methodology for assessing or comparing the energy consumption of wastewater treatment across national or transnational boundaries has been established. KPI selection should also be carefully considered as in some circumstances inappropriate KPIs have been used in the assessment of treatment. For example, volumetric flows to treatment have typically been used in assessments, despite evidence that this may give misleading results and the use of PE served or COD reduction being considered better measures of energy intensity.

Further Research

Following the review of literature, several areas for further research were identified:

- Securing safe and secure water, food and energy resources for all societies should be a priority for future global development and should be crucial to any investigation of the energy-water nexus. Research should also consider “nth order” impacts and those changes already apparent in the hydrologic cycle as a result of human intervention.
- Questions remain with regard to the influences of wet weather flows and influent variability on the energy intensity of treatment. In addition to the influence of these

flows on the treatment processes, the area of energy intensity in the collection system is one that has yet to be adequately addressed in the literature. This may be an important factor in the benchmarking of treatment, as the amount of pumping required in the drainage area is site specific but should be included in the overall assessment of energy intensity of treatment.

- While performance benchmarking studies have been conducted, these are reliant upon relatively large numbers of KPI sets. There may be difficulties associated with implementing such studies where there is a lack of the available data related to these KPI sets. A global performance benchmarking methodology based on a small number of KPIs would have several benefits: it would simplify the amounts of data required to conducts larger studies, allow for a simple method of quickly comparing plants at a global level and would improve potential stakeholder participation and interaction by making communication simpler.
- Can an easily integrated model be derived that can calculate the energy requirements of all infrastructures within the system and provide accurate values for influent concentrations based on rainfall measurements, while using a minimal number of parameters? Such a model would be invaluable in assessing the energy consumption of an UDS and would be useful for selecting iRTC strategies. If such a model can be derived it could also be developed into a predictive tool, optimising the operation of the system based on precipitation data over relatively short time horizons, on the scale of hours or days.

Based on the results of this literature review, four research objectives were identified in relation to the energy-water nexus, and the influence of rainfall in the context of Irish wastewater treatment. These objectives are:

1. As flows to treatment are typically non-normally distributed the implication of using the arithmetic mean and the median of these datasets for reporting is investigated.
2. The second objective is to investigate the contribution of the drainage area pumping stations on the electricity consumed for two drainage areas in Northern Ireland.
3. Does the median of flow datasets represent a suitable measure of the base flows to treatment annually or is an appropriate categorisation process required?
4. To avoid model complexity models are often simplified to the minimum required input parameters. In this context, the applicability of a relatively simple model such

as the non-linear reservoir model to calculate increased flows due to rainfall in the catchment area is investigated.

Materials and Methods

Initial Outline

At an early stage of research the ALICE project offered the opportunity to work with industrial and academic partners in the wastewater treatment field. ALICE stands for accelerate innovation in urban wastewater management for addressing the effects of climate change. It is a H2020 RISE project designed to facilitate the exchange of staff between the wastewater industry and academia across the EU with the aim of overcoming barriers and expanding knowledge in the face of climate change. This program facilitated the secondment of early stage researchers to Northern Ireland Water (NI Water) as well as access to data, site visits and consultations with NI Water staff. This was invaluable experience, answering some of the initial research questions and informing the research that followed. It was also hoped that the apparent lack of available energy data under a variety of operating conditions in the literature could also be addressed during these secondments.

Initially the process information, hydraulic flow data and electricity consumption data at a plant level were made available for two treatment plants in Northern Ireland, referred to as Plant A and B. The selection of these plants was made after a consultation with the staff in NI Water and based primarily on the similarity of design and capacity of the two plants. Some of the research that followed formed the basis of a conference paper titled "Impact of Rainfall Events on the Electricity Consumption of Two Wastewater Treatment Plants" that was presented by the author (Daly et al., 2018) at the 13th Conference on Sustainable Development of Energy, Water and Environment Systems at Palermo in Italy.

Additional data regarding the electricity consumption and hydraulic flows through pumping stations in the drainage area of one WWTP, Plant B, were also provided. The selection of a single plant and drainage area was based on the availability of data and this information forms the basis of this thesis. The flows through pumping stations are not typically measured at all pumping stations in the drainage area. Plant B was relatively unique in this regard as flows were measured at all three terminal pumping stations (TPSs), the last pumping stations before the treatment plant headworks, as well as at the headworks of the plant itself.

Data Collection

NI Water Data

Treatment Plant Data

An outline of the processes involved at each plant is shown in Figure 16, and the discharge criteria in Table 5. The design capacities of the plants differ: Plant A has a design capacity of over 85,000 PE while Plant B has a design capacity of 100,000 PE. Plant A has an inlet screw pump at the head works followed by preliminary treatment with screening and grit removal as well as storm separation and storage. Primary treatment follows, with secondary treatment using an activated sludge process and settling with chemical precipitation. Plant B is similar, an obvious difference however being the absence of a screw pump at the inlet. Preliminary treatment features the screening, grit removal and storm storage with additional emphasis on the removal of fats, oils and greases (FOG) owing to a significant hospitality sector presence in the drainage area.

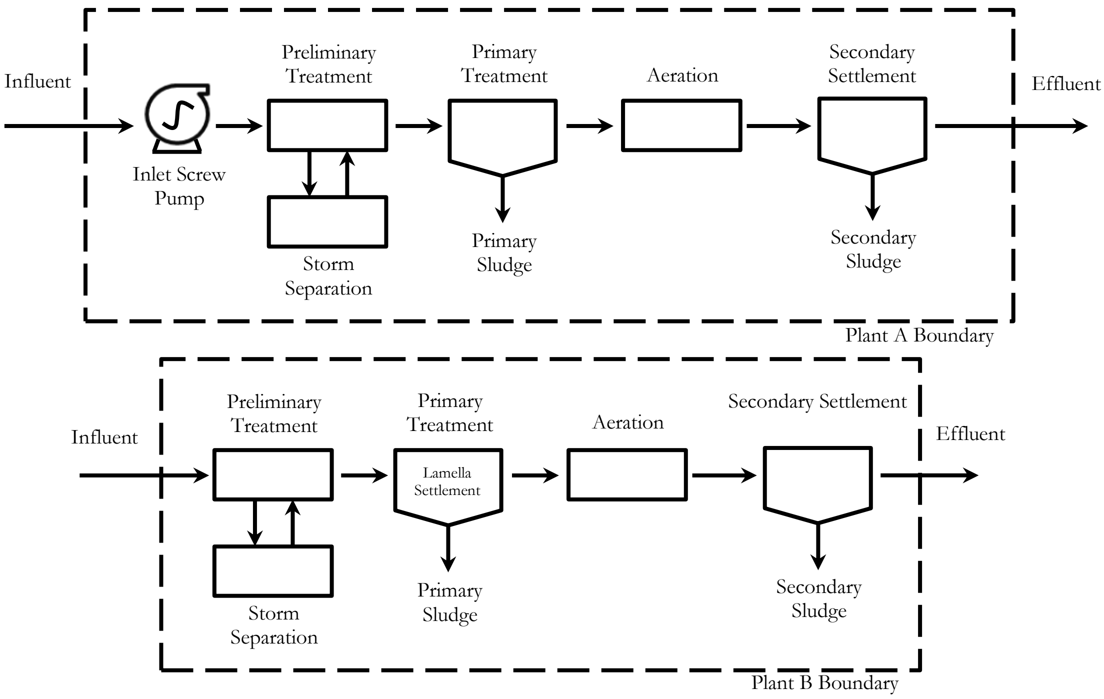


Figure 16: Process Outline for Plants A and B (Daly et al., 2018)

Table 5: Discharge Criteria for Plants A and B (Daly et al., 2018)

	Capacity	BOD	Suspended Solids	Ammonia	Total Nitrogen	Total Phosphorus
Plant A	~85,000 PE	10 mg/l	20 mg/l	15 mg/l	15 mg/l	15 mg/l
Plant B	~100,000 PE	30 mg/l	50 mg/l	-	-	-

Plant B Drainage Area Data

In NI Water the area of land served by the drainage system is subdivided into areas known as the sub-drainage area catchments (SDACs). Plant B was selected for the detailed analysis presented in this thesis, based on the availability of data for this catchment area, and services three such SDACs, with several wastewater pumping stations (WWPSs) dispersed throughout the drainage area. Each SDAC has one final terminal pumping station (TPS) through which the flows are pumped from the SDACs to the headworks of the wastewater treatment facility. Figure 17 shows an example of how the drainage area at Plant B is arranged, with three SDACs draining into three TPSs and on to the WWTP headworks.

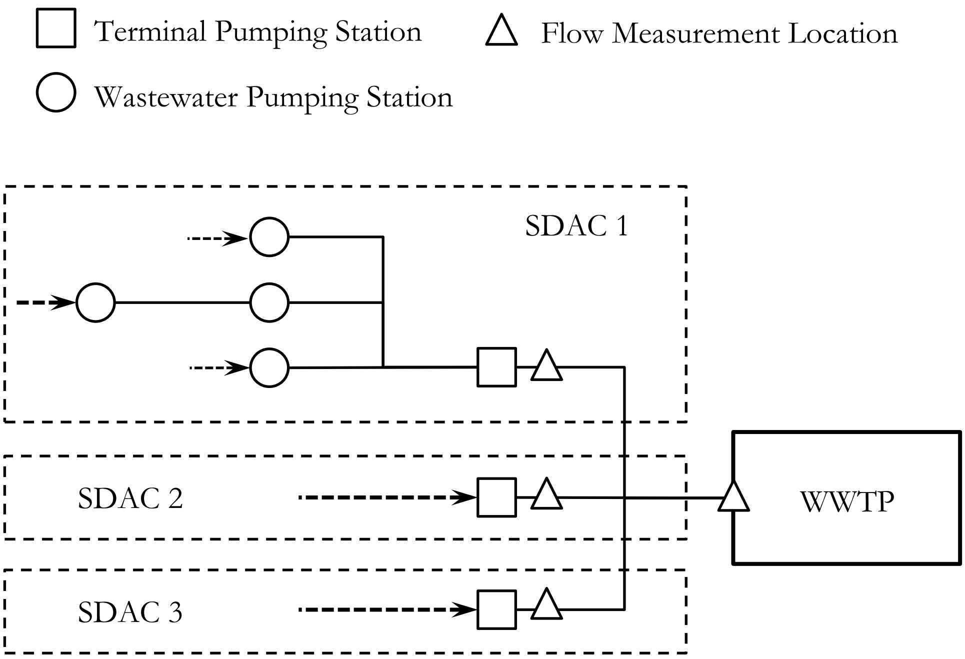


Figure 17: Example of NI Water Sub-Drainage Area Catchments

Flow Data

NI Water made available the daily flows to treatment for both Plants A & B for the years 2016 and 2017. These flows are measured in cubic metres per day (m^3/d) at the headworks of each plant and represent the full flow to treatment (FFT) for both WWTPs. A histogram summary of these flows for both plants can be seen in Figure 18, while cumulative frequency distribution plots for the same flow data are shown in Figure 19. The plots shown in Figure 19 indicate there are several flow outliers at both plants: both plants show maximum flows approximately four times the minimum. This follows what can be found in published literature regarding the variation of flows to treatment (Bertrand-Krajewski et al., 1995).

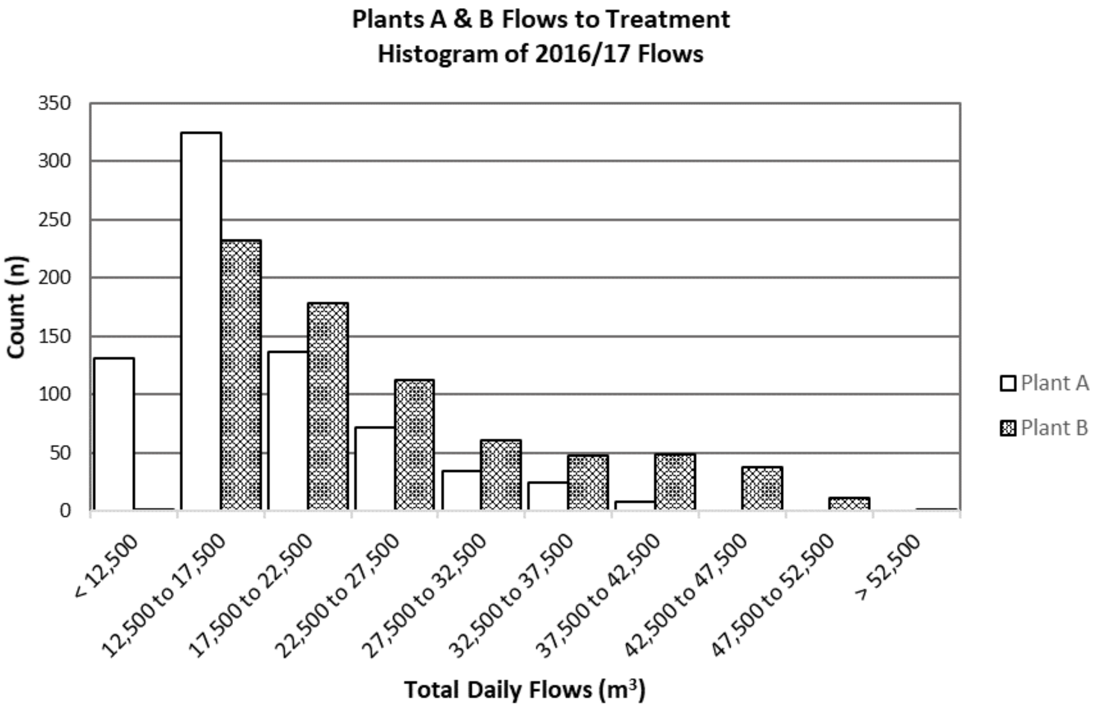


Figure 18: Histogram of FFTs for 2016/17

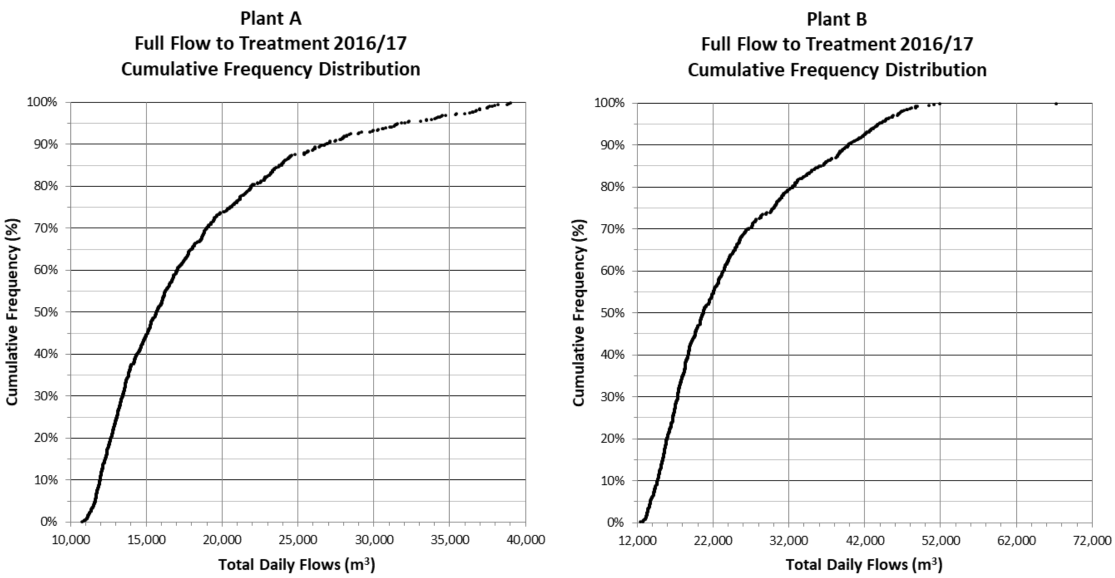


Figure 19: Plant A & B FFTs for 2016/17 Cumulative Frequency Distribution

In addition to the FFTs for 2016/17, NI Water staff also made available the 2016 flows through the three terminal pumping stations, referred to TPS 1, 2 & 3, throughout the drainage area of Plant B. The location of where these measurements are taken is shown in Figure 17, while a histogram summary of the TPS flows are shown in Figure 20.

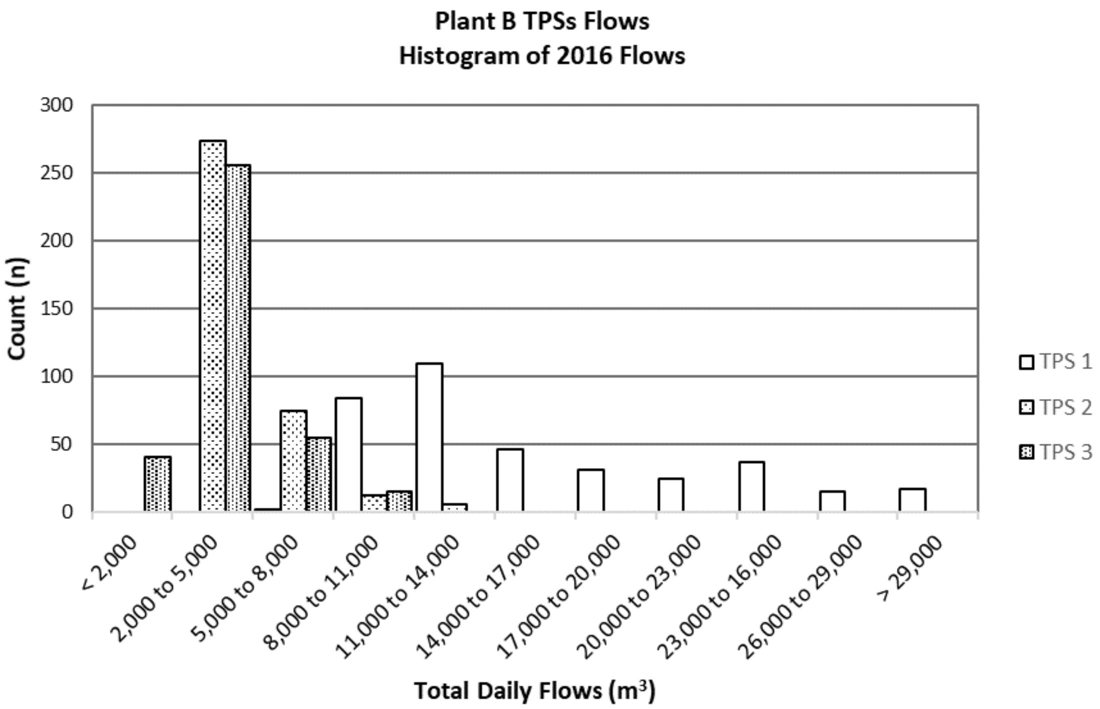


Figure 20: Histogram of Plant B TPSs 2016 Flows

Electricity Data

NI Water also provided the electricity consumption data for Plants A & B for the year 2016, and the cumulative distribution of this data is shown in Figure 21. This data is in kWh and was collected from automated meter readings taken at half hourly intervals. The total daily kWh consumption of the plants is then calculated as the sum of these measurements over a 24-hour period.

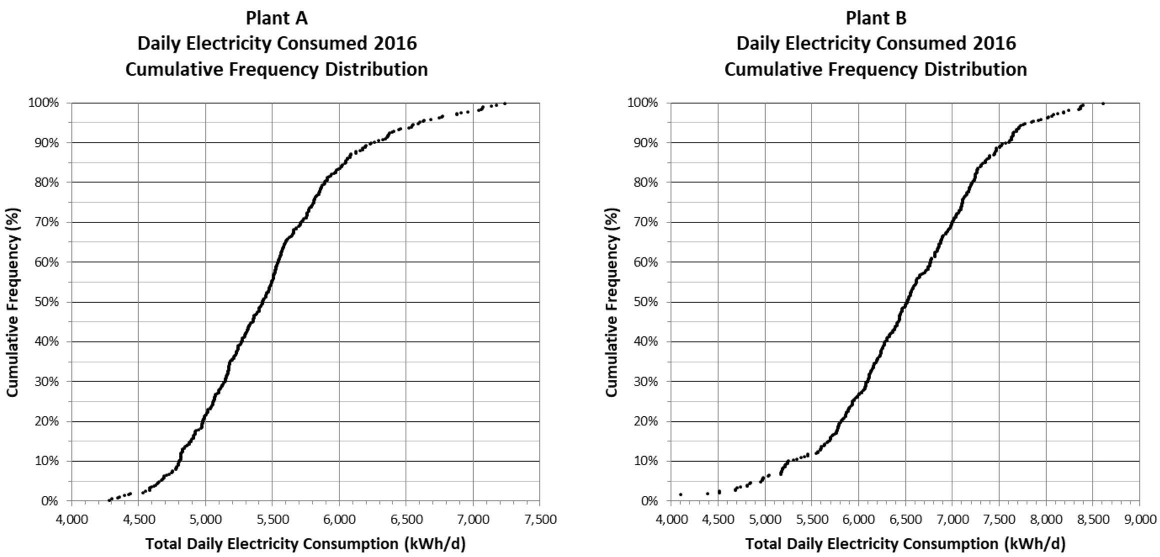


Figure 21: 2016 Electricity Consumption at Plant A & B

The electricity consumption data for three pumping stations in the drainage area for both Plant A & B were also made available. The cumulative distributions of electricity consumption for three wastewater treatment pumping stations in the Plant A drainage area are shown here as Figure 22. Similar plots for the three terminal pumping stations in the Plant B drainage area are also shown in Figure 23.

It should be noted that while there are only three terminal pumping stations at Plant B, all of the electricity consumption for 2016 is shown in Figure 23, there is a total of ten pumping stations before the headworks at Plant A. While the electricity data for the remaining seven stations was available, in order to keep data collection manageable the data collected and represented in Figure 22 is for the three largest pumping stations in the Plant A drainage area.

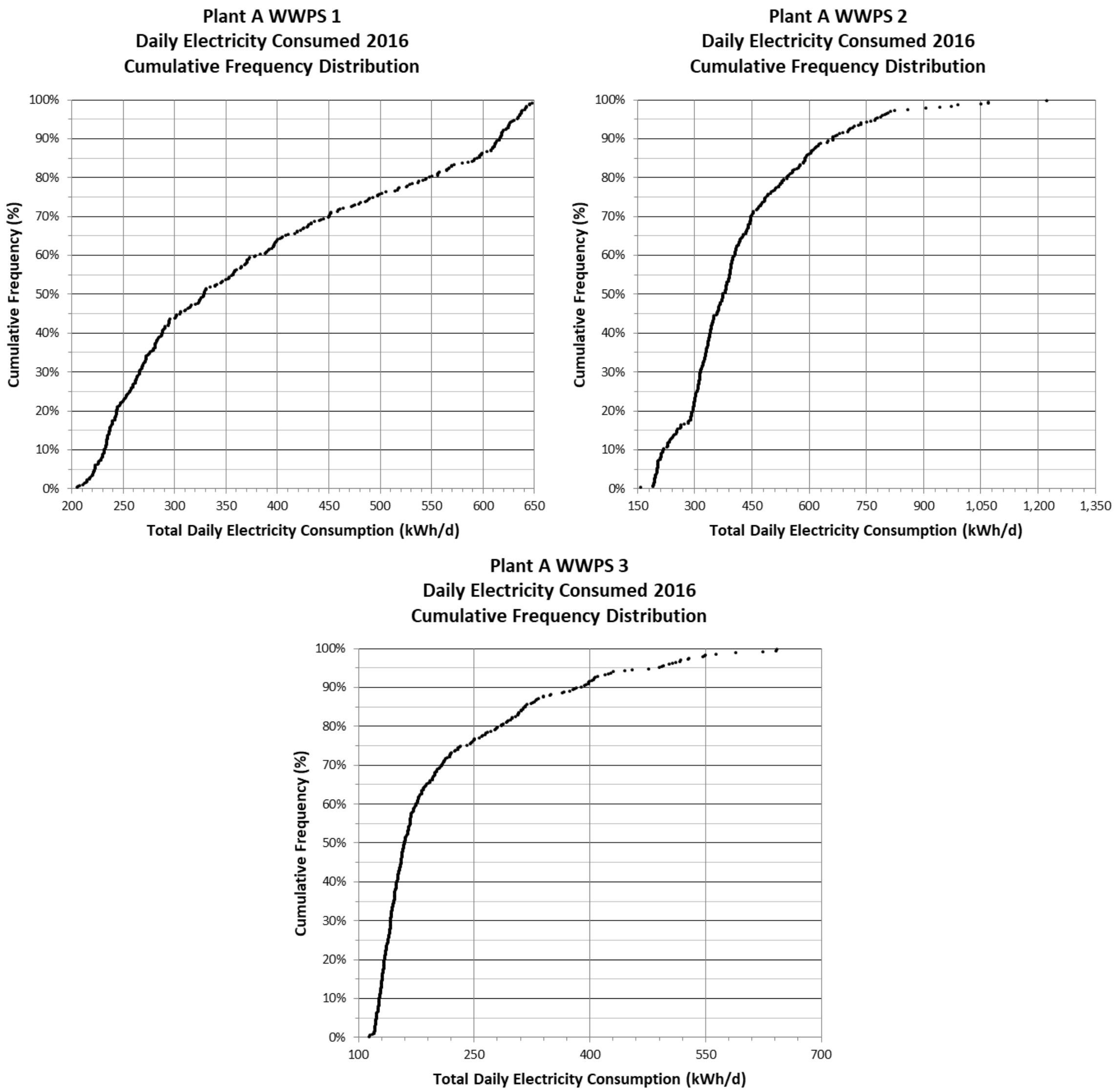


Figure 22: 2016 Electricity Consumption at 3 WWPSs in Plant A Drainage Area

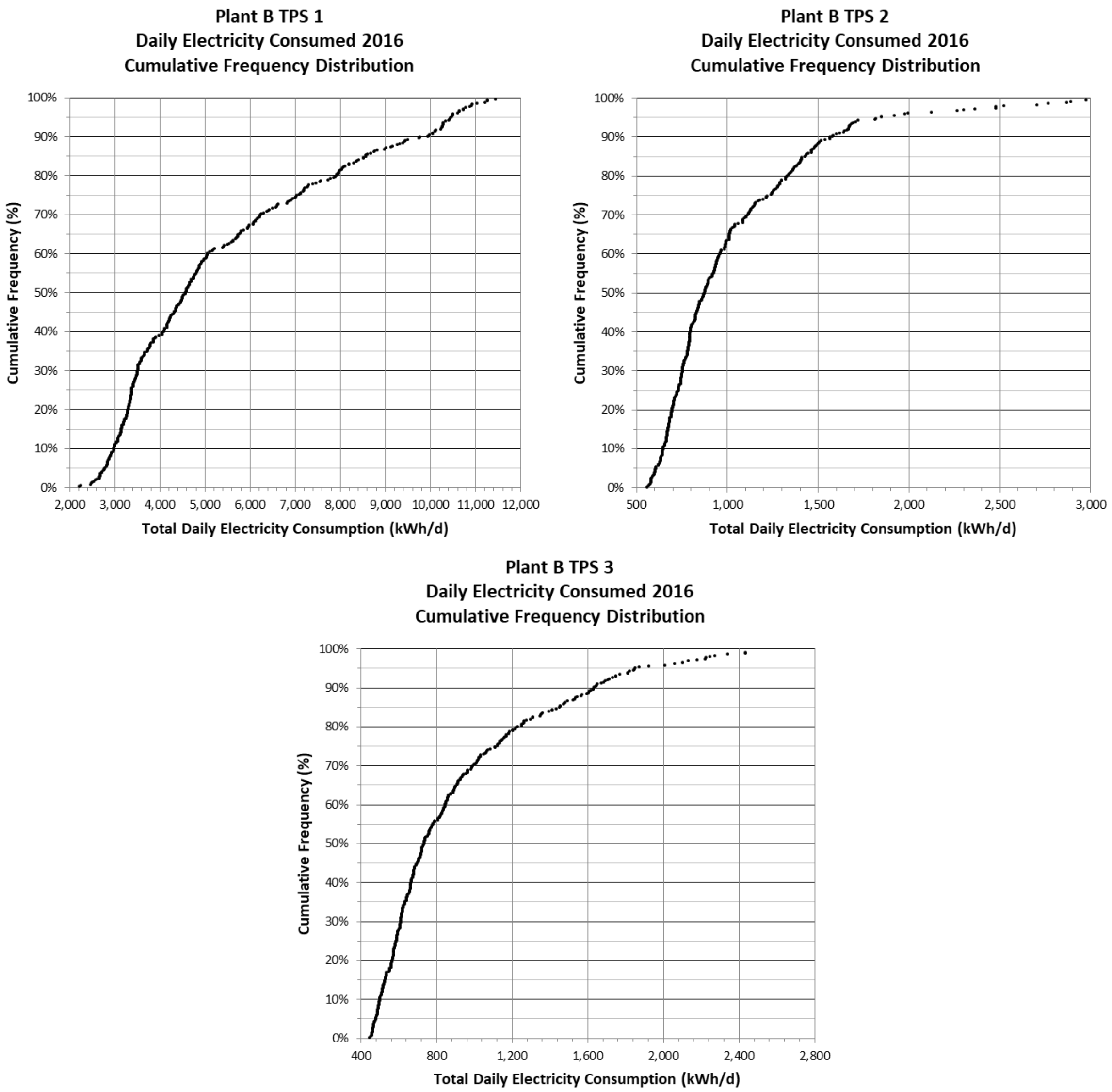


Figure 23: 2016 Electricity Consumption at 3 TPSs in Plant B Drainage Area

UK Met Office Data

Rainfall Data

In addition to the data collected from NI Water, rainfall data was also collected from the UK Met Office's online database (Met Office, 2019). For this research a monitoring station within the drainage area of Plant B was selected, following similar methods used elsewhere in the literature (Richard O Mines Jr et al., 2007). Daily rainfall measurements are taken at this station and are measured to the nearest 0.1 mm. The rainfall data for this station was collected from the database for the year 2016 and a sample is shown here as Figure 22.

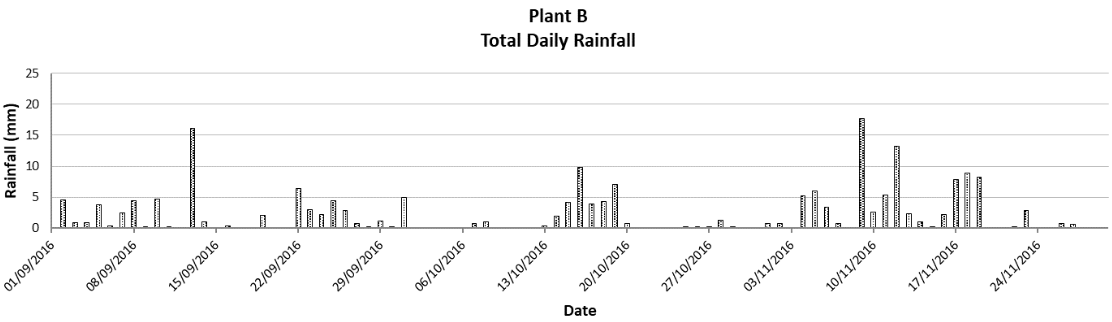


Figure 24: Sample of Plant B Daily Rainfall Dataset

Research Objectives

The analysis performed as part of this research is broken down into four sections based on the research objectives outlined in the previous section:

- Average vs. Median Flows
- Drainage Area Electricity Consumption
- Determination of Treatment Base Flows
- Using the Non-Linear Reservoir Model as a predictive tool

Average vs. Median Flows

The data from NI Water indicates that, for the period 2016 to 2017, 90% of the daily flow rates at Plant A were between 10,786 m^3 and 26,771 m^3 and 12,446 m^3 to 39,863 m^3 for Plant B. Both the cumulative distributions shown in Figure 19 and the histograms shown in Figure 20 indicate that the flow rates to treatment for both Plant A and Plant B are not normally distributed. This concurs with information found during the literature review, i.e. that WWTP flows to treatment are usually log-normally distributed (Burton et al., 2013; Weiss, 2005), not normally distributed.

The recognition that flow data is not typically normally distributed should have implications for any analysis of the flow data, as many commonly used tools used in data analysis rely on the assumption that the data is normally distributed. Though there are other tools available where data is not normally distributed (Weiss, 2005), in data analysis it is up to the individual to make the decision on the correct toolset to use and it is possible that selecting the wrong tool for the wrong type of data can give incorrect or erroneous results. For instance, when analysing right skewed datasets the arithmetic mean of the data is greater than the median of the same data because the mean of any dataset is more sensitive to outliers.

Despite this the mean of flows are still used for certain calculations in relation to wastewater treatment. An example of the continued use of the mean for such calculations in an Irish context can be seen in the application for a wastewater discharge consent by Louth County Council for the wastewater treatment plant in Dundalk (Louth County Council, 2007). While the full data set used in this appendix is not shown the mean as well as the standard deviation, another measure that should be reserved for normally distributed datasets, are both referenced in relation to the flows used to calculate the PE and capacity values for a plant that is still in operation today.

Where these values are used in the context of design capacity the choice of summary statistic will inevitably have an impact on benchmarking where utilisation factors are to be included and considered. To investigate the impacts of using mean values inappropriately, in the context of flows to treatment, the mean and median flow values for the data collected from NI Water were collected and compared.

Drainage Area Electricity Consumption

The electricity used during treatment pumping is a considerable consumer of energy. Despite this, few studies have considered the impacts of electricity consumption of pumping throughout the drainage area. As outlined in previous sections, wastewater pumping stations are often scattered throughout the drainage areas in Northern Ireland. Such stations are required to overcome local topography by pumping wastewater to the wastewater treatment plant. In addition to this, combined sewer overflows (CSOs) at times require pumping to discharge excess flows to receiving waters, representing another consumer of electrical energy.

To investigate this, in the context of the data available from NI Water, the electricity consumption data collected for pumping stations in the drainage areas of Plant A & B was quantified and considered in conjunction with the electricity consumption data for the treatment plants for the year 2016. Plant B has three large terminal pumping stations prior to the headworks of the treatment plant, so these plants were used in this analysis. In the case of Plant A there are ten pumping stations of varying size pumping to the treatment plant. In order to keep data collection manageable for staff at NI Water the electricity consumption for the largest three of these plants was collected.

The total annual electricity data for the three pumping stations at each plant was then summed and combined with the annual electricity consumption at the associated plant. This was then divided by the total treated influent flows in the same period (2016) to give a revised formula for the calculation of specific volumetric electricity consumption (kWh/m^3). An outline of these calculations is presented here as equations (16) & (17).

$$Plant\ A\ (kW\ h/m^3) = \frac{WWPS\ 1 + WWPS\ 2 + WWPS\ 3 + WWTP\ A}{2016\ Total\ Treated\ Flow} \tag{16}$$

$$Plant\ B\ (kW\ h/m^3) = \frac{TPS\ 1 + TPS\ 2 + TPS\ 3 + WWTP\ B}{2016\ Total\ Treated\ Flow} \tag{17}$$

Where:
WWPS N is the total annual electricity consumed at the WWPS N
WWTP A is the total annual electricity consumed at Plant A
TPS N is the total annual electricity consumed at TPS N
WWTP B is the total annual electricity consumed at Plant B
2016 Total Treated Flow is the total flows treated at each plant.

Determination of Treatment Base Flows

When the total inlet flow at the treatment plant is overlaid on the measurements of rainfall in the catchment area, as is shown here in Figure 25, a trend emerges in the days following a peak rainfall event. When the flows to treatment were modified by a rainfall event the inlet flow to treatment would increase but returned toward a steady state flow at what appeared to be a relatively constant rate.

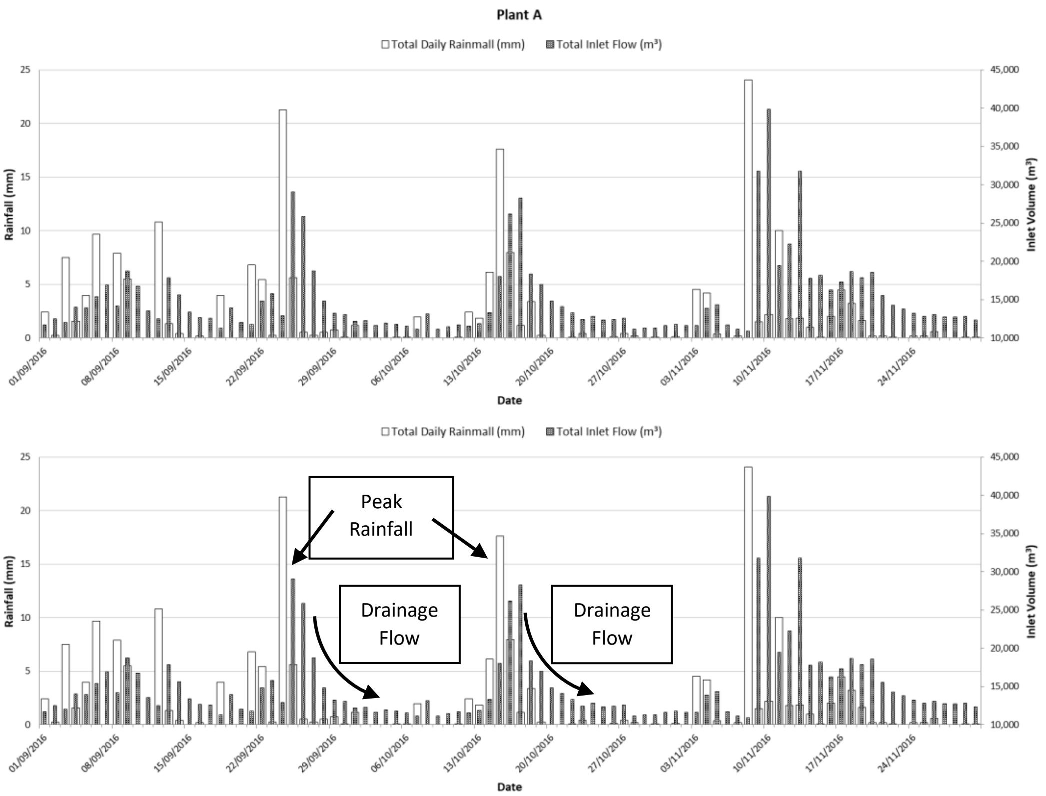


Figure 25: Plant A Rainfall and Total Inlet Flow Comparison

The example in Figure 25, showing the beginning of September to the end of November 2016, shows examples of where the flows appear to be outside of this pattern of rainfall and drainage. On closer inspection however it can be seen that there are few days without rainfall in this time period, which would make the expected behaviour more complex. Despite the increase in complexity there still appears to be a drainage period following a rainfall event where the flows to treatment each day declines at a constant rate. In applying the non-linear reservoir model, as is presented in this research, it is hypothesised that this constant rate is related to the K-constant in the non-linear reservoir model. This hypothesis is based on comparing the urban drainage environment being modelled in this research with the typical application of the non-linear reservoir in assessing natural surface runoff following rainfall events. The K-constant dictates the rate at which flows drain away, and while this rate may vary between urban areas and the natural environment, the underlying principles are applicable in both cases.

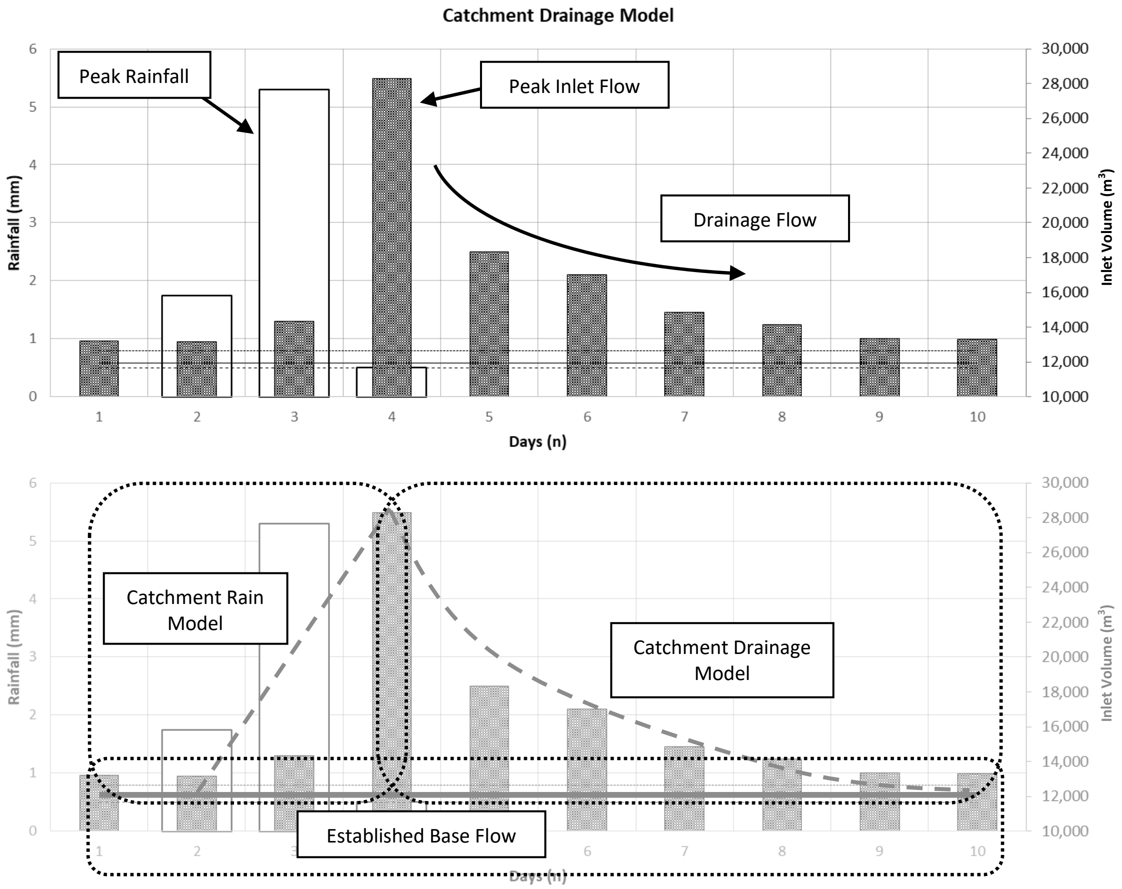
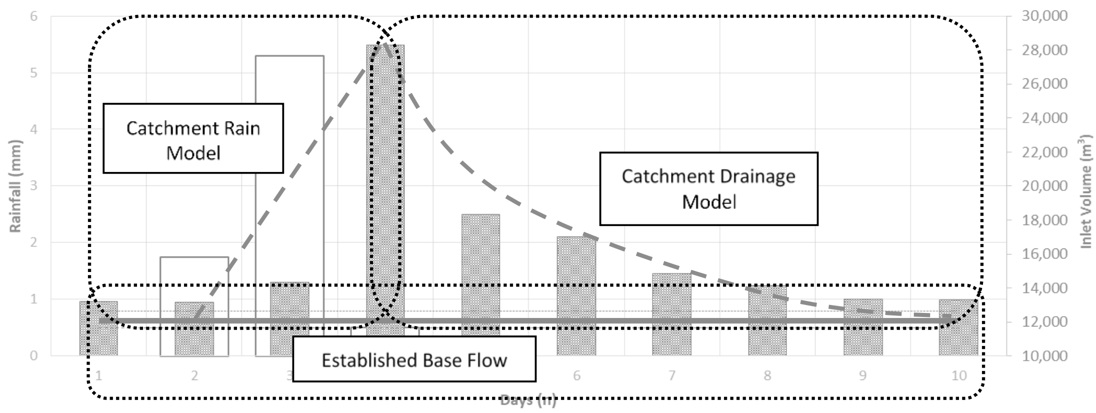


Figure 26: Characterising Rainfall Events

Taking a closer look (Figure 26) at one event shown in Figure 25, three distinct sections were isolated. The first was the base flow, which is the continuous daily flow without the influence of rainfall events. It is assumed that this flow is relatively constant, predictable, is a function of human behaviour in the catchment area and is related to the treated PE at the WWTP. This determination has been made as human activity is considered relatively predictable when compared to the influence of other factors such as weather.

The second section was the period between the beginning of the rainfall event and the peak inlet flow at the headworks of the WWTP. It is assumed that this section is only the result of rainfall in the catchment area resulting in inflow and infiltration to the collection system.

The third and final section of the event is the drainage period, or the time taken for the catchment area to drain into the collection system following the peak flow. As has been previously stated, it is assumed that this decay rate is constant, as it is a function of the hydro-morphology of the drainage area.



$$Q_2 = \underbrace{\boxed{Q_1 e^{-K(t_2-t_1)}}}_{\text{Drainage Model}} + \underbrace{\boxed{i\left(1-e^{-K(t_2-t_1)}\right)}}_{\text{Rain Model}}$$

Figure 27: Modelling the Catchment

This research proposes that the non-linear reservoir model can adequately model the flows through the drainage area as a result of rainfall. When considering the non-linear reservoir equation it can be seen in Figure 27 how the equation represents two of the three sections of a rainfall event as discussed. The first term on the right hand side of the equation represents the drainage section of the event, while the second term on the right represents the rainfall section.

In dealing with the base flow section, there are two options: the first is to retain this portion of the flow when calculating the K-constant and when using the non-linear equation to model the flows through the drainage area. This option is the simplest and assumes that the drainage response faction (K-constant) will adequately describe the return to the steady state flow of the system (i.e. the base flow).

The second option is to remove the base flow portion of the flows when calculating the K-Constant and when using the non-linear reservoir equation to model the flows due to rainfall. The base flow can then be re-introduced to give a total flow to treatment at a later stage. Separating the base flow from the rainfall excess calculations means that the factors that influence this (e.g. population, industrial development, tourism etc.) can be treated independent of the investigation of the impacts of rainfall in a drainage area. While this introduces an added level of initial complexity, the alternative would require introducing the factors that determine the base flow into the calculation of the drainage area response factor (K) at a later stage, thereby increasing the intricacy of this calculation at a later stage. For this reason it was decided to determine and remove the base flows prior to applying the non-linear reservoir equation for this research.

Two methods for estimating the base flow using the available data are investigated: the first is to take the median of the annual flows and the second is to calculate the median of a subset of the annual flows on dry weather days (DWDs). There were a number of reasons for selecting these methods for investigation. While other measurements, such as influent BOD levels, may be more accurate in determining the base load to the plant this data was not readily available. Flow data is also recorded by automated systems on a more frequent basis than other measurements, and is already available within the automation and control systems within NI Water. Using data already collected and automated in implementing the non-linear model reservoir as proposed will speed up the process of its application in control strategies. It is also hoped that using flow measurements, which are taken at a greater temporal resolution, in conjunction with influent load measurements will aid the further development of the model as a tool to determine influent quality based on changes to dilution due to rainfall. Two methods for calculating these base flows were investigated because of the increased complexity in collecting the data required for and determining the DWFs. If the additional steps taken result in only marginal improvements then the median of all flows could be used as an appropriate estimate of the base flow.

The method for characterising DWDs used rainfall data from the UK Met Office. As can be seen in Figure 25, the inlet flow following a rainfall event typically returned to the median seasonal flow within seven days of the initial peak inlet flow. A dry day could therefore be defined as any consecutive seven-day period, with no rainfall on that day or the 6 previous days. The difficulty with this definition is that it has proven quite difficult to find seven consecutive days without rainfall due to the regional weather conditions. Therefore the definition of a dry day used was the fifth consecutive day with less than 0.5 mm of rainfall. An example of the results of this data processing is shown here as Table 6.

Table 6: Dry Weather Characterisation

	Total Daily Rainfall (mm)	Weather Classification
.	.	.
.	.	.
.	.	.
11/03/2016	1.80	-
12/03/2016	5.60	-
13/03/2016	0.00	-
14/03/2016	0.00	-
15/03/2016	0.00	-
16/03/2016	0.00	DRY
17/03/2016	0.00	DRY
18/03/2016	0.00	DRY
19/03/2016	0.00	DRY
20/03/2016	0.00	DRY
21/03/2016	0.00	DRY
22/03/2016	0.00	DRY
.	.	.
.	.	.
.	.	.

Once the dates of dry days had been established, the flows on those dates can be defined as dry weather flow (DWFs) which were then tabulated, an example of the resulting table is shown here as Table 7. It is the median of these DWFs that was used as the second measure of the base flow in this research.

Table 7: Establishing the Dry Weather Flow

	Weather Classification	Flows (m^3)
.	.	.
.	.	.
.	.	.
16/03/2016	DRY	24,431
17/03/2016	DRY	24,395
18/03/2016	DRY	22,342
.	.	.
.	.	.
.	.	.

Once the median of the DWFs and the median of all flows had been found, the suitability of the values as a representative base flow could be determined by a comparison to a known PE value. NI Water calculate these PE values using BOD measurements and customer information from the drainage area, e.g. the number of connections to the sewer network. To compare the calculated flows to the known PE, an estimated PE was calculated based on a value of 200 L/PE/day. A second check could be performed by estimating the volume of wastewater treated per PE in the catchment area. This number could then be compared to NI Water's estimated water usage across their network per head of population, which is 145 L/d.

Using the Non-Linear Reservoir Model as a predictive tool

Following the assessment of electricity consumption in the drainage area, which will be discussed further in the results & discussion section, the utility of a tool that could adequately describe the electricity consumption in the drainage area became apparent. To increase the utility of such a tool further it was decided that a modular approach that allows analysis of the variation due to the different sources of influent to a treatment plant. One of the greatest sources of influent variations to treatment are rainfall events in the catchment area, which consequently have direct implications for electricity consumption. While models do exist for the assessment of drainage patterns and resultant flows through drainage networks, such models typically require a level of data that may not be accessible to researchers or potentially even staff within utilities. For this reason it was decided to first attempt to model the impacts of rainfall on flows to treatment in cases with minimal data availability. Additionally, the potential of such a model to be able to use forecasted weather data to predict the impacts of rainfall events was also to be assessed. The steps used to derive the model for the wastewater treatment plant and catchment area under investigation are outlined here.

The flows to treatment data for Plant B as well as outlet flows for TPSs 1, 2 & 3 that Northern Ireland Water had recorded for 2016 as well as rainfall data for the drainage area from the same period formed the basis for this analysis. The main objective is to describe the general behaviour of the catchment area following a rainfall event. To meet this objective the previously described non-linear reservoir model for drainage modelling was used. The choice of this model is based on several factors, first being the relatively small amounts of data required. Secondly, referring to Figure 27, it is seen that the non-linear reservoir equation describes two of the three phases of rainfall events discussed in previous sections. While this model is typically used to model surface rainfall run-off in hydrogeology, to the author's knowledge it has never been used in the manner discussed here and represents a novel use of the model.

In order to apply the non-linear reservoir model flow measurements are needed. Under a typical application of the model the drainage channel parameters are measured and the speed of flow through them after different rainfall events are recorded to obtain the required volumetric flows. Given the scale of obtaining all the required measurements to calculate the flows through the specific sewer pipes within a SDAC it was deemed impractical to undertake such a measurement regime. An alternative was identified based on NI Water's own measurement scheme: the outlet flows from each of the three terminal pumping station were measured by NI Water already, as were the inlet flows at the headworks of the treatment plant.

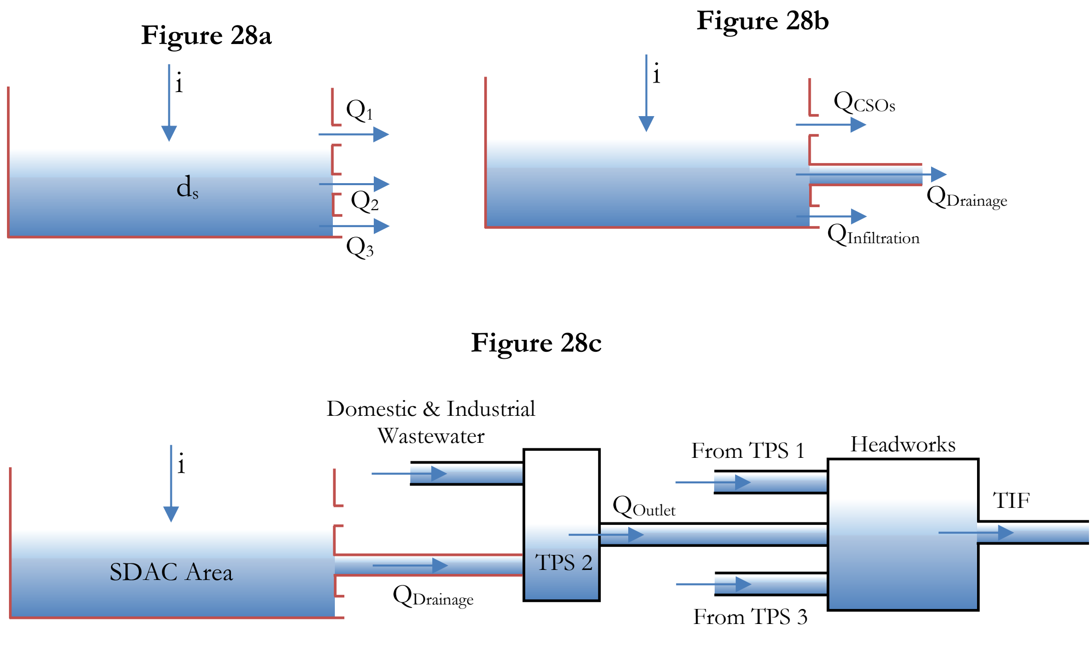


Figure 28: Application of the Non-Linear Reservoir Model

Figure 28a shows the basic outline of the non-linear reservoir model as discussed in the previous section (Figure 15): the outlet flows (Q_1,Q_2 & Q_3) are a function of the catchment area, the stored depth of water in the catchment area (d_s) and the input to the catchment area due to rainfall (i). Figure 28b shows a more practical example of how the non-linear reservoir model may be used to solve for a flow. The different flows out of the drainage area in this case may refer to the CSO flows (Q_{CSO}) or the flows to groundwater due to infiltration ($Q_{Infiltration}$). The flow to the drainage system ($Q_{Drainage}$), due to inflows and infiltration to the network, are of interest in this analysis. Figure 1c shows how the non-linear reservoir model is applied in the specific case of this drainage area and could be modified to meet other requirements. Shown in Figure 28c (Q_{Outlet}), these flows are a combination of the domestic and industrial wastewater flows normally served by the drainage system and the excess flows due to rainfall in the catchment area. In order to apply the non-linear reservoir model these flows would need to be separated so the $Q_{Drainage}$ could be estimated. Based on the results of the analysis discussed previously, the dry weather flow (DWF) was deemed a suitable proxy for a measurement of the wastewater flows due to human activity, also referred to as the base flow. By subtracting the median of the DWFs from the daily measured flows at the outlet of the TPSs, the drainage flows from the SDAC ($Q_{Drainage}$) due to rainfall events could be approximated.

Once the $Q_{Drainage}$ was determined it was used to determine the "K-Constant", or drainage parameter, for each of the SDACs under investigation here. When the K-constant was found for each of the SDACs each one was used in the non-linear reservoir model to predict the expected flows from each SDAC based on the rainfall in the drainage area.

These same calculations were also performed for the total inlet flows at the headworks, which were also available. This method considered the entire catchment as a single non-linear reservoir that discharged into the headworks, treating the three SDACs as a single drainage area. This means that a single K-constant for the overall drainage area was determined and used to predict the flows to treatment based on given rainfall events.

The method of analysis that follows is broken down into two phases: the preliminary and predictive stages. Stage One, presented in Figure 29, is where the availability of relevant data is assessed and the catchment area parameters are determined. This work is performed prior to predictions and can be done with relevant historical data if available. As time progresses and additional data becomes available this stage can be repeated to update parameters such as the drainage area response factor (K) and median DWFs.

Stage two (described in Figure 30) is the predictive phase and is where the model is used to determine the predicted flows to treatment based on the rainfall measurements in the catchment areas. This is done by using the data and catchment parameters that were determined during stage one and is therefore dependent on its completion.

Phase One – Catchment Parameters

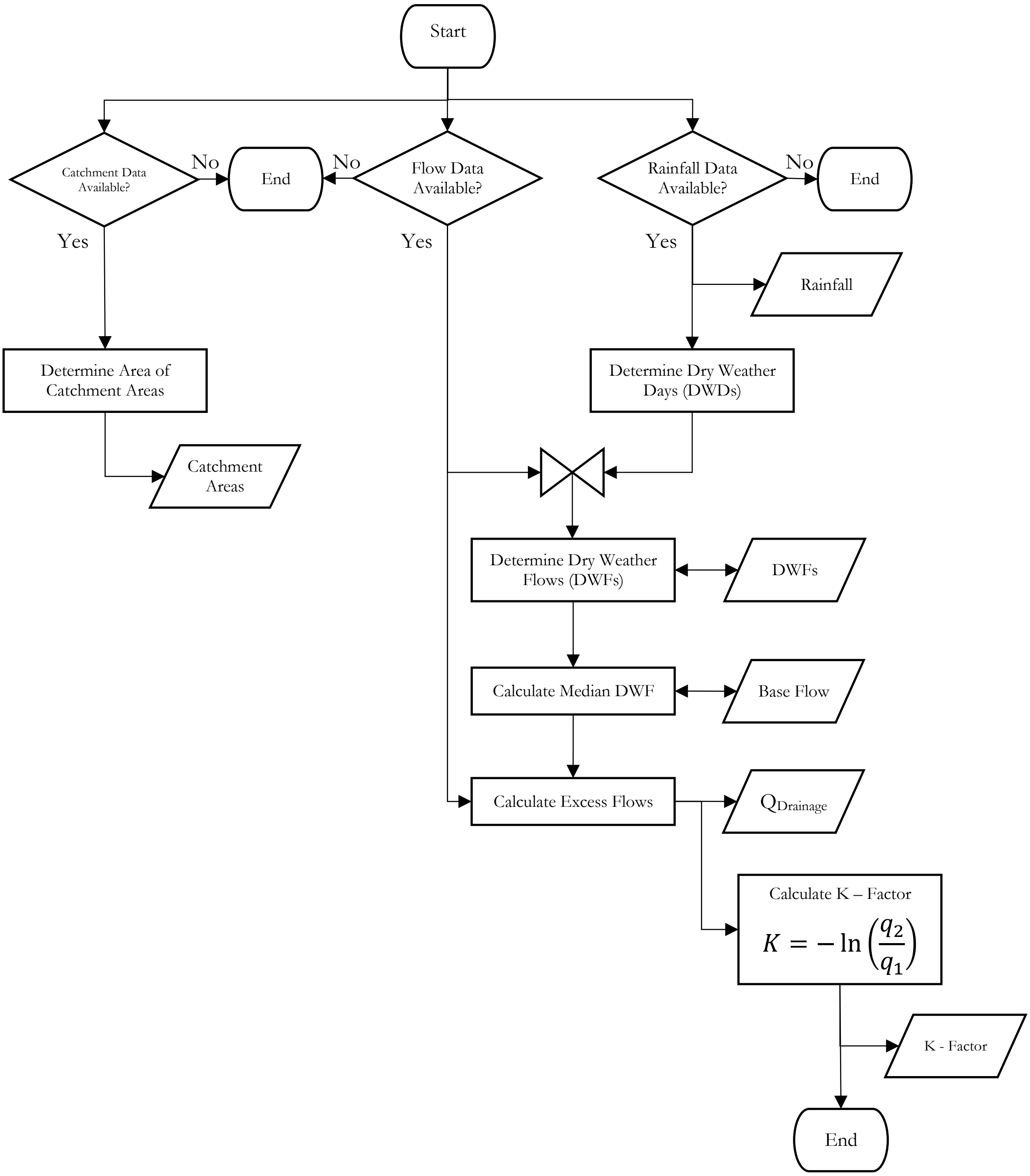


Figure 29: Stage One Workflow

Phase one begins with an assessment of the available data for the drainage area under investigation. In the case of this analysis much of the methodology has been developed based on the available data, though the methodology could be modified following an analysis of the available data in other drainage areas.

Step One

The first branch of the workflow diagram shown in Figure 29, i.e. the branch farthest to the left, assesses the availability of data relating to the area of land served by the drainage system. It is also necessary to investigate all relevant sub-divisions of this drainage area, as the areas of the drainage area and sub-divisions are needed later in the analysis.

According to the data made available by NI Water, for the drainage system being investigated here, the drainage area was broken down into sixteen sub-catchments (SCs). These SCs are then grouped together into three larger sub-drainage area catchments (SDACs). The SCs and relevant SDACs are shown in Table 8.

Table 8: Drainage Area Breakdown

Plant B	
Sub-Catchment	**Sub-Drainage**
SC 1	SDAC 1
SC 2	SDAC 1
SC 3	SDAC 1
SC 4	SDAC 1
SC 5	SDAC 1
SC 6	SDAC 1
SC 7	SDAC 1
SC 8	SDAC 1
SC 16	SDAC 1
SC 9	SDAC 2
SC 10	SDAC 2
SC 11	SDAC 3
SC 12	SDAC 3
SC 13	SDAC 3
SC 14	SDAC 3
SC 15	SDAC 3

The second branch of the workflow diagram shown in Figure 29 checks the availability of relevant flow data for the drainage system under investigation. The drainage area investigated was one of the better monitored catchment areas under NI Water, with flow data available for a total of six points throughout the network. Three terminal pumping stations (TPSs) serving the catchment area, one for each of the SDACs, had flows measured at their outlets and a composite breakdown of two inlet flows and the total inlet flows at the treatment plant. The composite breakdown of the inlet flows measured the flows coming from SDAC 1 and a combined flow from SDACs 2 & 3. The total inlet flows (TIFs) were also available, which is a measurement downstream in the plant of the total flows through the headworks. A sample of these flows are shown in Table 9.

Table 9: Flow Data Table

	TPS Outlet Flow				Inlet Flow Measurements
	SDAC 3	SDAC 2	SDAC 1		TIFs
Date	(m^3)	(m^3)	(m^3)		(m^3)
01/01/2016	9,819	9,906	27,894		47,817
02/01/2016	6,865	6,683	27,294		43,387
03/01/2016	6,591	5,929	23,867		38,165
04/01/2016	6,425	5,896	22,615		37,920
05/01/2016	9,532	9,918	25,218		47,081
06/01/2016	6,891	7,491	26,243		43,641
…	…	…	…		…
…	…	…	…		…
…	…	…	…		…

The final branch, i.e. on the top right of the workflow diagram, checks whether there is relevant rainfall data available for the analysis. When assessing this data care should be taken to select measurement stations close to the area under investigation, wherever possible. The availability of measurements from several sources should also be investigated, some utilities take rainfall measurements at the headworks of plants, but relevant data may also be available from the relevant meteorological offices for a region. Furthermore, the possibility for variable weather conditions within catchment areas should also be considered. Rainfall may be measured at one point while no rainfall is recorded a short distance away and vice versa. For this reason, multiple measuring locations within the catchment area would be the ideal, with a single measurement within the catchment area being used only where necessary.

Step Two

The second step of Stage One the data relating to the drainage area and the rainfall is collected, collated, and presented in a manner suitable for the analysis.

The SDACs connected to each of the terminal pumping stations were made available as pdf maps by NI Water. These pdf maps were imported to the QGIS software package and a GIS file was created using the data. As outlined previously, the total drainage area is served by three TPSs, which are connected to three SDACs, made up of the 16 sub-catchments (SC). In order to calculate the areas of the SDACs, the relevant SC areas are summed. An outline of these SCs and SDACs, as well as the areas determined using the GIS data, are shown in Table 10. The areas shown in Table 10 are assumed to be constant throughout 2016 for the purposes of this study.

Table 10: Catchment Area Information

Plant B		
Sub-Catchment	**Sub-Drainage**	**Area**
		(m^2)
SC 1	SDAC 1	1,009,436
SC 2	SDAC 1	699,511
SC 3	SDAC 1	1,703,969
SC 4	SDAC 1	367,126
SC 5	SDAC 1	7,533,688
SC 6	SDAC 1	136,512
SC 7	SDAC 1	900,441
SC 8	SDAC 1	300,937
SC 16	SDAC 1	296,787
SC 9	SDAC 2	3,279,146
SC 10	SDAC 2	146,835
SC 11	SDAC 3	207,013
SC 12	SDAC 3	384,950
SC 13	SDAC 3	110,272
SC 14	SDAC 3	1,728,632
SC 15	SDAC 3	387,875

Step Two also requires the preparation of the rainfall data and the determination of the dry weather days (DWDs) for the period considered. As outlined in previous sections, the method used defined a DWD as the fifth consecutive day with less than 0.5 mm of precipitation. Once the DWDs have been determined the measured flows through the network on these days are then categorised as dry weather flows (DWFs).

Step Three

Once the Dry Weather Flows (DWFs) are known the median of these flows is calculated and is assumed to be the daily wastewater flow through the network from industrial and domestic sources. Two median flows were investigated: the first was the annual median DWF and the second was the seasonal median DWF. The reason for this was to investigate the influence of seasonal variations in the base flow, due to human activity and other factors, and the possible impact of this on the predicted values.

As stated, the difference between the DWF and the daily measured flow is assumed to be the drainage flow ($Q_{Drainage}$). It is these calculated drainage flows that are used to determine the catchment's drainage area response factor (K) for the area under investigation. The equation used to determine this value is derived directly from the non-linear reservoir equation discussed in the previous section, shown as equation (13).

$$Q_2 = Q_1 e^{-K(t_2 - t_1)} + i\left(1 - e^{-K(t_2 - t_1)}\right) \tag{13}$$

Where:
Q_1 is the drainage flow rate at time t_1 (m^3/t)
Q_2 is the drainage flow rate at time t_2 (m^3/t)
K is the drainage area response factor (1/t)
i is the volume of rainfall per unit time (m^3/t)

To use equation (13) to determine the drainage area response factor (K) for the catchment area, the rainfall (i) in the second term on the right hand side of the equation must be equal to zero, which is the reason for selecting a day with no rainfall. Once this condition has been met equation (13) can then be rewritten as equation (14) and can then be rearranged to determine K equation (15).

$$Q_2 = Q_1 e^{-K(t_2 - t_1)} \tag{14}$$

If $\Delta t = 1$ (i.e. $t_2 - t_1 = 1$) then equation (14) can be re-arranged as:

$$K = -\ln\left(\frac{Q_2}{Q_1}\right) \tag{15}$$

Where:
Q_1 is the drainage flow rate at time t_1 (m^3/t)
Q_2 is the drainage flow rate at time t_2 (m^3/t)
K is the drainage area response factor (1/t)

Since equation (14) describes the flow Q_2 only in terms of the drainage from the drainage area without additional rainfall based on the previous day's flow (Q_1), using equation (15) has a second requirement to determine the drainage area response. This is that no additional rainfall occurs between t_1 and t_2, but there must be rainfall prior to t_1 in order to determine the drainage parameter (K). For this reason when using equation (15) to determine K in this analysis, Q_2 is taken from days with no rainfall that had been preceded by at least one day with rainfall. These flows represent the drainage flows ($Q_{Drainage}$) that were determined during the previous process, Q_1 being the calculated $Q_{Drainage}$ on the first day and Q_2 is the $Q_{Drainage}$ on the second day.

Phase Two – Predictive Stage

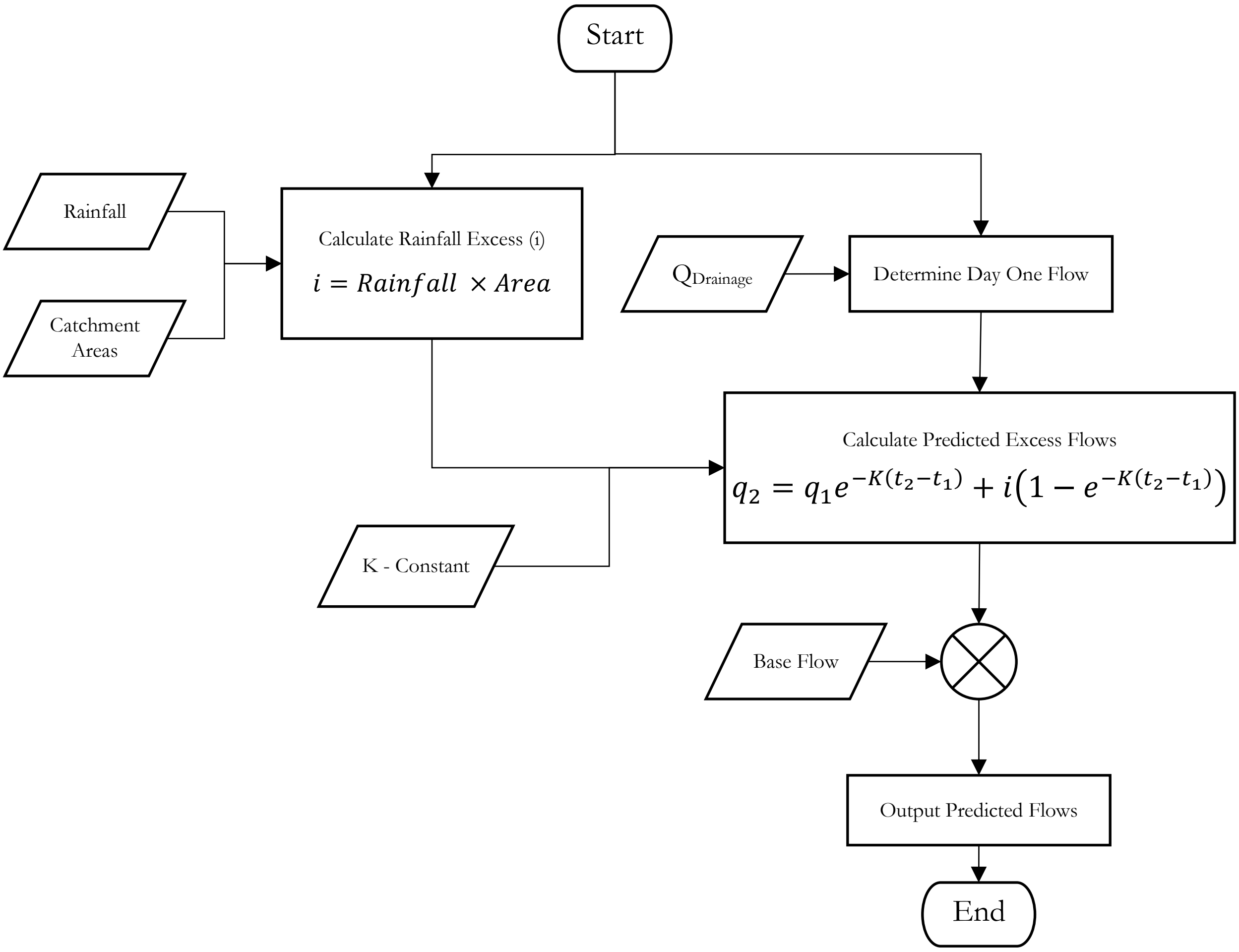


Figure 30: Stage Two Workflow

The workflow for phase two of the methodology is shown in Figure 30 and shows the predictive stage of the methodology. Once the catchment parameters have been determined in phase one they are then be used to predict the flows through the drainage area based on known rainfall measurements.

Step One

Step one of stage two is broken into two branches, the first (farthest left, Figure 30) involves the calculation of the estimated daily volume of rainfall that fell in the drainage area (i) based on rainfall measurements. The daily rainfall measurements used in this analysis are the same as were used in the determination of the DWDs and were collected from the UK Met Office for 2016. In addition to these rainfall measurements, the SC and SDAC areas found during stage one were used in the calculation of the daily rainfall volume.

The second branch in step one uses the measured flows at points throughout the network to determine the "Day One Flow". This is the $Q_{Drainage}$ for that day, i.e. the measured flow minus the median DWF, on the first day of the analysis which is assumed to be a known flow.

Step Two

Once the drainage area response factor (K) has been determined, the daily rainfall volume has been calculated, and the day one flow is known then equation (13) can be used to determine the predicted drainage flows ($Q_{Drainage}$). As stated previously it is assumed that the day one flow (Q_0) is known and what is being predicted is the $Q_{Drainage}$ (Q_{n+1}) for the following four days based on known rainfall measurements for each of the four days and the initial known flow. The iterative calculation that is performed is shown here as equation (18).

For n = 0, 1,…,4 :

$$Q_{n+1} = Q_n e^{-K(t_2-t_1)} + i_{n+1}\left(1 - e^{-K(t_2-t_1)}\right) \tag{18}$$

Where:
It is assumed q_0, or the day one flow, is a known measurement and that $t_2 - t_1 = 1$
n is the day for which flow is being calculated
q is the drainage flow (m^3/t)
i is the rainfall volume (m^3/t)
K is the drainage constant

On day 1 (n = 0) the predicted drainage flow for day 2 (n = 1) is calculated based on the measured flow for day 1. The day 3 (n = 2) is calculated similarly, but this time Q_n is assumed to be the flow predicted for day 2 (n = 1). This is the process used for each of the remaining predicted excess flows: each of the days Q_{n+1} is based on the previous days predicted excess flows.

An outline of how the calculations are performed and presented is shown here as Table 11, with the predicted excess flow for a particular day being a function of the rainfall measurement for that day as well as the predicted or measured flow for the previous day. In this way four days of excess flows were predicted based on one day's known flow and four days of rainfall measurements, although the analysis here used actual measured rainfall data for this calculation, not predicted values.

Table 11: Predicted Excess Flows

		SDAC 1					
	Rainfall Volume	**Measured Drainage Flow**		**Predicted Drainage Flow ($Q_{Drainage}$)**			
				Day 1+1	**Day 1+2**	**Day 1+3**	**Day 1+4**
Day 0	i_1 m^3	Q_0 m^3		$Q(i_2, Q_{Day\ 1})$	$Q(i_3, Q_{Day\ 1+1})$	$Q(i_4, Q_{Day\ 1+2})$	$Q(i_5, Q_{Day\ 1+3})$
Day 1	i_2 m^3	Q_1 m^3		$Q(i_3, Q_{Day\ 2})$	$Q(i_4, Q_{Day\ 2+1})$	$Q(i_5, Q_{Day\ 2+2})$	$Q(i_6, Q_{Day\ 2+3})$
Day 2	i_3 m^3	Q_2 m^3		$Q(i_4, Q_{Day\ 3})$	$Q(i_4, Q_{Day\ 3+1})$	$Q(i_5, Q_{Day\ 3+2})$	$Q(i_6, Q_{Day\ 3+3})$
Day 3	i_4 m^3	Q_3 m^3		$Q(i_5, Q_{Day\ 4})$	$Q(i_4, Q_{Day\ 4+1})$	$Q(i_5, Q_{Day\ 4+2})$	$Q(i_6, Q_{Day\ 4+3})$
Day 4	i_5 m^3	Q_4 m^3		$Q(i_6, Q_{Day\ 5})$	$Q(i_4, Q_{Day\ 5+1})$	$Q(i_5, Q_{Day\ 5+2})$	$Q(i_6, Q_{Day\ 5+3})$
…	…	…		…	…	…	…
…	…	…		…	…	…	…
…	…	…		…	…	…	…

Once the predicted drainage flows were calculated, the predicted total flow was determined by adding the predicted drainage flow to the initial median DWF value that was used.

Revised Predicted Flows

The predicted daily flows were then plotted against the measured TPS outlet flows and headworks inlet flows for the same day to assess the accuracy of the approach. The resulting graph, seen here in Figure 31, shows a logarithmic trend of reasonable significance. This indicates that as the predicted flows increase, the measured flows do not increase to the same degree and potential reasons for this are discussed in the following sections.

As a result of this a final step not shown in the process workflow diagrams in Figure 29 and Figure 30 is added, that applies the logarithmic equation of this trend line to the predicted flows to give a revised predicted flow. The equations used to calculate these revised predicted flows are shown here in Table 12. This additional step is performed only on the data using the annual median flow at the three TPS outlet flows and the Total Inlet Flow (TIF) at the headworks.

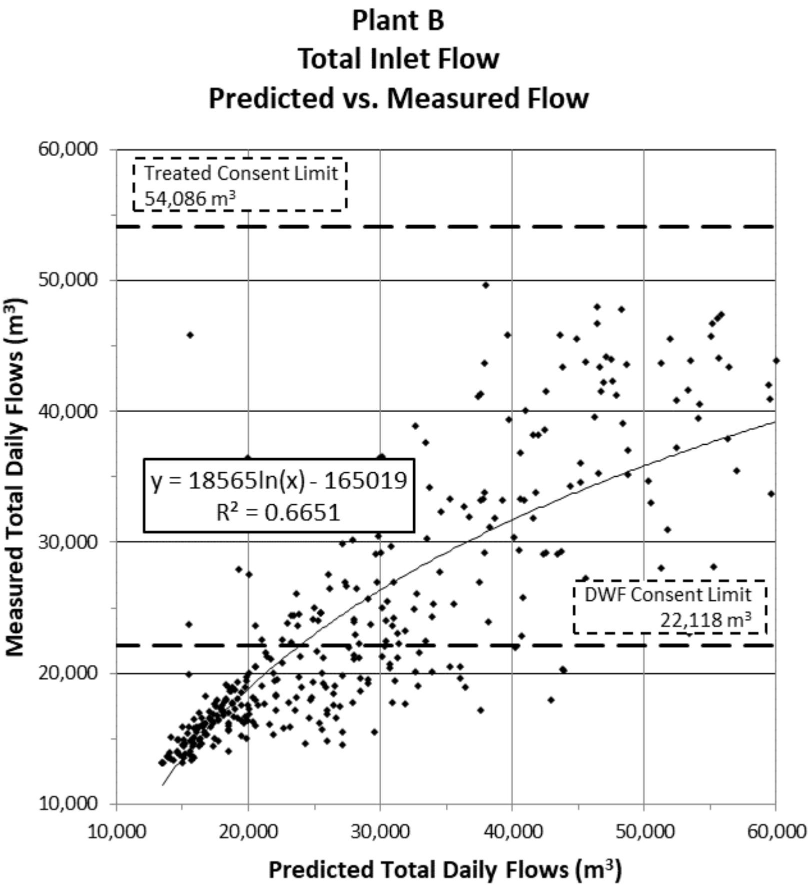


Figure 31: Total Inlet Flow Predicted vs. Measured Graph

Table 12: Revised Predicted Flow Equations

TPS Outlet Flow Equations

SDAC 1 $$Flow_{Revised} = 10{,}195\ln(Flow_{Predicted}) - 84{,}021 \quad (19)$$

SDAC 3 $$Flow_{Revised} = 2{,}988\ln(Flow_{Predicted}) - 20{,}768 \quad (20)$$

SDAC 2 $$Flow_{Revised} = 3{,}322\ln(Flow_{Predicted}) - 23{,}237 \quad (21)$$

Total Inlet Flow Equations

$$Flow_{Revised} = 18{,}565\ln(Flow_{Predicted}) - 165{,}019 \quad (22)$$

Determination of Errors

To determine the appropriateness of these modelling approaches errors were calculated for each location and day using the measured flow rates and equation (23):

$$Model\ Error = Predicted\ Flow - Measured\ Flow \quad (23)$$

A percentage error for each location and day was also calculated using equation (24):

$$\% \; Error = \frac{Predicted \; Flow - Measured \; Flow}{Measured \; Flow} \tag{24}$$

The resulting values were then tabulated and summarised as the median and interquartile range for each predicted day at all SDACs the plant total inlet flows.

Results & Discussion

As with the previous chapter, the Results & Discussion are broken down here into four sections based on the previously outlined research objectives.

Average vs. Median Flows

The calculation of the mean and median of the flow data show how these summary statistics can differ for the same datasets: the arithmetic mean for Plant A & B were found to be 17,681 $m^3\ d^{-1}$ and 24,049 $m^3\ d^{-1}$ respectively, while the medians were calculated as 15,666 $m^3\ d^{-1}$ and 20,703 $m^3\ d^{-1}$ respectively.

In both cases the arithmetic mean overestimates the daily flow for the same dataset for the two plants analysed. Despite this difference it had been noted during the literature review that the arithmetic means are typically used to calculate the flows to treatment expected for treatment plants at the design stage (Louth County Council, 2007; Młyński et al., 2016; U.S. Environmental Protection Agency, 2014), but the median has also been noted by some (Longo et al., 2016).

The arithmetic mean may be appropriate when calculating the values when designing treatment plants as it will overestimate the flows expected to treatment when compared with the median values, giving additional treatment overhead for the plant. When taken in the context of treatment efficiency however, the use of the arithmetic mean compared to the median can be considered an inefficient use of resources and will impact the utilisation factor of plants. The use of these values inherently over-size the treatment plant being designed, leading to capacity in excess of what is needed. This will inevitably have implications on the overall efficiency of treatment and benchmarking of plants making the use of the arithmetic mean questionable in this context. It is for these reasons that for the purposes of this analysis the median flows to treatment as opposed to the arithmetic means will be used for the purposes of comparison.

Drainage Area Electricity Consumption

To investigate the impact that pumping in the wider drainage area can have on the electricity consumption of wastewater treatment the electricity consumption for both plants A & B were considered in conjunction and combined with the electricity consumption at the three largest pumping stations in the relevant drainage area. This electricity consumption was then divided by the total flow to treatment as measured at the headworks of both plants for 2016, giving an annual specific volumetric electricity consumption. The results of these calculations are shown in Table 13.

Table 13: 2016 Energy Assessment of Wider Drainage Area

	Plant A	Plant B
Total WWTP Elec.	2,002,826	2,369,871
PS 1	138,214	1,974,979
PS 2	151,998	374,273
PS 3	76,387	333,800
Total Electricity	2,369,424	5,052,923
kWh/m^3 (WWTP Only)	0.32	0.37
kWh/m^3 (WWTP + PS)	0.37	0.58

When considering the data shown in Table 13 it should be noted that the three pumping stations shown at Plant A only represent the three largest pumping stations and there are 7 remaining pumping stations that were not included in this analysis.

With this said, there is a considerable amount of pumping required in the drainage area of Plant B when compared to Plant A. As can be seen from the results of this analysis in Table 13 there is a significant jump in the electricity consumption at plant B when the three pumping stations are considered. Indeed, at Plant B a single pumping station (PS 1) uses more electricity than the second and third largest pumping stations combined. This is also reflected in the specific volumetric energy consumption for the drainage system.

These two plants were initially chosen for analysis in consultation with NI Water because both were similarly sized and operated plants. The selection was based on the metric typically used to compare plants by NI Water: the electricity consumption per PE (kWh/PE) and NI Water report that Plant A & B used 30 & 31 kWh/PE respectively in 2016. The addition of electricity consumption from the pumping stations however causes these figures to increase to 36 & 65 kWh/PE for plant A & B respectively when using NI Water's reported PE for both plants.

There is a significant difference between the two plants here that must be considered in the context of these new figures. Though there are several relatively small pumping stations in the drainage area, Plant A is mostly a gravity fed sewer network flowing towards the headworks at the plant. Once the wastewater reaches the headworks of the treatment plant it is then raised by a large screw pump works for treatment which is included in the electricity data for Plant A. Plant B on the other hand does not have a large pump at the headworks, despite being situated at a higher elevation than the drainage areas it serves. In order to pump wastewater to the top of this elevation three large pumping stations are required to overcome the height differential, accounting for the considerable electricity consumption in the drainage area.

Staff at NI Water had indicated suspicions that this may be the case, but to the best of the author's knowledge a similar analysis of drainage area pumping has not been done in Northern Ireland before now. The results have many implications for wastewater treatment, particularly in relation to efficiency and benchmarking. NI Water staff indicated that the siting of Plant B at the top of an elevation was largely influenced by social factors, namely a desire to not have a WWTP sited near to dwellings. This reluctance to accept the siting of the plant at a more reasonable site with lower elevations has contributed to the electricity consumption at the pumping stations and consequent environmental impacts such as increased GHG emissions.

As a result of this it is proposed that models that use available flow data in addition to electricity consumption and asset information be derived to calculate the drainage area electricity consumption based on hydraulic loading in the network. This can be used to build a picture of how the drainage area is operating in terms of its electricity consumption in its current configuration and the potential impacts of future development on the electricity being used by the utility. The social requirements of any future development can then be more evenly balanced with the economic cost and environmental impacts of additional pumping requirements at a planning stage. This drainage area electricity model could then be used in addition to other models, such as the non-linear reservoir model presented in this research, to build a tool kit for WWTP operators. Such a tool kit will allow the operator model and predict electricity consumption in the drainage area to assess, for example, the impacts of increased rainfall in a drainage area due to climate change on electricity consumption. It would also allow water utilities more accurately predict their electricity consumption in drainage areas based off rainfall forecasts. This is of interest to water utilities in the context of bulk purchasing of electricity: where more accurate predictions can be made of expected consumption, greater savings can be made for the utility when purchasing.

There are also opportunities for improving utilisation of existing infrastructure using these models. By providing plant operators with information ahead of time regarding increased hydraulic loading due to rainfall they can extend the time horizons over which they can make necessary changes to prepare WWTPs for greater flowrates of more dilute wastewater. The drainage network itself could also be used more efficiently and control strategies could be developed. By knowing in advance where in a drainage area increased hydraulic loading can be expected due to rainfall, the drainage system can be controlled to prioritise the removal of these excess flows to prevent flooding. Alternatively these more dilute flows due to rainfall could be mixed with more concentrated wastewater from other portions of the network to minimise changes to influent quality and the consequent changes to plant efficiency.

Determination of Treatment Base Flows

The median of all flows for 2016 at Plant B was calculated and was found to be 20,755 m^3d^{-1}. A subset of the flows was then classified into DWFs, based on the rainfall in the catchment area as previously described. The median of this smaller data set was found to be 15,912 m^3d^{-1}. This reduction in the flows on DWDs is to be expected; as it excludes the flows to treatment that include infiltration and inflows due to rainfall in the drainage area that will result in a higher median value.

Next these two values were used to estimate the PE in the drainage area based on a value of 200 L/PE/d, which gave values of 103,775 PE and 79,559 PE respectively. The most recent data from NI Water gave the actual population equivalent treated as being ~77,500 PE. It is immediately apparent that the calculated value based on the dry weather characterisation is more accurate in this case, a difference of ~2,500 PE as opposed to almost 6,000 PE when using the median whole data set. The reason for this is most likely due to the flows to treatment due to rainfall events in the drainage area bringing the median of the flows higher, which in turn gives a higher estimation of PE. The further discrepancy between the dry weather day characterisation and known values could be explained by the water usage pattern for this drainage area not matching the figure of 200 L/PE/d used.

To further check the data the actual water treated per PE for this drainage area was calculated for both data sets, giving values of 268 L/PE/d based on the full data and 206 L/PE/d for the dry weather values. Both values are greater than the 145 L/PE/d of water use that NI Water claim, but this does not allow for the fact that greater volumes of water inevitably find their way into the drainage system even when allowing for rainfall. In some cases there are ground water tables and tidal patterns that can result in excess water leaking into drainage systems that did not come from the distribution network. This accounts for the increase in usage in both these figures, but once again the dry weather characterisation has been shown to be closer to the actual. It is also in line with the typical standard of 200 L/PE/d used by the industry.

For these reasons it was decided that the dry weather characterisation process was a more accurate indicator of flows to treatment that do not include the impacts of rainfall in the drainage area. While this method does introduce an added data processing step, this analysis shows it is a more appropriate indicator of the base flow to treatment for use in following sections.

Using the Non-Linear Reservoir Model as a predictive tool

Phase One

The areas that were calculated for the drainage area being investigated in this analysis during stage one are shown in Table 10 and a sample of the flow data that was collected is shown in Table 9. An example of the DWD classification process and the resulting DWFs is shown in Table 14.

Table 14: DWD Characterisation & DWF Results

Plant B							
			DWFs				
Date	**Rainfall (mm)**	**DWD**	**SDAC 3**	**SDAC 2**	**SDAC 1**		**Total Inlet Flow**
…	…	…	…	…	…		…
…	…	…	…	…	…		…
…	…	…	…	…	…		…
11/03/2016	1.8	-	-	-	-		-
12/03/2016	5.6	-	-	-	-		-
13/03/2016	0	-	-	-	-		-
14/03/2016	0	-	-	-	-		-
15/03/2016	0	-	-	-	-		-
16/03/2016	0	-	-	-	-		-
17/03/2016	0	DRY	3,566	4,083	13,354		24,395
18/03/2016	0	DRY	3,549	4,136	12,777		22,342
19/03/2016	0	DRY	3,392	3,922	12,710		21,313
…	…	…	…	…	…		…
…	…	…	…	…	…		…
…	…	…	…	…	…		…

Table 15 shows the calculated median flows based on the DWFs for each of the terminal pumping stations as well the total inlet flow (TIF).

Table 15: Median DWF Results

	TPS Outlet Flow				**Plant Inlet Flow**
	SDAC 3	**SDAC 2**	**SDAC1**		**Total Inlet Flow**
	(m^3)	(m^3)	(m^3)		(m^3)
Annual Median DWFs	2,383	3,044	10,030		15,616

Once the base flows had been determined the excess flows for the three outlets and three inlets were calculated, with a sample shown in Table 16.

Table 16: Calculated Excess Flows

	TPS Outlet Flow				Plant Inlet Flow
	SDAC 3	SDAC 2	SDAC1		Total Inlet Flow
	(m^3)	(m^3)	(m^3)		(m^3)
01/01/2016	7,436	6,863	17,864		32,201
02/01/2016	4,482	3,640	17,265		27,772
03/01/2016	4,208	2,886	13,837		22,549
04/01/2016	4,043	2,853	12,586		22,305
05/01/2016	7,149	6,874	15,188		31,465
06/01/2016	4,509	4,447	16,213		28,026
…	…	…	…		…
…	…	…	…		…
…	…	…	…		…

The drainage area response factor (K) for each of the SDACs were calculated based on the flows through each of the three TPSs, while an overall K value was independently calculated for the entire catchment based on the total inlet flow. Shown in Table 17 are the average; median; and interquartile K values.

Table 17: Drainage Area Response Factor Results

Annual Median DWF Base Flow						
TPS Outlet Flows						
	Count	Average K		First Quartile	Median K	Third Quartile
SDAC 3	51	-0.066		-0.180	0.054	0.152
SDAC 2	47	0.174		-0.320	-0.005	0.267
SDAC 1	47	0.257		0.002	0.183	0.591
Plant B Inlet Flows						
Total Inlet Flow	48	0.233		-0.028	0.142	0.348

As can be seen in Table 17 some drainage areas produced negative K values in their summary statistics. Negative K values produce undesirable effects when used in the model, i.e. they result in an increasing flow when used in equation (13). This means that if a negative K is used the model will increase flows over time, even without rainfall events to supply the water, which is an undesirable effect. These negative values were not excluded when calculating the mean, median and interquartile ranges shown in Table 17 however. The reason for this is because it was thought that the median value of these data points would be the closest to the "actual" K of the drainage areas. Initial indications were that this postulation was correct, when it was found that all but two cases the average and median values were positive numbers for the annual median flows. In particular, the median K value for SDAC 2 was a negative number and, as has been highlighted, this is problematic when using equation (13). For this reason the calculations for SDAC 2 used a K value of zero when solving, causing difficulties in later sections.

Phase Two

The measured rainfall as well as the areas of each of the Sub-catchments (SC) are used to calculate the volume of rainfall that fell on each of the SCs and consequently each of the three SDACs. A sample of some of these calculations are shown in Table 18. The resulting rainfall excess (i) for each day in each SDAC is then used with the relevant median K, shown in the summary Table 17, and equation (13) to calculate the excess flows. These calculations were performed for each of the three SDACs and the total inlet flows and the results are shown in Table 19. The excess flows for each of the SDACs were then combined with the base flows to give a total predicted flow for each day and sample results of these calculations are shown in Table 20. The predicted flows shown in Table 20 were then compared with the measured flows for each of the TPS outlets and the total inlet flow. The errors were then calculated, a sample of which are shown in Table 21, are summarised in Table 22 and % errors based on the median annual flows are shown in Table 23.

Table 18: Rainfall Excess Results

	Rainfall (mm)		SDAC 1 (m^3)	SDAC 2 (m^3)	SDAC 3 (m^3)		SC 1 (m^3)	SC 2 (m^3)	SC 3 (m^3)	SC 4 (m^3)	SC 5 (m^3)	SC 6 (m^3)	SC 7 (m^3)	SC 8 (m^3)	…	SC 16 (m^3)
01/01/2016	-		-	-	-		-	-	-	-	-	-	-	-	…	-
02/01/2016	1.20		15,538	3,382	4,111		1,211	839	2,045	441	9,040	164	1,081	361	…	356
03/01/2016	0.90		11,654	2,537	3,083		908	630	1,534	330	6,780	123	810	271	…	267
04/01/2016	8.30		107,472	23,396	28,436		8,378	5,806	14,143	3,047	62,530	1,133	7,474	2,498	…	2,463
05/01/2016	8.10		104,882	22,832	27,750		8,176	5,666	13,802	2,974	61,023	1,106	7,294	2,438	…	2,404
06/01/2016	3.30		42,730	9,302	11,306		3,331	2,308	5,623	1,212	24,861	450	2,971	993	…	979
…	…		…	…	…		…	…	…	…	…	…	…	…	…	…
…	…		…	…	…		…	…	…	…	…	…	…	…	…	…
…	…		…	…	…		…	…	…	…	…	…	…	…	…	…

Table 19: Predicted Excess Flows

	SDAC 1					SDAC 3					SDAC 2			
	Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5
01/01/2016	14,881	14,991	14,433	29,971		7,046	6,855	6,628	7,507		6,863	6,863	6,863	6,863
02/01/2016	16,976	16,087	31,349	43,629		4,424	4,325	5,324	6,242		3,640	3,640	3,640	3,640
03/01/2016	13,473	29,170	41,814	41,967		4,120	5,130	6,058	6,227		2,886	2,886	2,886	2,886
04/01/2016	28,432	41,199	41,454	65,886		5,056	5,988	6,161	7,980		2,853	2,853	2,853	2,853
05/01/2016	30,167	32,265	58,231	48,507		7,971	8,040	9,760	9,249		6,874	6,874	6,874	6,874
06/01/2016	20,641	48,549	40,441	34,336		4,760	6,652	6,303	6,017		4,447	4,447	4,447	4,447
…	…	…	…	…		…	…	…	…		…	…	…	…
…	…	…	…	…		…	…	…	…		…	…	…	…
…	…	…	…	…		…	…	…	…		…	…	…	…

Table 20: Predicted Flows

	SDAC 1					SDAC 3					SDAC 2			
	Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5
	(m^3)	(m^3)	(m^3)	(m^3)		(m^3)	(m^3)	(m^3)	(m^3)					
01/01/2016	24,910	25,020	24,463	40,000		9,429	9,237	9,011	9,889		9,906	9,906	9,906	9,906
02/01/2016	27,006	26,117	41,378	53,658		6,807	6,708	7,707	8,624		6,683	6,683	6,683	6,683
03/01/2016	23,502	39,200	51,844	51,997		6,503	7,513	8,440	8,610		5,929	5,929	5,929	5,929
04/01/2016	38,461	51,228	51,484	75,916		7,439	8,370	8,544	10,362		5,896	5,896	5,896	5,896
05/01/2016	40,197	42,295	68,261	58,536		10,353	10,423	12,143	11,631		9,918	9,918	9,918	9,918
06/01/2016	30,671	58,578	50,471	44,366		7,142	9,034	8,686	8,400		7,491	7,491	7,491	7,491
…	…	…	…	…		…	…	…	…		…	…	…	…
…	…	…	…	…		…	…	…	…		…	…	…	…
…	…	…	…	…		…	…	…	…		…	…	…	…

Table 21: Errors in Predicted Flows

	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5
	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)		(m^3)	(m^3)	(m^3)	(m^3)
01/01/2016	-2,384	1,153	1,848	14,782	2,565	2,647	2,586	358	3,223	3,977	4,010	-12		3,214	6,620	24,108	29,906
02/01/2016	3,139	3,502	16,160	27,415	216	283	-1,824	1,733	754	787	-3,235	-807		3,832	21,689	27,299	43,552
03/01/2016	887	13,982	25,601	25,331	78	-2,019	1,549	2,626	33	-3,989	-1,562	-237		18,365	24,416	28,843	-
04/01/2016	13,243	24,986	24,818	49,189	-2,092	1,479	2,559	-627	-4,022	-1,595	-271	-6,266		8,484	15,023	-	71,039
05/01/2016	13,954	15,629	41,534	31,935	3,462	4,438	1,153	5,501	2,427	3,751	-2,244	3,645		7,663	-	28,624	25,091
06/01/2016	4,005	31,852	23,870	19,567	1,158	-1,955	2,555	-185	1,324	-4,671	1,218	-1,040		-	22,857	20,852	16,069
…	…	…	…	…	…	…	…	…	…	…	…	…		…	…	…	…
…	…	…	…	…	…	…	…	…	…	…	…	…		…	…	…	…
…	…	…	…	…	…	…	…	…	…	…	…	…		…	…	…	…

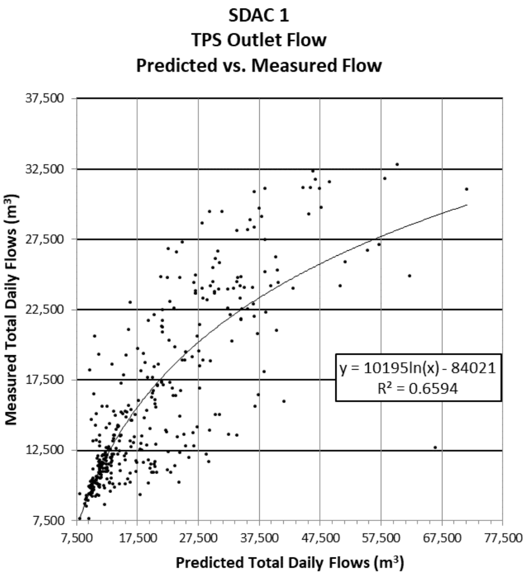
Table 22: Summary of Error Calculations

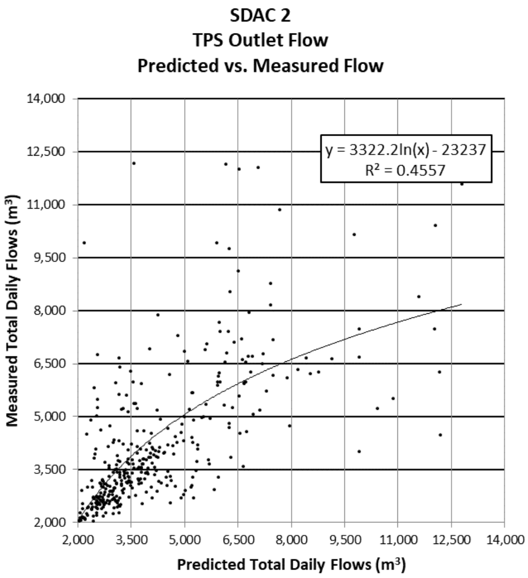
	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5
	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)	(m^3)		(m^3)	(m^3)	(m^3)	(m^3)
First Quartile	83	1,098	2,417	4,522	-85	43	166	273	-392	-598	-718	-638		512	2,641	4,873	4,485
Median	1,862	6,192	9,301	11,722	125	341	611	797	62	55	69	84		3,199	7,606	11,847	13,217
Third Quartile	7,702	13,203	17,790	21,753	708	1,217	1,663	2,074	522	657	623	725		8,854	14,841	20,591	26,974

Table 23: Summary of % Errors

	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5	Day 2	Day 3	Day 4	Day 5		Day 2	Day 3	Day 4	Day 5
	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)		(%)	(%)	(%)	(%)
First Quartile	1%	10%	20%	37%	-3%	1%	6%	11%	-10%	-15%	-16%	-16%		3%	15%	25%	23%
Median	13%	41%	64%	75%	4%	12%	20%	26%	2%	2%	2%	3%		14%	32%	50%	59%
Third Quartile	44%	81%	104%	130%	21%	37%	49%	63%	15%	19%	21%	23%		36%	56%	80%	103%

Once the predicted flows had been calculated, these values were then compared against the measured flows on the same day for each of the terminal pumping stations and the resulting graphs are shown in Figure 32.





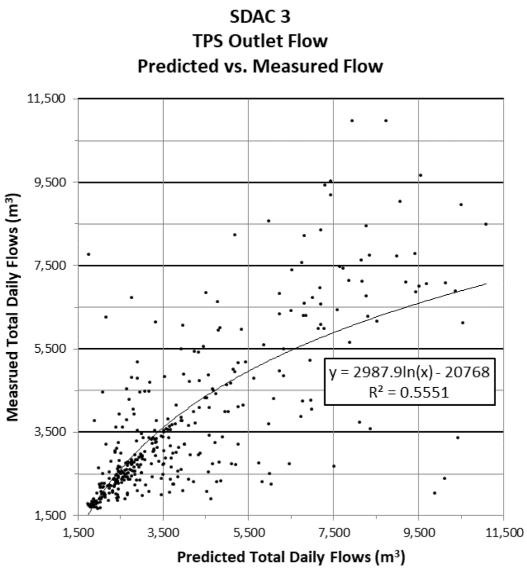


Figure 32: TPS Predicted vs. Measured Flow Graphs

A logarithmic trend line of reasonable significance is shown in the graphs in Figure 32. In addition to the significance of the trend line, it is also assumed that these errors will behave asymptotically, tending toward an upper limit on the y-axis. This is because the model directly relates rainfall to volumetric flows and assumes all of this flow arrives at the headworks of the plant, which is not always the case. In reality as rainfall increases excess volumes beyond the capacity of the network or plant are discharged through CSOs. This is suggested as a possible reason for the tendency of the model to overestimate the flows as can be seen in the error tables.

A graph similar to those shown in Figure 32 was produced for the measured and predicted flows at the headworks of the treatment plant, but more information was available for this point in the network. NI Water staff stated that the consent dry weather flow for the plant was 22,118 m^3, while the consent maximum flow for the treatment plant was 54,086 m^3, and both flows were included on the total inlet flow graph shown in Figure 31. Following this research it is proposed that these limits will need to be taken into account by any model of the hydraulic flows.

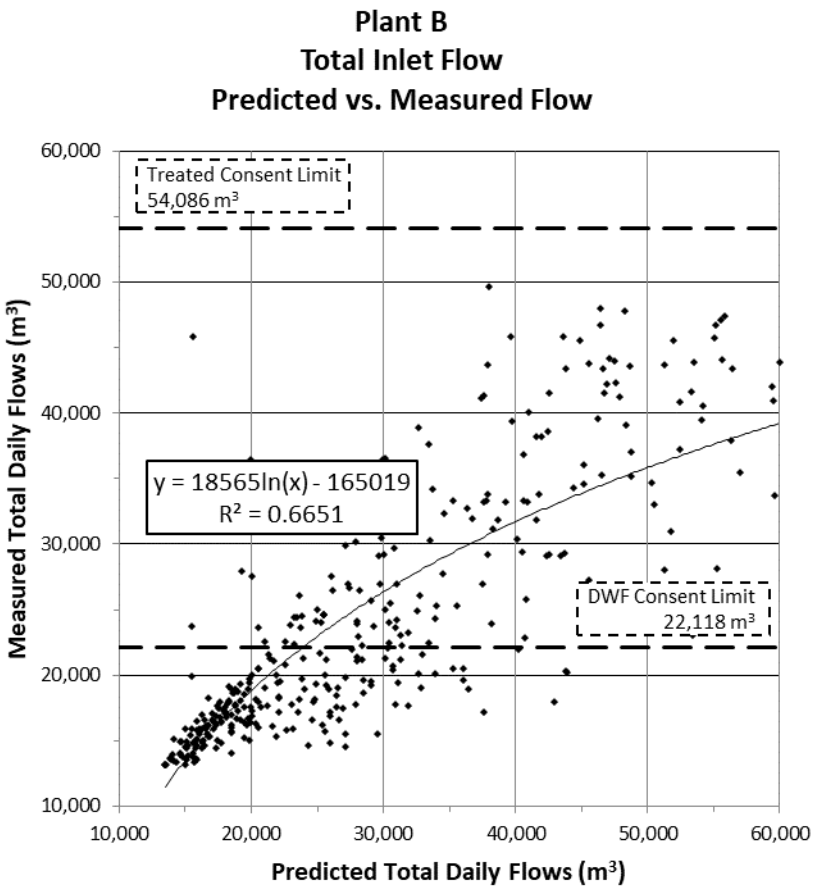


Figure 31: Total Inlet Flow Predicted vs. Measured Graph

The graph shown in Figure 31 was further developed by highlighting the predicted and measured flows on DWDs and comparing these in a separate graph shown in Figure 33. Further graphs were produced that compared the flows on days where 0.5 mm to 0.9 mm; 0.9 mm to 3.8 mm; and greater than 3.8 mm of rainfall and these are included in the appendices (Figure 34 to Figure 36). The selection of these rainfall ranges was based on the summary data for the rainfall in the catchment, with 0.9 mm being the first quartile of the daily rainfall measured in 2016, and 3.8 mm being the median. For each of these graphs, the linear trend lines that best fits the sub-set of data points were set to intercept the axes at the point (0,0), as it was assumed that flow at zero measured flow should be equal to zero predicted flow.

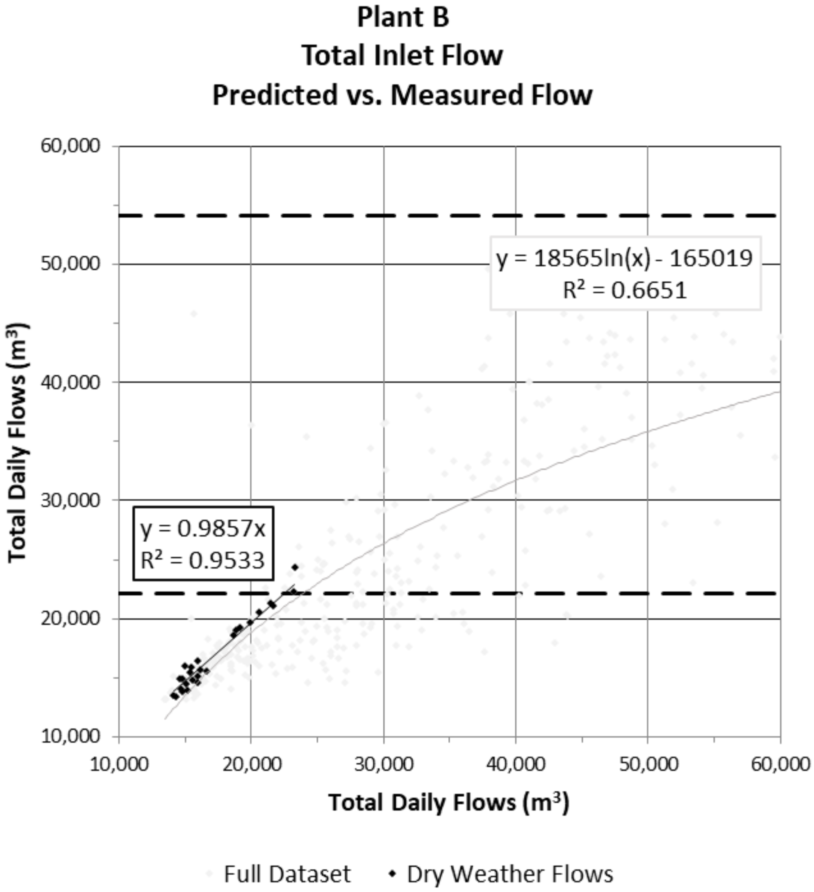


Figure 33: DWF Predicted vs. Measured Flows

The linear trend line for the dry weather flows shown in Figure 33 indicates a strong linear relationship between the model and actual flows. This to be expected however, as the model is predicting flows due to rainfall and so dry flows should be represented accurately by such a model. It also stands to reason that the flows on these days are less likely to be impacted by CSO events and other redirections of the flow away from the treatment plant.

Figure 34 through Figure 36, included in the appendices, indicate a breakdown in this linear relationship with increasing rainfall however, as the R^2 value for the highlighted rainfall scenarios decreases with increasing rainfall. This would imply that as the rainfall volumes increase, the reliability of the model begins to breakdown due to influences that are unaccounted for by the model, such as sewer overflows.

Flows through sewer overflows are rarely, if ever, measured by water utilities in Ireland and as a result accounting for the volumes of water lost from the system in this way is difficult. The calculations in shown in Table 24 through

Table 26 are intended to account in some way for these losses. Looking at Figure 31 the upper dashed line highlights the maximum permitted flow through the treatment plant (54,086 m^3). This means that every effort will be made to reduce the volume of water arriving at the plant during rainfall events to ensure that this limit is not breached. As has been stated previously the model does not allow for these losses from the system and it is hypothesised that a logarithmic relationship between the predicted and measured flows best describes this behaviour. By applying the logarithmic equation of that trend line it was expected that the model would produce more accurate results when this additional step is added.

Revised Predicted Flows

As per the equations in Table 12, the predicted flows were then revised based on the logarithmic trend that can be seen in Figure 31 & Figure 32. A sample of the results of these calculations are shown in Table 24 and the subsequent revised errors are shown in Table 25. Also note once more that, as no flow measurements were available at the headworks for the 7/1/2016, no error calculations could be performed. A summary of these revised errors is shown here in Table 26 and the % errors based on the annual median flow are shown in Table 27.

When comparing the summary of errors in Table 22 and Table 26 it can be seen that the revised table produces less variation than the model using the annual median DWF alone. Taking the Day 2 predictions for SDAC1 as an example the interquartile range, shown in Table 23 & Table 27, drops below zero using the revised method, indicating more underestimations of the flow. The key point of interest here however is the overall variation in the interquartile range: for the new method the difference between the first and third quartile is 3,106 m^3, compared to 7,619 m^3 for the original method. Based on the figures shown in Table 27 it can be said that just over half of the errors in modelled values one day in advance (Day 2) will be within ~20% of the measured flow for the drainage area under study. These errors are large, and increase over longer time horizons, but with further refinement it is expected that they can be brought into more suitable ranges for operational predictions.

Table 24: Revised Predicted Flows

	SDAC 1				SDAC 3				SDAC 2				Total Inlet Flows			
	Day 2	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**
	(m^3)	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**
01/01/2016	19,183	19,228	18,999	24,012	6,576	6,514	6,440	6,718	7,330	7,330	7,330	7,330	34,544	33,805	39,852	43,863
02/01/2016	20,007	19,666	24,357	27,007	5,602	5,559	5,973	6,309	6,023	6,023	6,023	6,023	32,612	39,114	43,224	46,174
03/01/2016	18,590	23,806	26,656	26,686	5,466	5,897	6,245	6,304	5,625	5,625	5,625	5,625	38,049	42,490	42,744	-
04/01/2016	23,612	26,534	26,585	30,544	5,868	6,220	6,281	6,858	5,607	5,607	5,607	5,607	37,809	38,817	-	51,614
05/01/2016	24,062	24,580	29,460	27,894	6,855	6,875	7,332	7,203	7,334	7,334	7,334	7,334	36,328	-	43,244	41,191
06/01/2016	21,304	27,901	26,382	25,068	5,746	6,448	6,331	6,231	6,402	6,402	6,402	6,402	-	41,748	39,971	38,278
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…	…	…	…	…	…	…	…	…	…	…	…	…	…	…	…	…
…	…	…	…	…	…	…	…	…	…	…	…	…	…	…	…	…

Table 25: Revised Flow Errors

	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**		**Day 2**	**Day 3**	**Day 4**	**Day 5**
	(m^3)	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**		**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**
01/01/2016	-8,111	-4,639	-3,617	-1,206	-289	-76	15	-2,813	647	1,401	1,434	-2,588		-8,844	-4,360	1,932	-3,218
02/01/2016	-3,860	-2,950	-861	764	-988	-867	-3,558	-582	94	127	-3,895	-1,468		-5,553	1,193	-3,858	2,533
03/01/2016	-4,025	-1,412	413	20	-959	-3,634	-647	320	-271	-4,293	-1,866	-542		128	-4,592	-897	-
04/01/2016	-1,606	291	-81	3,817	-3,664	-671	297	-4,131	-4,311	-1,884	-560	-6,556		-9,272	-4,824	-	5,775
05/01/2016	-2,181	-2,086	2,734	1,292	-36	891	-3,658	1,073	-157	1,167	-4,828	1,061		-7,313	-	-2,595	-383
06/01/2016	-5,362	1,174	-219	269	-238	-4,541	200	-2,354	235	-5,760	128	-2,129		-	-4,091	-1,603	-2,638
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…	…	…	…	…	…	…	…	…	…	…	…	…		…	…	…	…
…	…	…	…	…	…	…	…	…	…	…	…	…		…	…	…	…

Table 26: Summary of Revised Errors

	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**		**Day 2**	**Day 3**	**Day 4**	**Day 5**
	(m^3)	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**		**(m^3)**	**(m^3)**	**(m^3)**	**(m^3)**
First Quartile	-1,606	-602	-163	396	-292	-122	17	124	-271	-446	-660	-605		-1,905	-768	463	493
Median	64	1,056	2,660	3,573	99	275	442	593	280	291	291	330		-211	1,847	3,688	4,580
Third Quartile	1,500	4,050	5,751	6,676	437	853	1,153	1,449	671	811	899	896		3,294	5,862	8,235	9,619

Table 27: Summary of Revised % Errors

	SDAC 1				SDAC 3				SDAC 2					Total Inlet Flows			
	Day 2	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 2**	**Day 3**	**Day 4**	**Day 5**		**Day 2**	**Day 3**	**Day 4**	**Day 5**
	(%)	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**	**(%)**		**(%)**	**(%)**	**(%)**	**(%)**
First Quartile	-9%	-4%	-1%	3%	-9%	-4%	0%	4%	-7%	-11%	-14%	-13%		-10%	-4%	2%	2%
Median	1%	7%	17%	25%	8%	17%	25%	32%	8%	9%	10%	10%		4%	15%	24%	30%
Third Quartile	11%	30%	45%	54%	16%	31%	41%	57%	21%	27%	29%	30%		14%	28%	41%	48%

Conclusions

- It was shown that using average flows to treatment may be misleading when compared to using the median flow to treatment. This is because the flows to treatment experienced by a plant are not normally distributed. In the context of energy efficiency and benchmarking the decision to use the mean as opposed to the median at the design phase will have an impact on the utilisation factor of the plant. It is therefore recommended that any analysis considering the volumetric flows to a wastewater treatment facility should use the median value as opposed to the average.
- For the drainage areas of two wastewater treatment plants in Northern Ireland, there is a large amount of electricity usage in the wider drainage area currently going unaccounted for in many benchmarking and reporting methods. At one plant in this analysis there is almost double the electricity consumed per unit volume treated when a portion of the pumping in the drainage area is accounted for. The same increase was not seen at a second treatment plant in Northern Ireland, which would indicate that it is not always the case that this electricity consumption is always present. The scale of the increase in electricity consumption when considering the drainage area at Plant B confirms that it is a result of the siting of the drainage areas and pumping stations. Neglecting this in analyses of electricity consumption based on the lack of significant electricity consumption in other drainage areas is unwise and it is recommended that any standardised methods for the benchmarking of wastewater treatment should account for this electricity consumption.
- It was found that the median of the full flow datasets overestimated the base flows to the two wastewater treatment plants analysed in Northern Ireland. The dry weather flow to treatment was found to be a more accurate metric when compared with NI Water's own PE figures and was used to establish a base flow for the flow to treatment to two plants. While this is a more accurate measure of the base flows to treatment, it does require the collection of rainfall data to perform the categorisation process.
- It has been shown that the non-linear reservoir equation can be used to predict flows to treatment following rainfall events using only data already collected by NI Water and the UK Met Office for one drainage area in Northern Ireland. It has also been shown how one method of accounting for the model's overestimation of flows to treatment can reduce the overall error in predictions. There are still

significant errors in using the method, however it is expected that with further refinement this model can accurately predict the flows to treatment due to rainfall events in the drainage area.

Future Work

There are several areas for potential future work based on the work presented here. An analysis of the electricity consumption of the pumping stations in the wider drainage areas of wastewater treatment plants in Ireland is recommended. There may be several difficulties in relation to this analysis however as in researching this paper it was found that quite often the data required may not be recorded or is retained in paper records. There may also be issues in relation to the perceived commercial sensitivity of these records. Despite the hurdles, if the water sector is to truly understand the impacts it has on the environment, the potentially significant amounts of electricity currently going investigated in the drainage area should be considered. Planning and operational considerations can also produce cost saving potential if the impacts of the pumping of wastewater to treatment is also considered and it is suggested that an integrated and holistic modelling tool kit be developed that can aid in this.

Drainage Area Model

The primary focus of future work should be on the development of a drainage area model that can model the electricity requirement of the sewer network based on the hydraulic flows in the network. This model is expected to use the non-linear reservoir method outlined in this thesis to solve for the catchment area inflow and infiltration for a given rainfall event. This will require further refinement of the non-linear model to bring the errors to within 10% of median DWF for 90% of predicted values. There are several areas that can be considered to achieve this but three primary areas are to be addressed: flow outliers due to industrial discharges, refinement of the weather measurements used and the impacts of seasonal ground water variation on infiltration to the drainage network.

In analysing the flow data for Plant A & B there were several days where there were significantly higher flows to treatment without a corresponding rainfall event. After consulting with NI Water staff it was determined that these flows may be due to discharges coming from industrial sources in the drainage areas, e.g. days where plant or machinery is washed down. Working with industrial customers in the drainage area it could be expected that such flows could be accounted for, thereby reducing the uncertainty involved in what the sources of excess flows are in the drainage area.

It is also expected that refinement of the rainfall measurements used can yield greater precision in the predicted flows to treatment. The rainfall measurements presented in this thesis used point rainfall data available from the UK Met Office and assumed that the rainfall in the drainage area was uniform across the entire area. This is not the case however, particularly with widely spread out drainage areas and there is the potential that errors are introduced by using a point measurement of rainfall in one part of the drainage area when no rain fell on a significant part of the drainage area on the same day. Radar measurements of rainfall should give a more accurate picture of rainfall patterns across the entire drainage area and predicted rainfall intensities are typically presented in a similar manner to radar measurements. For this reason a method of converting rainfall radar data to rainfall over the drainage area is to be developed to replace the point measurement system used in this thesis. The inclusion of such a method should yield far more accurate results for the flows to treatment. It is intended to be validated by checking against the flows through pumping stations serving different parts of the drainage area.

It was noted in this thesis that there is a seasonal variation in the base flows to treatment, which is why a second method of calculating the base flows based on the seasonal DWF was investigated. A portion of this variation in the base flow is expected to be attributed to seasonal changes in the ground water table level in the drainage area being investigated. Groundwater levels can be impacted by a multitude of factors, including the intensity and frequency of rainfall events. By analysing and accounting for seasonal variations in the drainage area water table level in relation to the depth of the drainage network it is expected that a portion of this variability could be accounted for, improving the accuracy of the hydraulic model further.

Once this hydraulic model has been refined it is intended to then use the energy data from the pumping stations throughout the catchment areas to derive a model that can calculate the energy required for a given flow rate through the pumping station. This is to be derived from first principles, based on the hydraulic power equation shown here as equation (25).

$$\mathrm{P} = \frac{Q\rho g h}{\eta} \tag{25}$$

Where:
P is the Power (kW)
Q is the flowrate (m^3/s)
ϱ is the density of water (kg/m^3)
h is the head height (m)
η is the head height

One of the difficulties expected in this approach is expected to be the lack of currently available data to calibrate and validate the model. Many pumping stations throughout the network do not measure the flowrate through the station, although the electricity consumption is almost universally measured. In such instances it is intended to use the electricity data in conjunction with asset information, such as known pump power consumption and power factors, and the summation of downstream flows to calculate the flowrates through each pumping station. This is not without its own problems and an uncertainty analysis of the model will be needed to establish its validity.

The second avenue of research for such an integrated model is to develop a method for determining the potential changes in influent composition and dilution based on the results of the hydraulic analysis presented in this thesis. This information is important to the operation of the wastewater treatment plant and the electricity consumed. If a model can be derived that can inform operators of potential variations to influent concentration it can be used to model variable influent values into existing WWTP models. This will allow for more realistic modelling of treatment as well as the efficient operation of the plant and the development of RTC potential for the automated operation of the plant based on predicted and measured values. This model is a more complicated endeavour however as it will require high temporal resolution analysis of the changes to the influent based on dilution over long periods of time to calibrate and validate. Currently influent data is typically measured twice a month in Northern Ireland, which is not enough to perform the detailed analysis required and would therefore require a significant measurement project as a starting point.

The long-term goal of this modelling approach is to be able to use predictive rainfall data from weather models that will be able to predict the energy consumption throughout the network over a 5-day horizon. It is hoped that this will then incentivise utilities to invest in more measuring equipment, creating more accurate data and models, thereby refining the model further. Analysing the behaviour of WWTPs during events such as WWFs will allow for better planning and operation of treatment plants that would be in line with the predicted requirements for improved plant performance. Greater amounts of data and further analysis is necessary to confirm the decrease in plant performance that has been reported and would allow for a clearer picture of the efficiency during WWFs of the individual WWTPs observed in this study. This research will also go some way to increasing understanding of the energy/water nexus and other social and environmental impacts that are currently overlooked in the context of wastewater treatment.

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Appendices

Appendix A – Sample Calculation of K

Table 28: Sample of Drainage Area Response Factor (K) Calculations

	TPS Outlet Excess Flows							Inlet Excess Flows	
Date	SDAC 3	K	SDAC 2	K	SDAC 1	K		Inlet	K
	(m³)		(m³)		(m³)			(m³)	
13/03/2016	1,937	0.176	2,675	0.332	11,844	0.591		17,733	0.444
14/03/2016	1,624	0.153	1,918	0.418	6,562	0.224		11,371	0.232
15/03/2016	1,394	0.068	1,263	0.156	5,246	0.222		9,013	0.022
16/03/2016	1,303	0.097	1,080	0.038	4,203	0.234		8,816	0.004
17/03/2016	1,183	0.014	1,040	-0.049	3,325	0.191		8,780	0.266
18/03/2016	1,167	0.145	1,092	0.218	2,747	0.025		6,726	0.166
19/03/2016	1,009	0.033	879	-0.137	2,681	0.350		5,698	0.149
20/03/2016	977	0.031	1,007	0.087	1,889	0.382		4,910	0.179
21/03/2016	947	0.269	924	0.628	1,289	-0.266		4,105	0.108
22/03/2016	724	0.107	493	0.340	1,682	0.147		3,686	0.087
23/03/2016	651	-	351	-	1,453	-		3,379	-
19/04/2016	652	0.287	185	4.854	3,165	0.188		4,866	0.239
20/04/2016	489	0.061	1	-	2,621	0.126		3,831	0.198
21/04/2016	460	0.110	-53	-0.956	2,310	0.075		3,143	-0.111
22/04/2016	413	0.054	-138	2.893	2,143	0.004		3,511	0.148
23/04/2016	391	-	-8	-	2,135	-		3,029	-
13/05/2016	21	-1.629	-215	-	41	-		-185	-
14/05/2016	105	-1.026	8	-4.106	-41	-2.583		331	-0.956
15/05/2016	293	1.578	463	1.458	-544	-0.143		861	-
16/05/2016	60	-	108	-	-628	1.046		-81	-2.324
17/05/2016	-238	-	-555	-	-221	-		-824	-

Appendix B – Predicted vs. Measured Flows categorised by Rainfall

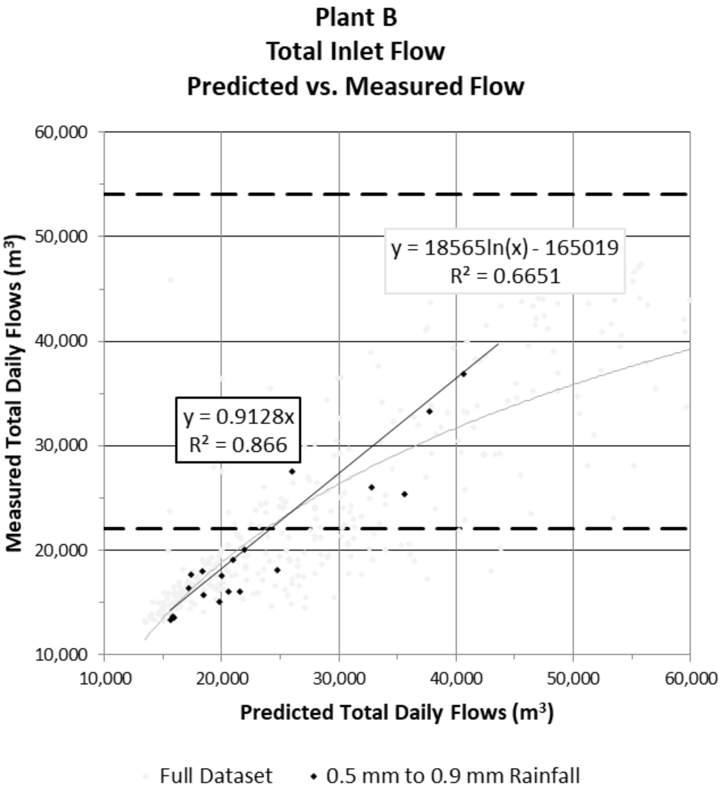

Figure 34: Predicted vs. Measured Flows for days with 0.5 to 0.9 mm Rain

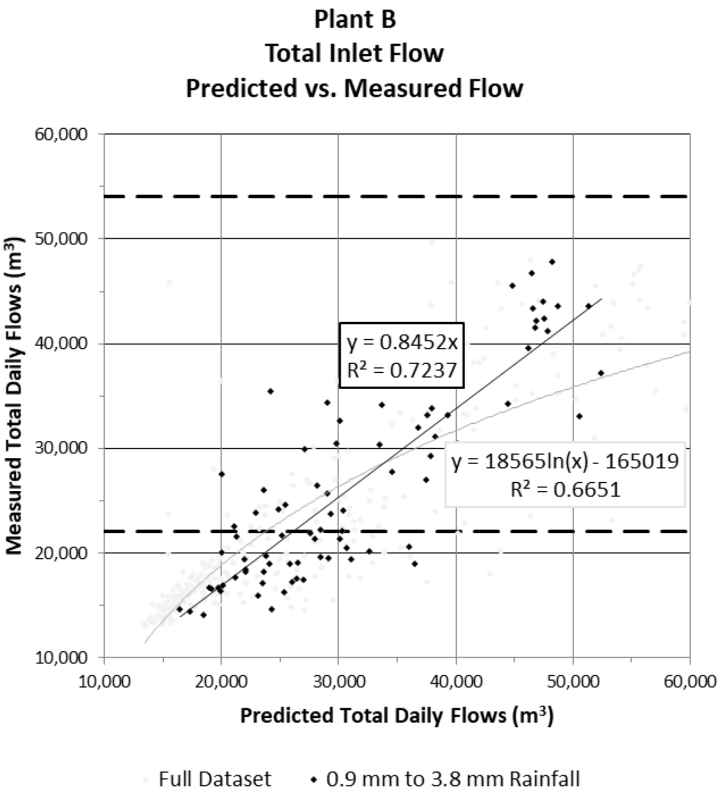

Figure 35: Predicted vs. Measured Flows for days with 0.9 to 3.8 mm Rain

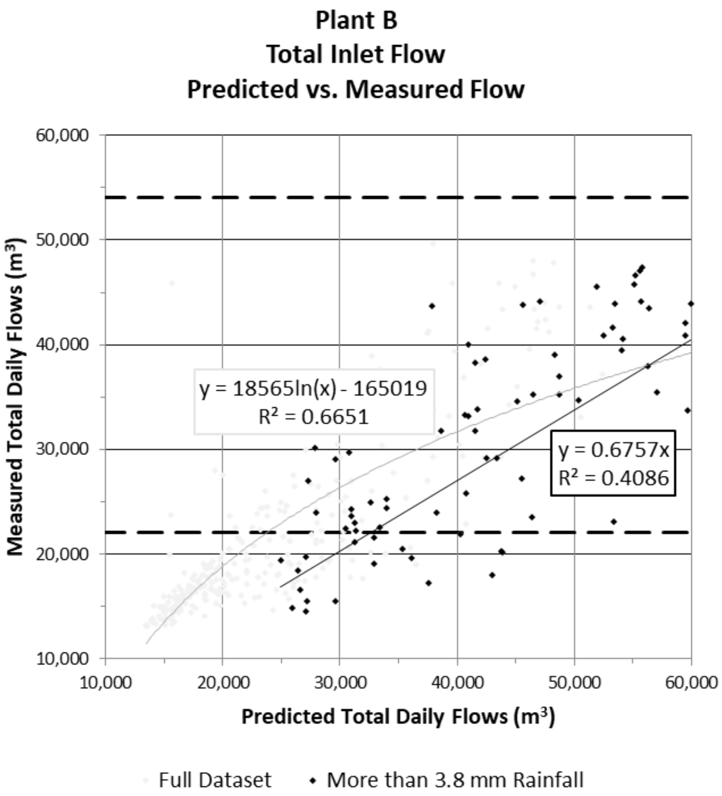

Figure 36: Predicted vs. Measured Flows for days with greater than 3.8 mm Rain