Performance Evaluation of the Aero-fac® Wastewater Treatment Process at Errol, Scotland

by

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ABSTRACT

Errol is a village in Scotland with a population of 1200 and no history of wastewater treatment. However, Errol was required to comply with the obligations outlined in the EC Urban Waste Water Treatment Directive (UWWTD) and provide appropriate treatment for their wastewater before 31 Dec 2005. Consequently the water service provider for Scotland, Scottish Water (previously North of Scotland Water Authority), started to search for appropriate solutions to the Errol wastewater discharges. The Aero-Fac® facultative system, an American designed biological system based on facultative lagoons has proved successful in many parts of the world because of the reduced sludge digestion, was chosen for Errol. The Aero-Fac system at Errol was constructed by Montgomery Watson Harza (MWH) and comprises two lagoons, control room, aerators, diffusers, pumping station and pipe line from Errol and to the estuary. The Errol Aero-Fac was the first to be built in the UK and so it was considered that an investigation of its performance was necessary to establish future design criteria. This thesis provides an evaluation of the performance of Errol Aero-Fac wastewater treatment system and based on an analysis of the performance data, describes how design of future plants might be improved. The hydraulic behaviour inside the lagoons was analysed by tracer study with rhodamine WT, which showed dispersion close to plug flow. The final effluent BOD and TSS complied both with SEPA and the performance guarantees consents at a 95 percentile and even though there was accelerated growth of algae in the summer period, this did not stop final effluent complying with the suspended solids consent. Bacteriology tests showed pathogen removal is comparable to activated sludge process and other similar systems. Ammonia removal was seasonal with a better performance in the summer months. The sewage received at Errol was weak and also contained a large non-biodegradable fraction, which prevented final effluent in many cases complying with the EC UWWTD COD consent requirements. Sludge was observed to accumulate in the lagoon but only sparingly and mainly around the inlet of the primary lagoon. The estimated per capita accumulation rate was only 0.0047m^3/person/year. The capital cost of the system is high compared to similar systems, but the operation cost of the system can compare favourably if business rates are not considered in the comparison.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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CHAPTER 1: INTRODUCTION

1.1 Population Growth, Water Pollution and Control

Historically low population densities and a prevailing rural economy have kept water pollution localised, preventing it from spilling over to the wider environment. With modest water consumption levels and no drains or sewers to collect and concentrate the wastewater and take it away, rivers and coastal zones remained comparatively free of human pollution. However, recent rapid growth in population has changed this situation. Between 1970 and 2000, the global population doubled from 3 billion to 6 billion people. Furthermore, the value of their combined economy grew tenfold. This human and economic growth is increasing the pollution load on the environment to such an extent that it can no longer cope. The result is damage to both the local environment and in many cases the wider aquatic environment. In some areas, damage has reached the level whereby it is threatening the existence of industries such as tourism and fishery, and in the long term the health and economy of current and future generations (UNEP/GPA, 2001).

In order to sustain economic prosperity, control of population growth and the environment is crucial. Responsibility for this control lies primarily with governments and legal authorities, as they are the ones who have authority to issue laws and regulations. In the UK, water companies or any business disposing effluent to water courses, rivers and seas are controlled by the Environment Agency (EA) in England and Wales and the Scottish Environmental Protection Agency (SEPA) in Scotland.

1.2 The EC Urban Wastewater Treatment Directive

The UK is a member of the European Union and thus the UK water companies must comply with the requirements of the EC Urban Wastewater Treatment Directive (UWWTD) (91/271/EEC) (Council of the European Communities, 1991) which sets minimum standards for the collection and treatment of effluent discharges. The UWWTD requires that all discharges from sewage treatment works with population equivalents (pe) of greater than 2000 to inlands waters and estuaries, and over 10000 pe to coastal waters, receive secondary treatment. For discharges less than 2000 pe the Directive requires “appropriate treatment” by 31 December 2005 (Council of the European Communities, 1991).

1.3 Complying with the EC UWWTD in the UK

When the UWWT Directive came out in 1991, many believed that water companies in the UK would face great difficulties complying with it. The real problem seen was complying with the part of the Directive which considers the discharges of populations less than 2000. This is because most of the areas with populations greater than 2000 were already provided with secondary treatment, while for most of the areas that are populated with less than 2000 (rural areas, settlements and villages) are provided with inappropriate treatment or in many cases no treatment whatsoever. Furthermore, it is more expensive per head to provide water and wastewater treatment to areas that are less-densely populated. Added to this is that water companies are pressured anyway by competition and regulations to reduce their charging prices (NoSWA, 2001).

1.4 The Option of Facultative Lagoons to comply with the EC UWWTD

In order to comply with the part of the EC UWWTD that considers discharges of less than 2000 (pe), one of the options available is to consider a complete biological process through the use of lagoons. This treatment option is well established throughout the world although its adoption in the UK has been poor. Lagoons offer a number of advantages such as simple plant construction, low construction and operating costs and ease of plant operation, but on the other hand there are many drawbacks such as odour, large land consumption and poorer effluent quality. One particular type of these lagoon systems, which is receiving more attention now is the facultative lagoon; because of it is ability to digest sludge when conditions inside the lagoon are right. So, this type of lagoon would provide an extra advantage to water companies in the UK if it was implemented. However, facultative lagoons have not been used in the UK apart from trial use and so it will be difficult to predict whether they will be a success or failure. Furthermore, facultative lagoons are known to be more efficient in sunny and warm places rather than colder places, and in a UK winter, would give doubts as to the ability of facultative lagoons to function properly. Additionally, the only detailed reference available on facultative lagoons use in the UK is the work by Abis (2002) on pilot-scale facultative ponds in Yorkshire which was documented in her PhD thesis and other published papers.

1.5 Scottish Water (Previously NoSWA) and the Errol Problem

Scottish Water is one of the water companies most affected by the UWWT Directive, because 94% of the population live in communities of less than 2000 people (Abis, 2002). One of the areas they were required to provide a proper treatment for by December 2005 is a village called Errol, around 10 miles from the outskirt of Dundee and with a population of 1200 people with no history of wastewater treatment. Scottish Water started the search for an appropriate solution in 1999 with environmental studies, site selection, an examination of a range of technical options, liaison with regulatory authorities and planning approvals.

1.6 Aero-Fac System as a Solution for Errol

The Aero-Fac process is a biological system that exploits the reactions that take place under facultative conditions to come up with a new system that extend the advantages of facultative lagoons and overcomes the problems accompanied with them. The system was developed by Lake Aid Systems (LAS) limited, an American company that has 20 years experience in wastewater research (NoSWA, 2001).

The aero-Fac system was chosen for Errol after NoSWA became aware of the claimed advantages of the process such as lower operating costs, removal efficiency and robustness, over other treatment options such as waste stabilisation ponds and activated sludge (NoSWA). The Errol Aero-Fac system was constructed by Montgomery Watson Harza under a new joint marketing offer with LAS named “One company, One contract, One price” concept. They took 10 month to construct the system which consisted of two lagoons, control room, aerators, diffusers, pumping station and pipe line from Errol and to the estuary. The Aero-Fac system will be discussed in more detail in Chapter 2.

1.7 Effectiveness of the Errol Aero-Fac system

Due to the fact that, the Aero-Fac system is a UK first there was a need to investigate the effectiveness of the system and thus NoSWA agreed with MWH to undertake a series of performance tests in addition to a one year investigation to be carried out by a third party. The University of Leeds was chosen as this third party because of its experience in researching lagoon systems.

1.8 Thesis organization

This thesis aims to evaluate the performance of the Aero-Fac wastewater treatment process at Errol, Scotland. It is arranged as follows.

Chapter 2 is a review of literature that is divided to 6 sections. The first section covers the features of small populations. The second covers the characteristics of wastewater production from small populations. The third covers treatment systems for small populations. The fourth section covers the implications of EC UWWTD. The fifth section covers the changes within Scottish water. The sixth section covers the Aero-Fac process.

In Chapter 3, the aims and objectives of this project are presented along with the materials and methods used to achieve these objectives.

Chapter 4 presents the results of the analyses carried out during this project. These include effluent quality results, sludge depth, faecal indicator removal and the tracing study results.

Chapter 5 covers the discussion of results presented in Chapter 4.

The thesis ends with conclusions of the main findings of this project.

CHAPTER 2: LITERATURE REVIEW

2.1 Features of Small Populations

2.1.1 Urban and rural populations

The definition of an urban community is different throughout the world. For instance in Sweden and Denmark, a village of 200 people is counted as an urban population, whereas below this it is referred to as rural. By contrast in Japan, a city has to have a population of 130,000 before it is termed urban. In most other countries, the definition of an urban population falls between these two figures. For example, Australia and Canada use 1000, Israel and France 2000, and the United States and Mexico refer to a town of 2500 residents as urban (About, 1997).

In this thesis a village of 2000 people will be regarded as urban whereas anything below that will be referred to as a small rural community. This approach has been taken simply because the EC Urban Waste Water Treatment Directive uses a population equivalent of 2000 as the cut-off point, below which 'appropriate' treatment of their wastewater is required and above that more sophisticated treatment options must be installed.

2.1.2 Social features of small communities

(a) Social and recreational activities

Entertainment in the rural areas is usually linked with nature as a result of the green countryside, friendly neighbourhoods, pristine lakes, streams and rivers. People also visit to undertake activities such as hiking, barbeques and fishing. The natural environment in the countryside becomes an attraction to people living in cities as it provides something they lack. Day trip tourism is now an essential income generator in Britain and worth about 9 billion pounds annually (Reeves, 2004). Residents of rural areas usually commute to nearby big cities or towns for nightlife entertainment, shopping, restaurants and cinemas, though with the growth of tourism in the rural areas and the new diversification in the type of people living in the countryside; many restaurants and bars are being opened in rural areas and popular shops are opening branches there.

(b) Dominant groups

In the UK, 14 million people live in rural areas. Historically, the dominant groups living in rural areas were farmers and their families and old and retired people and their families. However more recently there has been an urban to rural migration caused by the many problems in urban cities, such as pollution, noise, crime and congestion. Furthermore, there are a number of people who choose to commute everyday to work in a city while living in rural area. The tourist industry in rural communities is growing because of the many natural attractions which are not available in large cities. This tourism growth means that at seasonal periods there are tourists living in rural areas in addition to people working in the tourism industry.

The spread of BSE and Foot and Mouth disease had a devastating effect on the farming industry and led to many farmers losing their livelihoods. Now, out of the 14 million living in rural areas in the UK, only 174,000 are full-time farmers (0.3% of the population), and so people moving to the countryside will have a bigger chance of meeting a merchant banker, a tourist or a doctor than meeting a farmer (Reeves, 2004).

2.1.3 The economy of small communities

New technologies coupled with globalisation of labour and the economy are changing where and how people work and this is leading to new applications in resource extraction industries as well as growth in service occupations in small communities. This has helped to diversify many rural economies. This is better for the rural economy as it reduces the risk of economic crisis if farming business fails (as discussed earlier with reference to BSE and Foot and Mouth). However diversification in rural economies is largely a phenomenon associated with developed countries and still in most developing countries the economy of small communities is linked closely to natural resources, soils and water for crop and livestock production, hard rock minerals, coal, oil and natural gas extraction and forested land for timber (Sandoval, 2001).

Usually, more attention is given to the ideals of rural-living which are valued by the small community resident as well as the city dwellers who migrate to the countryside. However there are many challenges facing these small communities in order to maintain their quality of life. In the UK, prevention of soil erosion, flooding and eutrophication was estimated to cost as much as £1.5 billion (Reeves, 2004). Another issue is the lack of adequate facilities for the proper collection, treatment and disposal of wastewater and this is a serious issue as it is essential to protect the environment and the health of the public. Wastewater continues to affect a user's life even after it disappears down the drain. This is because wastewater generated by homes, farms, businesses and factories, eventually returns to the environment to be used again, so when wastewater receives inadequate or no treatment; the overall quality of the world's water supply suffers. Untreated wastewater is still the root cause of much environmental damage, human illness, misery and death around the world.

2.2 Characteristics of Wastewater Production from Small Communities

2.2.1 Sources of Wastewater in Small Communities

Sources of wastewater from small communities include homes, farms, hospitals and businesses. Some communities have combined sewers that collect both wastewater and storm water runoff from paved areas such as streets and car parks. Others (a minority in Scotland) have a separate sewer system whereby drainage is conveyed separately to watercourse and bypasses the sewage treatment process.

(a) Characteristics of the domestic wastewater in small communities

Wastewater from a typical household in a small community might include toilet wastes, used water from sinks, baths, showers, washing machines and dishwashers and anything else that can be put down the drain or flushed down the toilets. This wastewater is usually similar to the wastewater released from service businesses and hospitals (usually referred to as domestic wastewater) in the sense of the main type of pollutants released. The main pollution released from residential wastewater (or even domestic wastewater in general) is organic material (measured as BOD and COD), nutrients (nitrogen and phosphorus), suspended solids and faecal microrganisms.

(b) Characteristics of agricultural wastewater from farms

Wastewater released from farms might include animal wastes from meat packing and processing facilities as well as drainage. The associated animal wastes might include pathogens that if transmitted could cause the spread of many diseases. Farm wastewater includes inorganic pollutants such as pesticides, insecticides, herbicides and fertilisers. These pollutants contain nitrogen or phosphorus or both and these result in algal blooms or eutrophication in water and pollution of groundwater if these minerals leach to the underground (Table 2.1, Horan, 2000).

(c) Characteristics of storm water runoff from streets and other land areas

In communities where there are combined sewers that collect both wastewater and storm runoff from streets, farms and other land areas, the wastewater can include any debris from street and waste oils, pesticides, insecticides, herbicides, fertilisers and wastes from humans and animals.

The debris from urban runoff contains waste oils and toxins which persist in the environment and can cause long-term damage. Pesticides, insecticides, herbicides are all detected in urban runoff whereas farm runoffs contain fertilisers comprising nitrogen and phosphorus or both and these can cause eutrophication of rivers and groundwater pollution. Animal and human wastes could carry pathogens that cause many harmful diseases.

2.2.2 Environmental legislation to protect the environment from wastewater discharge

Rural communities usually lack adequate facilities for the proper collection, treatment, and disposal of wastewater. However, international, national and regional requirements are increasingly becoming more stringent to improve water and wastewater quality. This has left small communities to face a unique situation, as they must weigh the costs of necessary capital investments to meet all the environmental and health goals (Sandoval, 2001).

Table 2.1. The atmospheric deposition of total nitrogen and total phosphorus in watersheds grouped according to dominant land use (Horan, 2000)

Dominant land use	Total nitrogen (kg/ha.yr)	Total phosphorus (kg/ha.yr)
Coastal	5.8	0.31
Urban	7.2	0.48
Rural (non-agricultural)	6.2	0.27
Rural (agricultural)	8.8	0.66

All the member states of the European Union must comply with the requirements of the EC *Urban Waste Water Treatment Directive* (91/271/EEC). This Directive sets minimum standards for the collection and treatment of sewage and effluent discharges. It requires all communities of less than 2,000 population equivalent (based on an excreted BOD of 60 g/d) to have appropriate treatment by 31 December 2005. In the US, small communities need to comply with the requirements of the Safe Drinking Water Act (SDWA) and the Clean Water Act (CWA), both give regard to wastewater treatment.

2.2.3 Challenges facing small communities in order to comply with legislation

Usually, small communities lack the necessary financial resources, capacity, structure, access to technology and the right tools in their community to make informed and rational decisions. So, when it comes to the problem of lack of wastewater treatment in many rural areas; many small communities lack adequate financing, management skills and training to construct, operate, manage and maintain wastewater treatment facilities or systems (Sandoval, 2001).

2.2.4 Sections 2.1 & 2.2: conclusions

There is a noticeable urban-rural migration in many developed countries. City dwellers are moving to small towns and rural communities in search of a better quality of life. However, the challenges which rural communities in Western Europe and the US face, including the proper collection, treatment and disposal of wastewater, are not being headed. Wastewater from small communities could include: pathogenic bacteria, fertilisers, herbicides, insecticides and insecticides, inorganic pollutants, which could cause eutrophication and ground water pollution; organic wastes which would be decomposed by bacteria in water and thus deplete the oxygen level and sometimes toxins which could cause long-term damage to the environment. Water legislation puts pressure on small communities to solve the problem of wastewater, but small communities lack adequate financing, management skills and training to challenge their wastewater discharge problem.

2.3 Treatment Systems for Small Populations

2.3.1 Introduction

Within a country or city, there is likely to be wide variety of wastewater treatment needs. In a village or a settlement, the number of people is much less than the number within the city and usually the main activity is farming and recently services whereas in the city the major activities are industrial and commercial. As mentioned earlier, wastewater discharges from a city may potentially pollute water sources more than discharges from small communities because of the larger number of people living there and the associated industrial activities usually happening. However, usually small communities are not provided with wastewater treatment systems while most cities are provided with appropriate treatment systems. Consequently small communities can often contribute more towards polluting a watercourse than a city. The reason why small communities are not usually provided with wastewater treatment systems is that small treatment systems compared with large treatment systems, are subject to operation and maintenance problems and large *per capita* costs. Furthermore, small sewage treatment systems are usually subjected to varied flows and augmented by large point sources. In spite of this, especially with the implementation of the EC UWWT Directive, which requires all communities of less than 2000 people to have appropriate treatment for their wastewater discharges before 31 Dec 2005, all small communities now need to have appropriate systems for their wastewater discharges. Whereas the term appropriate is defined by the Directive as treatment of urban wastewater by any process and or disposal system which after discharge allows the receiving waters to meet the relevant quality objectives and the relevant provisions of this and other community Directives.

2.3.2 Different options of sewage treatment for small populations

Sewage treatment is a large industry and many new technologies are being developed and marketed vigorously, principally by the Water Companies. There are specific needs for wastewater treatment in small communities and there are several process options to consider. These include lagoon systems, activated sludge (in particular oxidation ditches), septic tanks, trickling filters and reed beds. In the following text some of these options will be presented in detail along with advantages and disadvantages.

(a) Lagoon systems

The term Lagoon refers to a diverse array of suspended growth biochemical operations with the common characteristics that they do not include down stream clarifiers and associated settled solid recycle. The term lagoon itself was originated from the technique historically used to construct them, as in-ground, earthen basins that resemble shallow ponds (Grady, 1999).

The use of lagoons can be traced back over 3,000 years, however the first recorded formal use in wastewater treatment goes back to the 1920's in the states of California, North Dakota and Texas (Middlebrooks *et al*, 1978). The first lagoon in Europe followed more recently, as a tertiary wastewater treatment for the city of Muchen (Mara and Pearson, 1987). Since then the use of lagoons has spread across the world because of their simplicity and low cost and in 1964, a survey conducted by the WHO showed that lagoons were used in 39 countries (cited in Mara and Pearson, 1987). Furthermore, another survey conducted in the 70's in the United States estimated the number of lagoons in the US to be 5,000. Now, lagoons are used extensively in almost every country across the globe.

Figure 2.1. Photo of a typical lagoon

(from: http://www.waldronmi.us/sewer_water/sewer_project.htm)

The typical structure of a lagoon is an earthen basin constructed with sloping sidewall. Natural sealing will occur to some extent as wastewater solids enter the pores of the soil and reduce the seepage control. However it is now common practice to provide a liner for positive seepage control. Liner materials used include natural clays (such as bentonite), asphalt, synthetic membranes and concrete. Regardless of the liner material used, a concrete apron is often provided at the water line to simplify maintenance. The reminder of the side wall is usually covered by grass. Influent and effluent structures complete the lagoon. Influent enters at one end, and treated wastewater is collected at the effluent structure, usually located at the opposite end (Grady, 1999).

The ecosystem inside the lagoon is complex. As in any biological treatment process, microorganisms break down and convert organic matter into new microorganisms with end products of carbon dioxide, water and other inorganic substances. Algae are present in significant amounts in many lagoons and they use carbon dioxide and sunlight via photosynthesis to produce new cells and oxygen. The oxygen balance inside the lagoon depends on the activities of the algae and aerobic microorganisms as well as oxygen transfer from the atmosphere.

In cases when oxygen is absent, for instance at the bottom of the lagoon or when the lagoon is covered, anaerobic reactions occur. Complete anaerobic decay results in the conversion of organic matter to carbon dioxide and ammonia. Intermediate products of anaerobic decay are soluble low molecular weight organic acids and other compounds that are released in to the upper layers in the lagoon and become available for attack by the suspended aerobic microorganisms if there is oxygen in the upper part. Both, aerobic and non-aerobic reactions inside the lagoon are dependent on factors such as temperature, light, wind action, detention time and geometry of the lagoon (Droste, 1997).

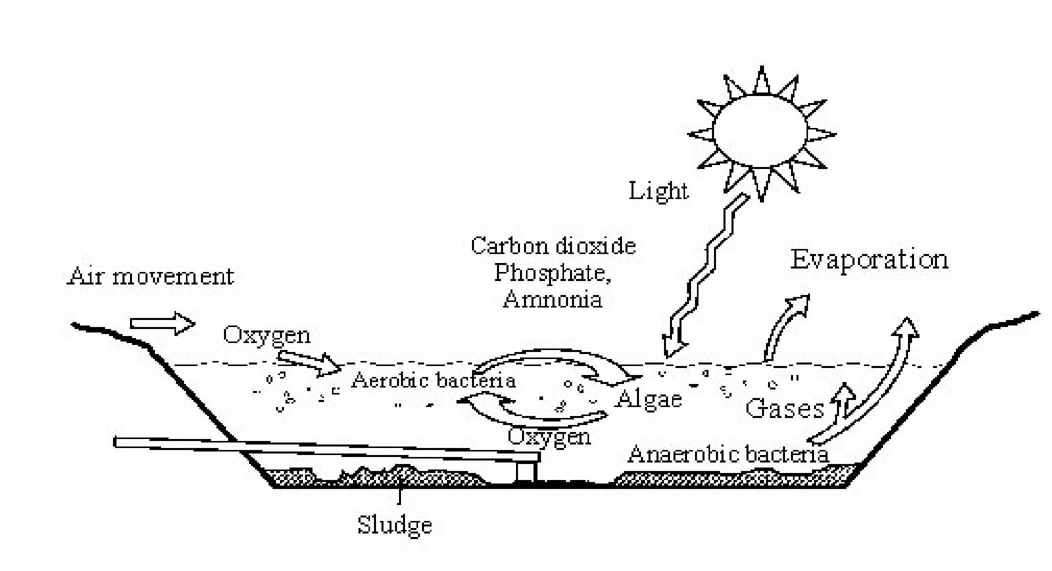

Figure 2.2. Processes occurring within a lagoon
(from: http://ianrpubs.unl.edu/wastemgt/g1441.htm)

Lagoons are classified according to the metabolic regime present in the lagoon with the following classifications usually used:

1. Anaerobic

In anaerobic lagoons, oxidation is inhibited through applying a high organic loading, where the result is anaerobic reactions. This type of lagoon is suitable for colder climates. They typically use less land as they can be made deeper than other lagoons, but require a longer retention period than the others types of lagoon. They can cause odours, especially when being cleaned (Grady, 1999).

2. Facultative

Facultative lagoons have an aerobic top layer, an anaerobic bottom layer and in between a tiny facultative layer where both aerobic and anaerobic reactions occur.

They usually tend to be large and shallow to allow for maximum diffusion of oxygen, which occurs at the surface, and for the maximum amount of algal growth to take place. The algae play a vital role in the lagoon as they use sunshine to convert carbon dioxide to oxygen that is used by the bacteria in the aerobic layer to digest biodegradable wastes. Facultative lagoons cause less odour and if designed properly could provide the advantage of digesting wastes, so eliminating the need for sludge disposal. However, facultative lagoons may have problems functioning during cold periods when ice forms on the surface.

3. Aerobic or maturation

Aerobic lagoons (maturation ponds) are aerobic systems. They tend to be large and shallow to allow for maximum diffusion of oxygen, which occurs at the surface, and for the maximum amount of algal growth to take place. Aerobic lagoons are one of the cheapest options of treatment but they require a large area of land and are usually located in areas where the climate permits year round algal growth.

4. Aerated lagoons

Lagoons that are oxygenated through the use of mechanical or diffused aeration units are termed aerated lagoons. The electrically powered aeration helps in maintaining aerobic conditions and improving the mixing inside the lagoons and consequently the time required to stabilise organic matter in the lagoon. The degree of aeration provided in aerated lagoons is variable. Some systems have a degree of aeration sufficient to keep all solids in suspension, whereas others referred to as partially mixed lagoons, have less power input. The special merits of this system are its low maintenance requirements compared with activated sludge processes, high operating reliability, ability to meet standards, reduction in area demand and flexibility in all climates (NoSWA, 2001).

(b) Activated-Sludge Processes

The activated-sludge process was developed around 1913 at the Lawrence Experimental Station in Massachusetts by Clark and Cage, and by Ardern and Lockett (1914) at the Manchester sewage works in Manchester, England (Metcalf and Eddy, 2003).

The activated-sludge process was so named because it involved the production of an activated mass of microorganisms capable of stabilising a waste under aerobic conditions.

In the aeration tank, contact time is provided for mixing and aerating influent wastewater with the microbial suspension, generally referred to as the mixed liquor suspended solids (MLSS) or mixed liquor volatile suspended solids (MLVSS). Mechanical equipment is used to provide mixing and transfer of oxygen to the process. The mixed liquor then flows to a clarifier where the microbial suspension is settled and thickened. The settled biomass, described as activated sludge because of the presence of active microorganisms, is returned to the aeration tank to continue biodegradation of influent organic material. A portion of the thickened solids is removed daily or periodically as the process produces excess biomass that would accumulate along with the non-biodegradable solids contained in the influent wastewater (Metcalf & Eddy, 2003).

The main advantage of activated sludge processes is that they are able to produce an effluent low in organic compounds and thus can be used to meet strict effluent standards. The system can be effectively used as part of a larger system when the removal of nutrients nitrogen and phosphorus are required. Other advantages are that it can be located on a small area of land, and it is relatively easy to expand this system by adding additional reactors. However, the operation of this system is more complex than others. The system does tend to be more costly to construct and operate than other systems, but usually linked with fewer maintenance problems over its lifetime (Water Quality Programme Committee, 1996).

(c) Oxidation Ditches

Oxidation ditches are a variation on the activated sludge process that features a shallow ditch (large holding tank) constructed with an impermeable lining. This gives the wastewater plenty of exposure to the open air for the diffusion of oxygen, helping prevent anaerobic conditions from occurring. One or more mechanical, horizontal, surface-aerators are used to slowly rotate, introducing oxygen to the wastewater and inducing mixing.

Raw sewage is delivered to the ditch where it is slowly mixed by the aerators and the long retention time allows for a great amount of organic matter within the sewage to be broken down by the aerobic bacteria. After aeration the effluent is pumped to a settling tank where the sludge and the water are allowed to separate or in some systems the aerator is turned off and the ditch serves as a settling tank (Horan, 1991). The sludge is removed from the bottom of the settling tank and a portion is taken to the ditch to facilitate microbial activity in the next batch of sewage.

Early oxidation ditches consumed great areas of land but recent oxidation ditches are mostly suitable for communities with limited access to land (Horan, 1991). Initial construction costs are relatively high, but the system's energy demands are moderate. The system requires a moderate amount of skill to operate and maintain, and it works well under most weather conditions (Water Quality Programme Committee, 1996).

(d) Trickling Filter

Trickling filters are circular tanks containing a packing material. Historically, rock was most commonly used as the packing material, with depths ranging 1.25 to 2m and reinforced concrete foundations to support the rocks. Most modern trickling filters are designed with plastic packing material that do not need much support and so can be built above the ground. Because of this, they are referred to as tower filters. Tower filters are typically 5 to 10m high and have smaller diameters than filters with rock media. They are designed such that about 90 to 95% of the volume in the tower consists of void space (Metcalf and Eddy, 2003).

Sewage first goes to a settling tank where much of the solid matter settles out of the wastewater. Most systems use a rotating distributor to distribute settled sewage on to the surface layer of the packing material, but some use a fixed distributor. Rotating distributors spread the sewage more evenly than fixed distributors, but they require more energy to operate. The distribution of the influent can either be intermittent or continuous depending on the system. When continuous, a portion of the wastewater is re-circulated back to the distribution system (Water Quality Programme Committee, 1996). The media act as a substrate to which microorganisms attach themselves. Empty space between the packing material allows for the presence of air, creating an aerobic environment for the microrganisms. Packing material made of plastic has more empty space, thus allows greater oxygen transfer. As the wastewater passes over the packing material, the microorganisms feed upon organic material. These microbial populations eventually grow to form a layer of slime over the media. Portions of this slime are sloughed off each time wastewater passes through the filter.

After it has been collected in an under-drain beneath the filter, the wastewater is then sent to a second settling tank where slime debris is allowed to settle out (Metcalf & Eddy, 2003).

Trickling filters are good at removing nitrogen and organic matter from the wastewater. This makes them beneficial for communities with strict nutrient discharge standards. Trickling filters can be expensive to build and systems that use a rock media are usually more expensive than those that use plastic. Moderate skill requirements are needed for maintenance and operation, and energy requirements will vary depending on the system. These systems are not well suited for very cold climates and may also cause odour problems (Water Quality Programme Committee, 1996).

Figure 2.3.Trickling filter showing the distribution arm and media
(from: http://www.vicksburg.org/pages/wastewater/photos.htm)

(e) Other systems recommended by the EA and SEPA

The following systems are recommended by SEPA and EA for non-sewered effluent discharges, where the connection to a sewer is not practicable (Abis, 2002)

1. Reed Beds

The problem with reed beds is that they usually require some prior solids settlement to avoid clogging.

2. Septic tanks

Septic tanks do not require prior solid settlement, but they require desludging at least every 12 month and their effluent usually requires further treatment before discharge.

3. Package plants

Package plants can treat wastewater to a high standard, but they require electrical power and regular skilled maintenance to ensure effective operation. They are also very expensive on a per population equivalent.

4. Composting toilets

Composting toilets are particularly suitable for every remote areas where there is no mains water supply

5. Cesspools

Cesspools are not sustainable, as they require frequent, expensive emptying services.

2.3.3 Section conclusion:

The EC UWWT Directive requires all communities of less than 2000 people in the European Union to have appropriate treatment before 31 December 2005. In the UK, appropriate treatment is interpreted by the authorities to be septic tanks, trickling filter, activated sludge plants, reeds beds or equivalent systems. Equivalent systems include lagoons, oxidation ditches, cesspools, composting toilets and package plants. The choice between these systems for treatment of a small community wastewater is governed by the advantages and disadvantages of each system and local conditions.

2.4 The EC Urban Waste Water Treatment Directive and Its Implications

2.4.1 The EC Urban Waste Water Directive

The EC UWWTD concerns the collection, treatment and discharge of urban wastewater and the treatment and discharge of wastewater from certain industrial sectors. The objective of the EC UWWT Directive is to protect the aquatic environment from the adverse effects of the above-mentioned wastewater discharges. The standards set by the Directive depend on the size of the population served and whether the receiving waters are classified as sensitive or less sensitive. Less sensitive areas are described by the Directive as the areas where discharge of wastewater does not adversely affect the environment as a result of morphology, hydrology or specific hydraulic conditions which exists in that area. On the other hand, the Directive defines an area as sensitive if it falls in to one of the following groups:

1. Natural freshwater lakes, other freshwater bodies, estuaries and coastal waters which are found to be eutrophic or which in the near future may become eutrophic if protective action is not taken.
2. Surface freshwaters intended for the abstraction of drinking water which could contain more than the concentration of nitrate laid down under the relevant provisions of council Directive 75/440/EEC of 16 June 1975.
3. Areas where treatment of wastewater is necessary to fulfil council Directives.

The UWWT Directive requires secondary treatment for all discharges greater than 15,000 population equivalent for all receiving waters by 31st December 2000. Discharges to inland and estuaries waters of between 2000 and 15,000 population equivalent and those between 10,000 and 15,000 PE to coastal waters must receive secondary treatment by 31st December 2005. All WWTW discharges less than 2000 population equivalent must receive appropriate treatment by the end of 2005, as must discharges with a population equivalent between 2,000 and 10,000 to coastal waters.

Appropriate treatment is defined by the Directive as treatment of urban wastewater by any process and/or disposal system which often discharge allows the receiving waters to meet the relevant quality objectives and the relevant provisions of this and other community Directives. Primary treatment is defined by the Directive as solid settlement by physical or chemical means to remove 50% of the suspended solids and 20% of the BOD and secondary treatment is defined as biological treatment to reduce BOD and suspended solids to levels suitable for discharge.

The EC UWWT Directive requires that member states shall ensure that the urban wastewater treatment plants built to comply with the Directive are designed, constructed, operated and maintained to ensure sufficient performance under all normal local climatic conditions. The Directive also requires competent authorities or appropriate bodies to monitor discharges from urban wastewater treatment plants, amounts and composition of sludge disposed of to surface waters and waters subject to discharges from urban wastewater treatment plants and direct discharges in cases where it can be expected that the receiving environment will be significantly affected.

In order to ensure that information on the disposal of waste water and sludge is made available to the public, the EC UWWT Directive requires all member states to ensure that every two years the relevant authorities or bodies publish situation reports on the disposal of urban waste water and sludge in their areas. Furthermore, the Directive requires member states to establish and present to the commission national programmes for the implementation of the Directive. Finally, the Directive requires member states to establish Committees to assist the Commission on matters relating to the implementation of this Directive and its adaptation to technical progress (Council of the European Communities, 1991).

It is important to note that sludges are not wastewater within the EC UWWT Directive. The UWWTD differentiates between wastewaters and residual sludges.

2.4.2 Implications of the EC UWWT Directive

(a) Investment

The EC UWWT Directive created pressure on governments, water authorities and water companies across the European Union to deliver improvements to their treatment works and to construct new ones to meet the EU standards, which meant increased investments on wastewater treatment (Goldsmith, 2002). Figure 2.4 was taken from a European Investment Bank report and shows the average annual investment in every EU country to comply with UWWTD from 1993 to 2005, where it can be seen that the investment cost could run to over a billion Euro in some countries. Nevertheless, the question which needs to be asked is who is going to pay for these investments? The answer is that, taking the UK as an example, the domestic customers are paying the greatest proportion of the cost. Considering that water companies are pressured by other factors such as tax and profit and government subsidiaries are usually not enough, the result is escalating water bill prices which the customers need to pay, though according to many surveys most customers are not willing to pay more even if it benefits their environment. The case could be even worse in holiday locations, as domestic bills for water include high seasonal loads from visitors leaving local customers to pay for the improvements, and so to summarise this could be stated that cost and benefit have not been weighed up sufficiently in the formation of the Directive (Mills, 1999).

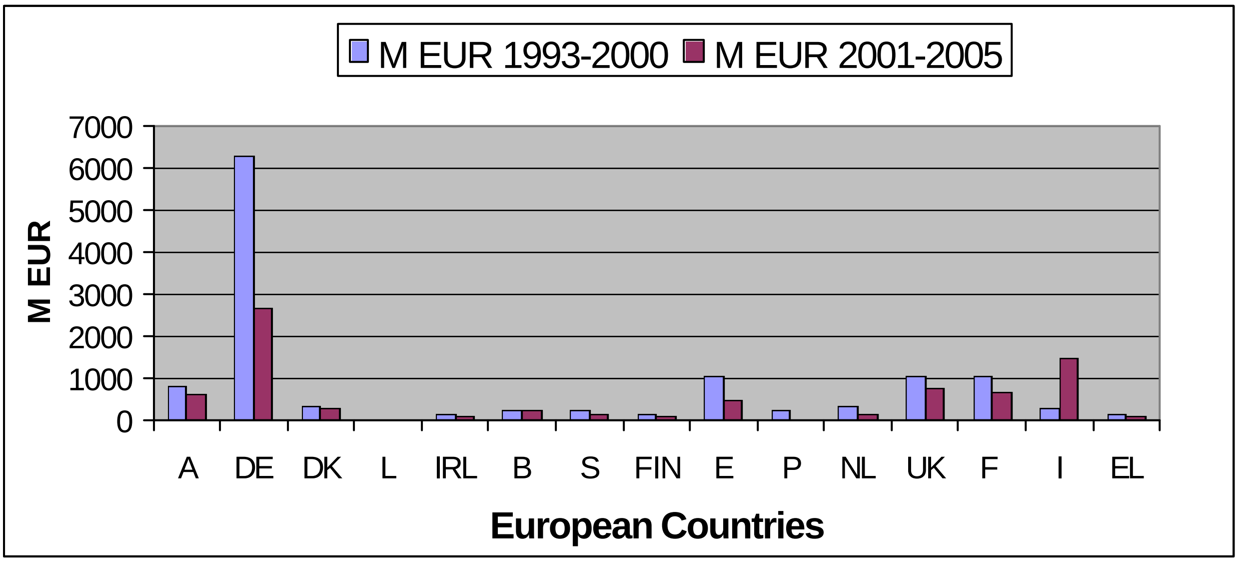

Figure 2.4. Average annual investment in Europe required to comply with the UWWTD, 1993-2005

(b) Small treatment works

According to Griffin and Pamplin (1998) the greatest risk to compliance with the EC UWWT Directive is for population equivalents less than 2000. The reason why is that compared to large works, small works are subject to operation and maintenance problems and large *per capita* costs. Moreover the number of works required to treat all less than 2000 people wastewater discharges can be enormous. In the UK 74% of treatment plants are for populations less than 2000, where as in the north of Scotland 94% of treatment plants are for populations less than 2000 (Abis, 2002). Furthermore, small sewage treatment plants are usually subjected to varied flows and augmented by large point sources. In order to fulfill the requirement of the Directive, water companies started to search for appropriate treatment systems for small communities, keeping in mind the important criteria of capital cost, operation cost, performance, sustainability, safety, minimisation of sludge produced, visual impact and simplicity (Abis, 2002).

(c) Receiving water quality

The EC UWWT Directive will definitely contribute toward improving coastal water quality however it has no requirements for removal of pathogens and thus other measures may be needed to improve bathing water quality (Mills, 1999).

(d) Sewerage standards

The EC UWWT Directive will help to equalise general sewerage standards throughout the EU, which means more people are connected to a recognised sewerage system (Mills, 1999).

(e) Transparency of water companies to the public

The EC UWWT Directive requires sampling of discharged effluents, publication of progress reports and implementation programmes. This will help to make the water services industry in the UK more transparent and accountable to the public in its improvements, programme and achievement of results (Mills, 1999).

(f) Member states involvement

The EC UWWT Directive does allow for some subsidiarity in that member states are being allowed to designate less sensitive and sensitive areas, though many states are using this to designate fewer sensitive areas. The fact that the European Union member states will have the same sewerage standards could help in enhancing cooperation between the states in the field of wastewater treatment (Mills, 1999). Also, the Directive could help in stopping disputes between member states, which may result from polluted water passing from one state to another

(h) Sludge disposal options

There is no EU guidance on sludge disposal to make it easier for water authorities and companies. Water companies in the UK are pressured with limited options for sludge disposal especially sea dumping is banned and the fact that they have no control over suitable land for sludge disposal on land (Mills, 1999).

(i) Compliance with the EC UWWT Directive

Some EU joint funded projects are failing to meet the standards set by the Directive. Also, the EU seems to be having difficulty gathering accurate data as to current levels of treatment in member states and hence how member states are progressing with compliance (Mills, 1999).

2.4.3 Section conclusion

The EC UWWTD was established with the intention of protecting the environment from the adverse effects of wastewater discharges. The Directive sets minimum requirements for the collection, treatment and discharge of wastewater in member states. The positive aspects of this Directive include; increased investment in wastewater projects to comply with the Directive, equal sewerage standards across the member states, improved member states relations, improved coastal water quality and transparency of water companies to the public. However, failure of many of the projects that were initiated by the EC UWWTD and inability in many member states to collect information on collection, treatment and discharge of wastewater could mean more control is required.

2.5 Changes within Scottish Water

Prior to the 1st of April 1996, water and sewerage services in Scotland had been managed by local government in nine mainland regions and three island regions. After the 1st April, all proprieties, rights and liabilities to which the regional and island councils were entitled in exercise of their functions relating to water supply and the provision of sewerage was transferred to three new authorities, East of Scotland Water Authority, West of Scotland Water and North of Scotland Water Authority (Marketing Science Institute, 1996). The responsibilities of these authorities were split as follows:

1. The East of Scotland Water Authority assumed responsibility for the Lothian, Borders, Central and Fife regions and also most of the assets of central Scotland development board.
2. The West of Scotland Water assumed responsibility for the Dumfries and Galloway and the majority of the Strathclyde regions.

3. The North of Scotland Water Authority covers the Grampians, Highlands and Tayside regions plus the Western isles, Orkney and Shetland Islands councils. The region covers a significant 60% of the land area in Scotland.

Scottish Water and Sewerage Customers Council monitored the three new authorities, to represent the interests of customers and potential customers of the new water and sewerage authorities.

In 2002, these three water authorities were merged to form Scottish Water, a public sector company that is answerable to the Scottish Parliament. Scottish Water serves a population of 5.3 million people and is monitored by the Scottish Environmental Protection Agency (SEPA) (Abis, 2002).

2.6 The Aero-Fac System

2.6.1 Introduction

The LAS Aero-Fac is a biological system that uses the basis of reactions in facultative lagoons to provide a novel treatment system that extend the advantages of facultative lagoons and overcomes the problems associated with them. It was developed by LAS who spent 20 years in the market of wastewater treatment (LAS International Ltd, 1999).

2.6.2 The treatment process

The treatment process of the Aero-Fac system is 100% biological requiring no pre screening or removal of any biodegradable solids, as all sludge is self digested by the biology within the primary treatment cell and inert wastes are accommodated for the life of the system (NoSWA, 2001).

Similar to the concept of treatment in a normal facultative lagoon, biodegradable organic matter in the wastewater is stabilised by both aerobic and anaerobic processes. Both reactions occur at the same time and in the same lagoon because dissolved oxygen does not reach the lower part of the lagoon and the result is that the lagoon divides into 3 layers: the top layer where the conditions are aerobic, the lower layer where conditions are anaerobic and an in-between zone where both conditions can occur (called the facultative zone) (Grady, 1999).

Figure 2.5. The three zones within the Aero-Fac lagoon
(from http://www.LASinternational.com)

When the raw wastewater enters the facultative lagoon, particulate organic matter settles to the bottom to be digested anaerobically by the anaerobic bacteria, resulting in the release of gases such as methane, carbon dioxide, ammonia and hydrogen sulphide. In the upper aerobic zone, aerobic bacteria digest dissolved solids and produce carbon dioxide and water. The gases produced in the lower zone are stabilised in the aerobic zone by dissolved oxygen and this reduces the odour problems. In the facultative zone, both aerobic and anaerobic bacteria can operate to break down matter aerobically or anaerobically. The final effluent is usually taken from the aerobic zone similar to most facultative lagoons.

The main difference between an Aero-Fac facultative lagoon and a normal facultative lagoon in term of the process of treatment is that in the Aero-Fac lagoon the diffusers maintain the level of oxygen and hence facultative condition inside the lagoon throughout the process.

2.6.3 Ability of the Aero-Fac system to digest sludge

When the heavy solids settle to the bottom of the facultative lagoon, they are broken down by anaerobic bacteria to produce gases such as methane, hydrogen sulphide, carbon dioxide and ammonia. These gases are released in the anaerobic zone but stabilised in the aerobic layer and this is claimed to reduce the odour problem. On the other hand, dissolved solids that do not settle are biodegraded in the aerobic zone by aerobic bacteria. The result of these processes means that only a few centimeters of sludge are generated initially and if the conditions are right this layer should grow steadily or not grow at all depending on the amount of non-biodegradable matter in the wastewater (LAS International Ltd, 2000).

This is claimed by LAS to be proven in many cases, one of them through a 10-year study of an aerated facultative system performed by the State of Kentucky Environmental Agency to verify the possibility of the lagoon not producing sludge. This was consequential by the fact that there are hundreds of lagoons throughout the US that have not been desludged for years. The study concluded that the system is truly self-digesting and there was no sludge growth for whole periods except during a few periods when the zone of aeration was out of action (LAS International Ltd, 2000).

It must be stressed that not every facultative lagoon is effective in the digestion of waste, as the right conditions have to be available and one of the advantages claimed for the Aero-Fac system is that facultative conditions are maintained throughout the process by means of monitoring the oxygen levels and supplementing the optimum amount of oxygen as required (LAS International Ltd, 2000).

2.6.4 The benefits provided by the Aero-Fac system

(a) Low operating cost

The claimed operating costs of this process are very low and in fact, up to 85% less than some activated sludge or extended aeration plants (NoSWA, 2001). In addition equipment items are very simple and designed for greater than a 20 year life, which makes capital replacement costs low (LAS International Ltd, 2000).

(b) Low construction costs

The fact that the process of treatment is nearly an entirely natural process means it should be cheaper to construct than electro-mechanical treatment plants. It should also be cheaper to construct than many types of activated sludge or extended aeration plants (NoSWA, 2001).

(c) Design capacity and above

Performance data from Carrington Aero-Fac in the US has shown that the system was working effectively in delivering an acceptable effluent standard even though operating at double the design flows for two years (LAS International ltd, 2000).

(d) Reduced energy requirements

The Aero-Fac system blends local wind power with low energy diffused air and this is why it requires less electrical energy to operate compared with extended aeration. (NoSWA, 2001). The diffused air is automatically switched on when wind power is not enough to generate the required oxygen to maintain facultative conditions inside the lagoon.

(e) Flow Flexibility

The Aero-Fac system can operate with virtually no flow, reverting to wind power only (and almost zero operating cost) or large flows, adjusting itself automatically based on influent flows and strength at any given hour, week or year (NoSWA, 2001).

(f) Robustness

The Aero-Fac system utilises a simplified, low energy, non-clog, stainless steel diffused air system with a long design life. It requires no ongoing routine service, cleaning, maintenance or spares and comes with a 20 year warranty. Also the fan blower used by the system has no internal bearings, thus producing almost no wear and requiring no ongoing service or maintenance (NoSWA, 2001).

(g) Sludge digestion

All sludge is claimed to self-digest in the Aero-Fac lagoon and this means that no sludge handling or disposal and so no pre-screening systems, clarifiers or other equipment is required. This also means there is no sludge disposal cost (LAS International Ltd, 2000).

(h) Ammonia-Nitrogen removals

The special LAS design and equipment arguably provide the correct environment and condition for ammonia and nitrogen removal by reducing the overall BOD in the first stage of treatment and providing an oxygen rich, well-mixed environment in the second polishing stage (NoSWA, 2001).

2.6.5 Application of the Aero-Fac system in the UK

The first use of aerated facultative lagoons in the UK began in 1993 with a one year pilot scheme undertaken by Wessex Water and LAS International at the Wick St. Lawrence wastewater treatment plant near Bristol. The pilot-system consisted of three ponds in series, each with its own wind-powered aerator. It was fed with screened sewage from an equivalent population of 250 people. Results of influent and effluent samples between December and July showed average removal rates of 84-95% for BOD and 53-92% for suspended solids. The concentration of ammonia in the final effluent rose steadily throughout the pilot scheme to reach an average value of 11 mg/l (Abis, 2002).

The Aero-Fac studied in this thesis has been constructed at Errol and consisted of the construction, installation and commissioning of the following plant and equipment (Montgomery Watson, 2001).

Figure 2.6. The two facultative lagoons at Errol

1. Interception of the existing sewage system at three separate locations at the south side of Errol, and thereafter, a new gravity sewer routed to a remote combined storm overflow (CSO) chamber and a remote pumping station.
2. Combined storm overflow chamber, (CSO chamber) with auger type overflow weir screen, bypass screen and emergency overflow to the Pow Burn.
3. Pumping station with duty and standby submersible pumps forwarding flows to the new Errol wastewater treatment works via a new rising pipeline.
4. Inlet works consisting of a mechanically cleaned, inclined Rotomat Microstrainer screen with integrated screenings press and screening washing, mounted in a raised concrete inlet channel arrangement located at the inlet to the facultative system.
5. Primary and secondary facultative lagoons, with LAS mark 3 wind/electric powered floating processors.
6. Each lagoon has a low pressure air supply from centrifugal fans and stainless steal air duct main to static air diffusers located in each lagoon.
7. Balancing chamber on the outfall from the secondary lagoon to set the top water level in the lagoons and a flow measurement chamber with v-notch weir.
8. Gravity outfall to River Tay.
9. Portable water service derived from the mains supply and wash water booster set.
10. Work drainage sumps with submersible pump discharging to the inlet works.
11. Site power supplies and motor control centres which are located in the control building at the treatment works site in a kiosk at the remote pumping station site.

2.6.5.1 Flow of wastewater from Errol to the River Tay

Raw wastewater from the Errol village flows via a 525 mm diameter gravity sewer to the CSO chamber which has a storm overflow weir in case effluent levels rises. The wastewater then flows via a 300 mm diameter pipe to the pumping station. The pumping station has two pumps with the selection of the duty pump undertaken automatically. The effluent is pumped to the inlet works at Errol WWTW. There it is first screened through a Huber Rotomat Ro9 micro strainer. The screened sewage then flows to the primary lagoon after that to the secondary lagoon through an interconnecting pipeline. The final effluent flows out of the secondary lagoon to the outfall chamber through a V notch weir flow meter. Finally, the final effluent flows from an interconnecting pipe to the River Tay (Montgomery Watson, 2001). The following photos show some of the components of the Aero-Fac system during construction. (The photos were taken by Scottish Water)

Figure 2.7. The first lagoon during construction

Figure 2.8. The lagoon pumping station

Figure 2.9. The second lagoon during construction

2.6.6 Conclusion

The LAS Aero-Fac is biological system that bases its treatment on the natural reactions in facultative lagoons to come up to new system that extend the advantages of traditional facultative lagoons as employed in waste stabilisation ponds. The Aero-Fac system provides advantages such as low operating costs, low construction costs and less energy requirements compared to activated sludge processes and oxidation ditches. The system also provides benefits such as ability to deal with different flows, robustness and ammonia-nitrogen removal compared to other lagoon systems. However, the greatest advantage is the ability to digest sludge so eliminating the need to de-sludge the lagoon and pre screening stage. The system is currently used in the US, Canada and other countries.

CHAPTER 3: PROJECT AIMS AND OBJECTIVES

3.1 Introduction

The Aero-Fac system was chosen as the treatment option for Errol based on the claimed advantages of this treatment process which included: low operating and construction costs compared to conventional treatment options, robustness, flow flexibility and *in situ* sludge digestion.

The process was the first to be installed in the UK and thus the aim of this research study was to investigate in detail its effectiveness and performance in order to verify the claims made for it and to ensure that the design approach employed is appropriate for UK conditions.

A series of agreed objectives were drawn up between Scottish Water, MWH which would ensure that the above aim was met and these were:

1. Determine whether the lagoons operate in a facultative mode with separation of aerobic and anaerobic layers.
2. Determine sludge accumulation rates and seasonal cycles of accumulation and degradation.
3. Determine the validity of the design basis and revise the design equations accordingly.
4. Determine predominant processes for ammonia removal at given loadings and temperature and estimate their rates. Investigate options for enhancing nitrification.
5. Monitor operating and maintenance cost and suggest options for reducing cost.
6. Determine the predominant algal populations and their potential impact on meeting SEPA consents.
7. Determine rates of disinfection and the tertiary requirements necessary to achieve bathing water standards in the receiving watercourse.

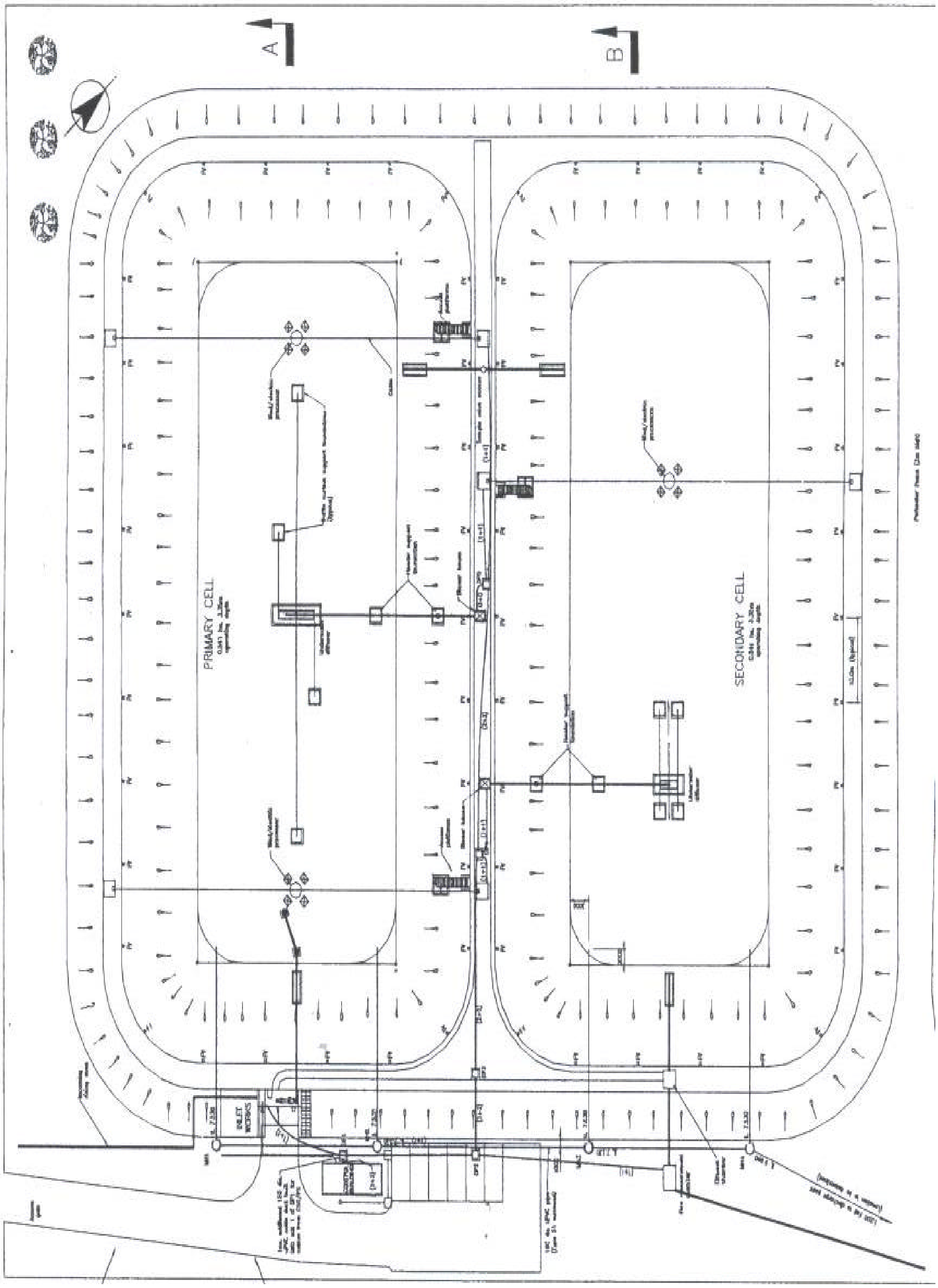

Figure 3.1 Plan drawing of the Aero-Fac system at Errol
(taken from NoSWA)

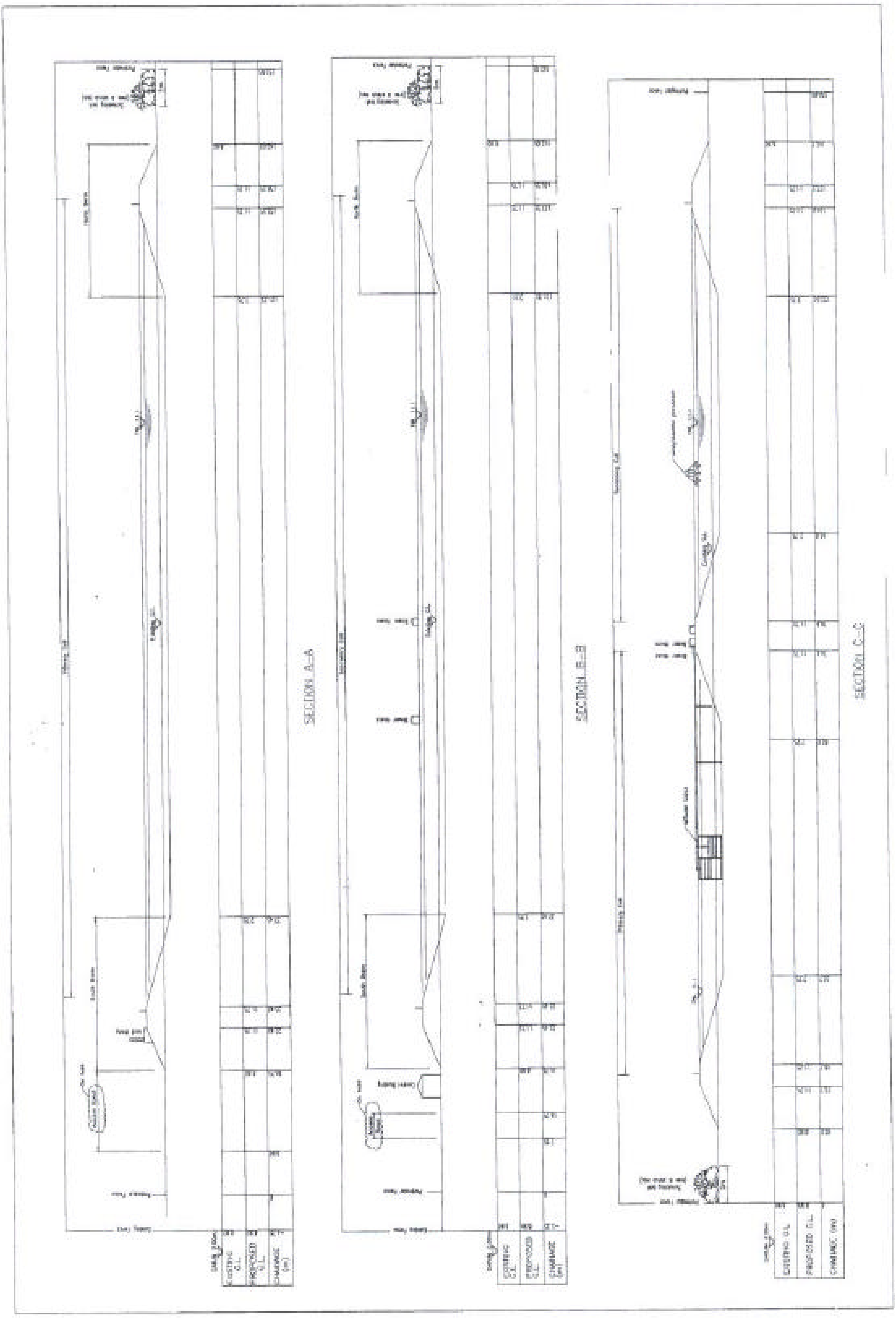

Figure 3.2 Cross-section of the Errol Aero-Fac lagoons (taken from NoSWA)

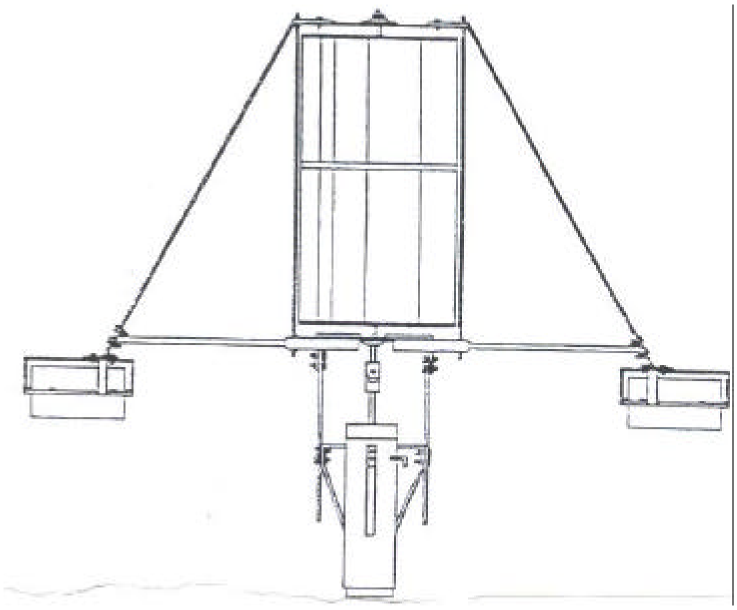

Figure 3.3 Wind-powered processor at Errol (taken from NoSWA)

3.2 Analysis Required to Achieve the Project Objectives and What Has Been Achieved

The project objectives were considered further in the light of the analysis required to achieve them and what have been achieved. Consequently Table 3.1 has been prepared to establish what was required and what was actually achieved during the project.

Table 3.1. Project resource requirements in order to achieve the above objectives and work undertaken during the project

	Objective	Requirement to Meet Objective	Analysis Frequency	Actual analyses carried out
1	Determine whether lagoons are operating in a facultative mode with separation of aerobic and anaerobic layers	Seasonal stratification studies to measure: • Temperature • DO • pH	Twice a week. This was later modified to 6 times in total	4 times. But, later discarded because of irrelevant results
2	Determine sludge accumulation rates and seasonal cycles of accumulation and degradation	Seasonal measurement of sludge depth	Three times over the study period.	Due to the concern over the safety of the white towel test, the experiment was carried once.

3	Determine the validity of the design basis and revise current design equations accordingly	Tracer study to determine mixing regime and "tanks in series". Seasonal sampling of influent and effluent BOD and COD (filtered and unfiltered) for both lagoons	1/ Rhodamine measured daily for 60 days on two occasions. 2/ Influent and effluent sampled twice weekly for COD, BOD and solids throughout the study.	1/ The tracing study experiment was carried once. 2/ Influent, mid-lagoon and effluent sampled once a week for COD and BOD and suspended solids.
4	Determine predominant processes for ammonia removal at given loadings and temperature and estimate their rates. Investigate options for enhancing nitrification	Seasonal sampling of influent and effluent for both lagoons for: • Ammonia • Nitrate • Algal biomass (*chlorophyll a*)	Influent and effluent sampled twice weekly over three periods of 4 weeks for • Ammonia • Nitrate • *Chlorophyll a*	Influent, mid-lagoon and effluent sampled once a week over 4 periods of 3 weeks.
5	Monitor operating and maintenance cost and suggest options for reducing these	Set up site operatives visit diary, log daily power consumption and estimate power required for aeration	No analysis required	No analysis was required.
6	Determine predominant algal populations and their impact on meeting SEPA consents	Seasonal sampling as specified above, plus the following analysis of influent and effluent for both lagoons: • pH • Microscopic algal speciation	Samples taken in 1 above examined microscopically	Influent, mid-lagoon and effluent sampled once a week over 4 periods of 3 weeks for pH and algae
7	Determine rates of disinfection and the tertiary requirement necessary to achieve Bathing Water standards	Seasonal sampling of influent and effluent for both lagoons for: • Faecal coliforms • Faecal streptococci • Enterovirus Lab-trials of UV disinfection on effluent	Samples taken in 4 above analysed for faecal coliforms, faecal streptococci and enteroviruses. Lab trial on UV disinfections of *E. coli*	Last 4 samples were analysed for Total Coliforms, faecal coliforms and E.*coli*

3.3 Materials and Methods

3.3.1 Introduction

The aim of this research project was to fulfill seven objectives drawn up between Scottish Water, Montgomery Watson Harza and the University of Leeds. The analytical work undertaken was carried out both by the author and by the analytical laboratories of Scottish Water. The later were used when accredited data was required that would be used for assessing compliance with SEPA standards.

3.3.2 Performance tests summary and analysis

The contract between SW and MWH includes the use of performance tests over a period of 1 year commencing immediately after the issue of the Take-Over Certificate, in order to meet MWH's Contract Performance Guarantees, which to achieve the tests results should achieve 95% compliance with 95% confidence. These performance tests include effluent quality tests and power consumption tests.

The Effluent Quality Performance tests comprise the analysis of final effluent over a period of 28 consecutive days, together with the analysis of crude sewage over a period of 40 days preceeding each 28-day period. The purpose of final effluent testing is to determine whether it complies with SEPA's discharge consent and with MWH's contract Performance Guarantees. The reason for crude sewage analysis is to protect MWH from having to guarantee final effluent quality in the event that the crude loading was considerably higher than could reasonably be anticipated by any party to the contract.

3.3.3 Method and materials used for Errol study (University of Leeds)

The following sections describe the methodology employed by the author in those parts of the testing protocol which were the responsibility of the University of Leeds.

(a) Algal identification

For each period of sampling 4 influent, mid-lagoon and effluent samples were taken for identification. Identification of algae in the samples was done using a microscope, where a magnification of x100 was used to identify the large cells and a magnification of 400 was used to identify the smaller algae.

(b) Chlorophyll *a* Determination

Determination of Chlorophyll *a* in influent and effluent samples was required to fulfil objective 4 in the Errol R&D objectives, which was to determine predominant processes for ammonia removal at given loadings and temperature and estimate their rates and to investigate options for enhancing nitrification.

The sampling regime for chlorophyll *a* determination was in three periods covering summer, autumn/winter and spring. Results of chlorophyll *a* determination are available in chapter 4 and an analysis of the results is presented in chapter 5.

For each period of sampling for chlorophyll a identification; 4 influent, mid-lagoon and effluent samples were taken. Successive 10 ml volumes were filtered through a filter paper until pumping become difficult. The filter paper was then placed in a graduated tube and pushed down to the bottom. Then, a 10 ml volume of 90% methanol was added and the tube placed a water bath at 60^{O} for two minutes. After allowing the tube to cool, the resultant solution was filtered through a 25 mm filter rig and the filtrate collected. Finally, the filtrate was tested for absorbance at wavelengths of 663 and 750 using a spectrophotometer. The following equation was used to determine chlorophyll a from the spectrophotometer result:

$$\text{Chl-a}(\mu g/l) = (\text{abs}663 - \text{Abs}750)/\ 77 \times 10/\text{vol} \times 10^{6}$$

(c) Sludge Depth Monitoring

Two people were required to undertake the white towel test; one person was on shore for safety monitoring, while the other person carries out the test in the dinghy. The stages of carrying out the test were as follows;

1. Outside the control room, the boat was inflated to the pressure recommended by the manufacturer using the pump supplied with the dinghy. The boat was then lowered in to the first lagoon after it was secured to the platform by tying a rope to the front of the boat. A second rope was attached to the boat to tie the processors when carrying out the test for stability. One of the two people then don a life jacket and entered the boat with assistance when required. Then the poles, paddles, ruler, notebook and pencil were lowered down to him.
2. The pond was marked out roughly to give a grid for the test. The person in the boat paddled to the processor to attach the second rope to the processor,

when the wind was strong and then to the first and the following test points. At each test point, the pole was lowered to the bottom of the lagoon to mark the sludge depth. At the end of the tests, the person in the boat paddled back to the platform in addition to being pulled by the other person through the first rope. When the boat reached the edge of the lagoon, the person in the boat left the boat with assistance if required and removed his life jacket.

3. The boat was lifted out of the pond using the rope attached to the front of the boat. Boat removal required assistance from both individuals. Both persons can now transport the boat to the second lagoon.
4. One of the two people then don his life jacket and entered the boat with assistance when required. Then the poles, paddles, ruler, notebook and pencil were lowered down to him. The person in the boat paddled to the processor to attach the second rope to the processor, (when the wind is strong) and then to the first and the following test points. At each test point, the pole was lowered to the bottom of the lagoon to mark the sludge depth. At the end of the tests, the person in the boat paddled back to the platform in addition to being pulled by the other person through the first rope. When the boat reached the edge of the lagoon, the person in the boat departed the boat with assistance when required and removed his life jacket.
5. The boat was lifted out of the pond using the rope fixed to the front of the boat. Boat removal required assistance from both individuals on site. After the boat was removed from the second pond, it was transported to near the storage point. The boat was washed down with clean water prior to deflating and storage.

(d) Rhodamine tracer study

Rhodamine WT was used to determine the hydraulic retention time in the lagoons. 130 g rhodamine WT was dissolved in 131 tap water and two batches prepared, one for each lagoon. This gives an initial dye concentration of 100 mg/l. One batch of the solution was then poured into the inlet of each lagoon. The effluent and mid-lagoon samplers were set to take effluent and mid-lagoon samples, twice daily for 24 days. The samples were analysed for fluorescence using a fluorimeter, and HRT was determined based on the time taken for a peak in fluorescence to be detected.

3.3.4 Method and materials used for the Errol study (Scottish Water)

Analyses for pH, ammonia, nitrate, BOD, COD, suspended solids and Bacteriology were carried in Scottish Water Laboratory under accredited procedures in place at the laboratories. Influent and effluent samples were taken using a fixed Aquatics Aqua 10 refrigerated wastewater sampling system, while mid-lagoon samples were taken using portable samplers. Influent samples were taken immediately down stream from the inlet works stream. Mid-lagoon samples were taken from the interconnecting pipe between the first and second lagoon. Final effluent samples were taken from the final effluent sampling chamber.

It was suggested by Scottish Water that mid-lagoon and effluent samples should be taken after their respective influent samples according to the retention time, but due to the frequent breakdowns in samplers and the high cost of transportation, many mid-lagoon and effluent samples were not taken according to the estimated retention time.

All influent, mid-lagoon and effluent samples were analysed for COD, BOD and suspended solids. Analysis for pH, ammonia and nitrate took place on the three periods of influent, mid-lagoon and effluent samples. Bacteriological analysis took place on the last 4 influent and effluent samples. The following is the summary of the method used by Scottish Water to carry each of these tests.

(a) BOD

The biochemical oxygen demand was determined using a SKALAR SP100 Semi Robotic Analyser, after giving appropriate dilution to the samples and adding suitable microorganism and Allyl theourea to suppress nitrification. Similar to all BOD tests, dissolved oxygen was measured using a DO probe; the sample was then incubated in the dark at 20^{o}C for 5 days and afterward dissolved oxygen was measured again using a DO probe. The BOD was calculated from the difference in dissolved oxygen.

When samples were tested for total BOD; the samples were shaken before appropriate dilution was given.

(b) COD

The chemical oxygen demand was determined using Sealed Tube Mercury Suppressed Method with Spectrophotometric Determination. Samples were first mixed (unless filtered COD or Settled COD are required) and then diluted if necessary. Next, samples were oxidised through heating at approximately 148 ^{o}C for 2 hours with sulphuric acid and potassium dichromate. There silver sulphate was added oxidise the more refractory organic matter and Mercury sulphate was added to reduce the interference caused by the presence of chloride ions. The amount of chromium (III) generated in the process was quantified using a XION 500 spectrophotometer for two ranges; LCK314 (final effluent samples) and LCK114 (industrial samples, crude and primary sewages).

(c) Suspended solids

The amount of suspended solids was determined from the difference in weight of two recovered material, which resulted from placing two GF/C papers (one washed with the sample and the other with deionised water) in the oven at 105 ^{o}C for 15 minutes.

(d) pH

Well mixed (50 ml to 100 ml) volumes of the samples were added in stages to a 100 ml beaker. Then a magnetic stirrer was added for analysis and the beaker was placed under an electrode and ATC probe for 1 to 2 minutes to allow the reading to stabilise. Finally, the value displayed was recorded.

(e) Nitrate

The amount of nitrate was calculated empirically from the subtraction of analytically derived NO_2 from the analytically derived TON value. TON was determined by reacting the sample with a hydrazine –copper reagent to reduce any Nitrate present to Nitrite, then with sulphanilamide and N-1-napthylethylene di-amine in dilute hydrochlororic acid to produce azo-dye which its absorbance was measured to be used to determine TON. Nitrite was determined by reacting the sample with sulphanilamide and N-1-napthylethylene di-amine in dilute hydraulic to produce azo-dye which its absorbance was measured to determine the amount of Nitrite.

(f) Ammonia

Ammonia was calculated through reacting the sample with phenol and hypochlorite in the presence of nitro-prusside to form blue indophenol compounds which its absorbance was measured to determine the Ammonia level in the sample.

(g) Total coliforms, faecal coliforms and E.*coli*

Total coliforms are known to survive at 37 ^{o}C, while faecal coliforms and its sub group E.*coli* are capable of performing reactions at 44 ^{o}C. In Scottish Water laboratory; the usual standard bacteriological methods of were used to determine the numbers of total coliforms, faecal coliforms and E.*coli* in Errol influent and effluent.

CHAPTER 4: RESULTS

4.1 Influent Wastewater Composition

Using data from both the contractual performance tests undertaken by Scottish Water and the analysis undertaken as part of this study, the composition of the influent wastewater to the Errol lagoons was evaluated (table 4.1). The actual population discharging to the lagoon has been estimated at 1,200 by Scottish Water and thus, based on the EC recommended figure for per capita daily excreted BOD of 60g, then a daily BOD load of 72 kg would be expected. In addition based on a typical per *capita* daily water usage of 180 litres, a dry weather flow (DWF) of 216 m^3/d would be expected. By contrast the actual load averages 58.3 kg BOD/d with a DWF of 498 m^3/d (based on an average flow = 1.3 x DWF). Clearly therefore the flow and load received at Errol is not typical of a UK sewage as it has a higher flow and reduced load. This is also confirmed by results of the BOD test which give an average of only 90 mg/l. Indeed the highest BOD recorded was only 292 mg/l. Thus the sewage is very weak and may be indicative of a number of factors including: high rainfall, high infiltration into the sewers and undersized CSOs which overflow routinely and thus contribute to a loss of load.

However as the performance guarantees agreed between Scottish Water and Montgomery Watson Harza (MWH) for the influent wastewater BOD were 400mg/l, then clearly the Errol influent falls within this consent. Nevertheless the performance guarantees also had an upper limit of 800 mg/l for the COD, presumably based on the reasonable assumption that the COD:BOD ratio would be 2. However the actual COD:BOD is 3.4 which is extremely high for this type of domestic wastewater (table 4.2). Consequently at times the COD has been in excess of 2,000 mg/l which exceeds the performance guarantees. Certainly this figure is unusual and suggests a large non-biodegradable fraction which may have implications for the ability of the lagoons to comply with the UWWTD COD consent of 125 mg/l.

Table 4.1 Influent characteristics of the Errol raw wastewater based on the results of the Errol R&D tests and the performance tests

Parameter	Average	Range	No of Samples
Unfiltered BOD (mg/l)	91	8 – 292	179
Unfiltered COD (mg/l)	314	27 – 2140	182
TSS (mg/l)	148	12 – 1044	180
COD:BOD	3.4	0.6 – 40.4	*
Ammonia (mg/l)	14.7	0.5 – 46.9	132
Nitrate (mg/l)	1.3	0.1 – 9.6	132
pH	7.3	6.7 – 8.4	175
Flow (m^3/d)	647	166 – 5667	676
Load (kg BOD/d)	58.3	26 – 128	*

As a consequence of this anomaly with the COD:BOD the value of the other determinants were examined in more detail and the typical composition of sewage that might be expected from a population of 1,200 has been compared to that actually received at Errol (table 4.3).

Table 4.2 Typical raw wastewater COD:BOD ratios

Wastewater type	Typical COD:BOD ratio
Residential	1.25:1
Industrial	1.4:1
Agricultural	2.5:1

Table 4.3 Comparison of the composition of a predicted wastewater with that currently received at Errol

Parameter	Predicted (PE 1200)	Predicted (PE 2000)	Received at Errol
Unfiltered BOD (mg/l)	256	101	90.6
Unfiltered COD (mg/l)	513	351	314
TSS (mg/l)	375	165	147.9
COD:BOD	2	3.47	3.444
Ammonia (mg/l)	24.2	22	14.74
BOD:ammonia	10.6	4.6	6.15
BOD:TSS	0.684	0.6122	0.6125
Flow (m^3/d)	280.8	965.2	647.46
Load (kg BOD/d)	72	109	58.3

Comparison between the predicted sewage composition from a population of 1200 and the composition of the sewage actually received at Errol illustrates that the sewage received at Errol has a higher flow and reduced load than a typical UK sewage and thus reinforces the suggestion that the sewage received at Errol is weak. The predicted BOD (256 mg/l) is nearly triple the actual BOD concentration and the predicted total suspended solids is 2.5 times the actual TSS level in the received sewage. The predicted COD is comparative to the actual influent COD concentration but is still 1.6 times higher. The average ammonia concentration in a typical rural agricultural wastewater is 24.2 mg/l and this is higher than the average received influent ammonia concentration of 14.74 mg/l.

The second column in table 4.3 shows the predicted sewage composition from a population of 2000 (design population), based on the actual sewage received at Errol from a population of 1200. Comparison between the actual sewage received at Errol from a population 1200 and the predicted sewage composition from the design population shows an expected rise in daily flow and load. Also, all current influent wastewater characteristics (BOD, COD, TSS and ammonia) are predicted to rise as a sequence of population increase. This is important to bear in mind as the lagoons have been constructed to treat a population that is predicted to rise to 2,000.

4.2 Final Effluent Quality

The consents for the Errol lagoons are based on compliance at a 95 percentile value with samples taken at the lagoon outlet and these consents are summarised in table 4.4. Thus a comparison of this table with the actual performance of the lagoons as summarised in table 4.5 has been used to assess compliance.

The final effluent unfiltered and hence filtered BOD passed the SEPA and the performance guarantee consents for filtered BOD as 95% of the final effluent BOD results were below 20 mg/l. Comparison of the average influent BOD and average effluent BOD shows an average BOD removal by the system of 89%.

Table 4.4 Errol final effluent consent conditions

Consent	Filtered BOD (mg/l)	Total SS (mg/l)	pH	Filtered COD (mg/l)	Total Oxidised Nitrogen (mg/l)
SEPA consent	30	150	5-9	125	15
Contract Performance Guarantees for existing 1200 PE	20	150	6-9	*	*
Contract Performance Guarantees for future 2000 PE	25	150	6-9	*	*

Table 4.5 The characteristics of the final effluent during the course of this study

Parameter	Average	95 Percentile	Ratio Average : 95 Percentile	Number of Samples
Unfiltered BOD (mg/l)	9	20	1 : 2.3	163
Unfiltered COD (mg/l)	85	137	1 : 1.6	164
TSS (mg/l)	28	69	1 : 2.5	161
Ammonia (mg/l)	7.6	17	1 : 2.25	131
Nitrate (mg/l)	3.6	11	1 : 3	131
pH	7.7	8.1	1 : 1.05	152
Total coliforms	7×10^4	15×10^4	1 : 2.1	9
Faecal coliforms	2500	6550	1 : 2.6	5

There is no known SEPA consent for final effluent COD, but the EC UWWT Directive requires that the filtered COD in wastewater discharges should not exceed 125 mg/l. The 95%ile final effluent unfiltered COD value was 137 mg/l, but the average final effluent unfiltered COD was 85 mg/l. Comparison of the average influent unfiltered COD and average effluent unfiltered COD shows an average COD removal of 73%.

The 95%ile final effluent total suspended solids concentration is lower than the limit set by SEPA and the performance guarantees. Comparison between the average influent and average effluent suspended solids concentration shows an approximate suspended solids removal rate of 81%.

Both, the average final effluent pH and the 95%ile pH value were also within the range set by SEPA and the performance guarantees for final effluent pH which are 5 to 9 and 6 to 9 respectively.

4.3 Pond Performance

It is usual to assess the actual performance of a pond using samples taken from mid-lagoon as these give a more realistic assessment of overall biological performance and are not subject to the diurnal variations generally seen in the effluent samples (Mara, 2001). Mid-lagoon performance is summarised in table 4.6 and the overall performance of the pond system was evaluated from these average results.

Table 4.6 Mid-lagoon effluent characteristics

Parameter	Average	Range	No. of Samples
Unfiltered BOD (mg/l)	19	6 – 98	50
Unfiltered COD (mg/l)	128	29 – 317	44
TSS (mg/l)	60	13 – 174	51
Ammonia (mg/l)	9.8	1.23 – 27	39
Nitrate (mg/l)	1	0.3 – 7.3	39
pH	7.5	6.9 – 9.2	41

4.3.1 Primary pond performance

The primary pond in Errol includes two wind-powered processors, one more than the secondary lagoon and this is necessary because the primary pond receives a higher loading than the secondary pond. Primary pond performance will affect the characteristics of the effluent outgoing to the secondary pond and thus is directly responsible for the overall performance of the lagoon system. Table 4.7 was established by comparing tables 4.1 and 4.6 and it shows the average reduction of BOD, COD, TSS and ammonia by the reactions in the primary pond. The average influent unfiltered BOD was reduced by nearly 80% and the effluent quality would comply with SEPA's consent for effluent filtered BOD without the need for secondary treatment. Furthermore, the average effluent TSS removal in the primary pond was 60% and as with the BOD, the average effluent TSS can pass the SEPA's consent for TSS (table 4.4). The primary pond average effluent unfiltered COD is 128 mg/l which is slightly higher than the limit set by the UWWTD for maximum filtered COD (125mg/l). The average pH of the mid lagoon effluent is 7.5 and this is within the limits set by SEPA. Thus based on the current population load, the primary pond alone is able to produce an acceptable final effluent.

Table 4.7 Primary pond performance

Parameter	Reduction %
BOD (mg/l)	79.6
COD (mg/l)	59.3
TSS (mg/l)	59.8
Ammonia (mg/l)	33.5

4.3.2 Secondary pond performance

The secondary pond has only one wind-powered processor as it receives a lower loading than the primary lagoon. Table 4.8 was established by comparison between tables 4.6 and 4.5 and it shows the average reduction % in BOD, COD, SS and ammonia by the reactions inside the secondary pond. As expected the performance of the secondary pond with regard to the percentage reduction of BOD, COD and ammonia is not as good as the primary pond performance. It is recognised in sewage treatment that the rate of biological removal of a pollutant is usually proportional to the concentration remaining. Thus in the secondary pond where very little pollution remains, the removal rate would be expected to be low. By contrast the removal of suspended solids is a purely physical phenomenon and thus the removal rate should be constant in both ponds. Indeed it was observed that the TSS reduction in both lagoons was comparable (table 4.8).

Table 4.8 Secondary pond performance

Parameter	Reduction %
BOD (mg/l)	52.8
COD (mg/l)	33.3
TSS (mg/l)	52.6
Ammonia (mg/l)	22.2

4.4 System Performance (organic)

The design objective of the Aero-fac system, indeed for all wastewater treatment plants, is to achieve final effluent BOD concentrations that fall within consent standards. Because of this it is important to consider those factors that might affect BOD reduction efficiency and the amounts of BOD removed. In the following sections the effect of hydraulic loading, surface BOD loading and hydraulic retention time on the BOD reduction percentage and the amount of BOD removed is examined.

The effect of the influent flow rate (effectively the pond hydraulic retention time) on BOD removal was established (figure 4.1) and it is clear that performance increases at hydraulic loading rates below 0.03 m^3/m^3 d (a retention time of 33 days). However the relationship is not one that lends itself to analysis and there is much scatter with the data. Above 0.03 m^3/m^3 d performance is consistent with around 50% BOD removal achieved.

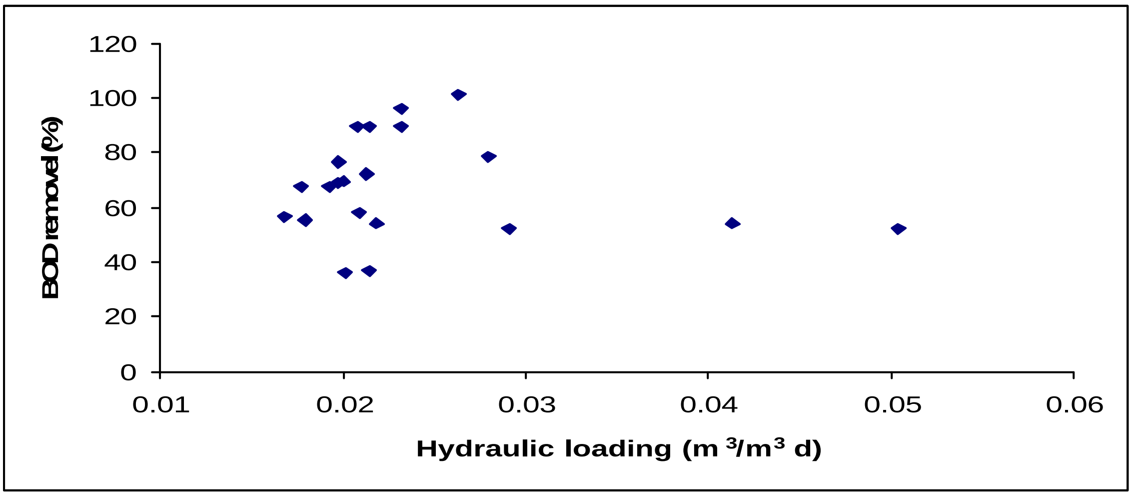

Figure 4.1 Effect of hydraulic loading on BOD removal (%)

The information in figure 10 was replotted in terms of the hydraulic retention time (hrt) and this ranged from 38 to 75 days with the BOD removal increasing as the hrt increased (figure 4.2). Ideally experiments to derive such relationship should be undertaken at steady-state conditions with around three retention times allowed for stability. Clearly this is not possible in full-scale studies where the influent flow rate varies every day. However the flow values were derived from weekly average values in order to remove some of the variability in the flow data, and as a result the number of data points has also reduced.

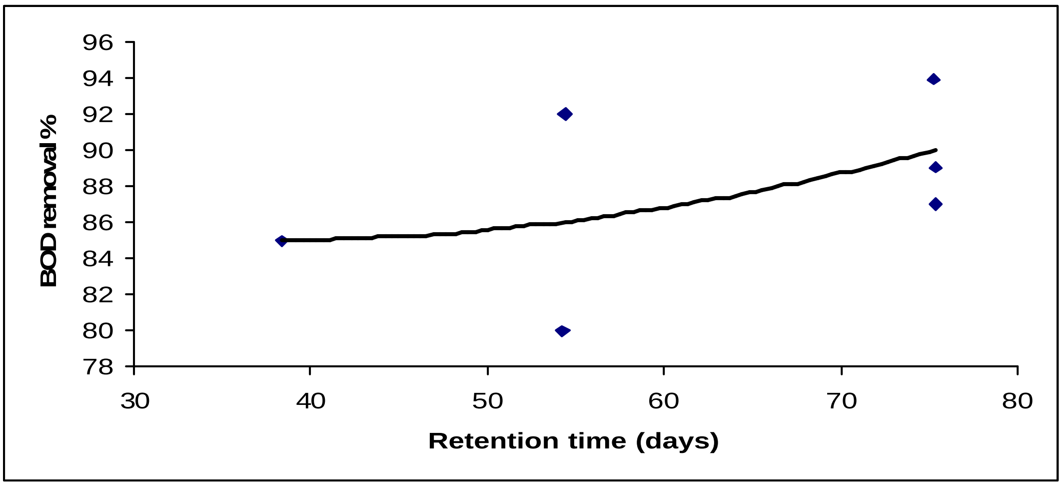

Figure 4.2 Effect of hydraulic retention time on the BOD removal (%)

When the effects of the BOD concentration of the influent (in other words the organic load) are considered (figure 4.3) over a range of surface loadings from 0.004 to 0.018

kg BOD/m^{2}d, then there is no clear relationship between the amount of BOD removed and the loading to the lagoon. The maximum removal was around 98% and this was achieved at loadings between 0.01 and 0.018 kg BOD/m^2 d.

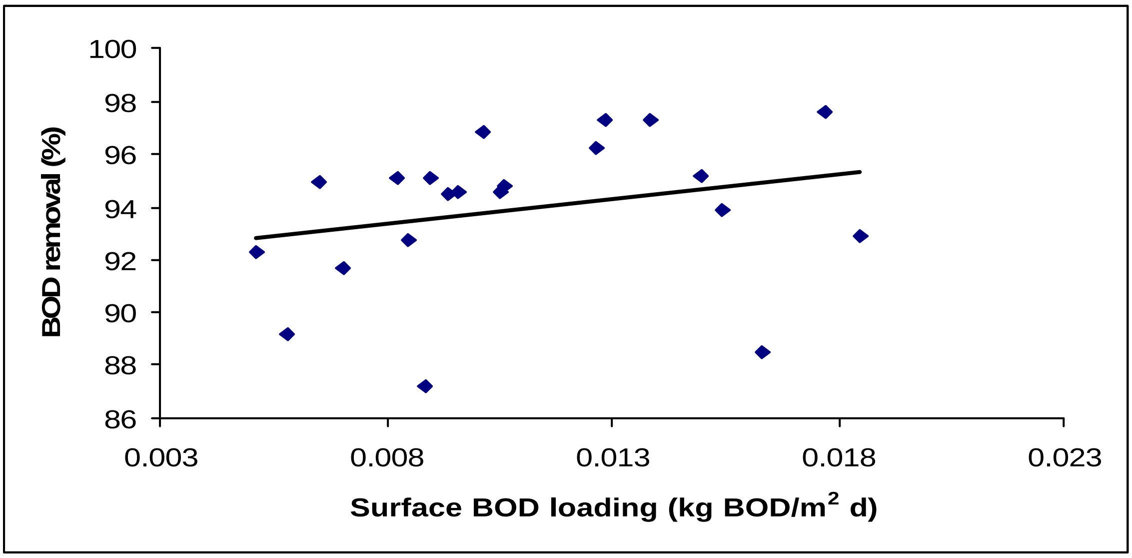

Figure 4.3 Effect of surface BOD loading on the BOD removal (%)

The relationship between the hydraulic retention time and the amount of BOD removed (mg/l) was plotted in figure 4.4. Similarly to the BOD removal % in figure 4.2, the amount of BOD removed (mg/l) increased as the hrt increased. The highest BOD removed value was 130 mg/l and this was when the retention time was 75 days. Once more, the flow volumes used to derive the hydraulic retention time were derived from weekly average values in order to remove some of the variability in the flow data and as a consequence the number of data points has also reduced.

Figure 4.5 shows the influence of the surface BOD loading on the amount of BOD removed. It is clear from the scattered points that between the surface loadings of 0.003 and 0.018, no specific relation can be determined. However, the trend line constructed between the scatter shows that the BOD removed (mg/l) increases when the surface BOD loading increases.

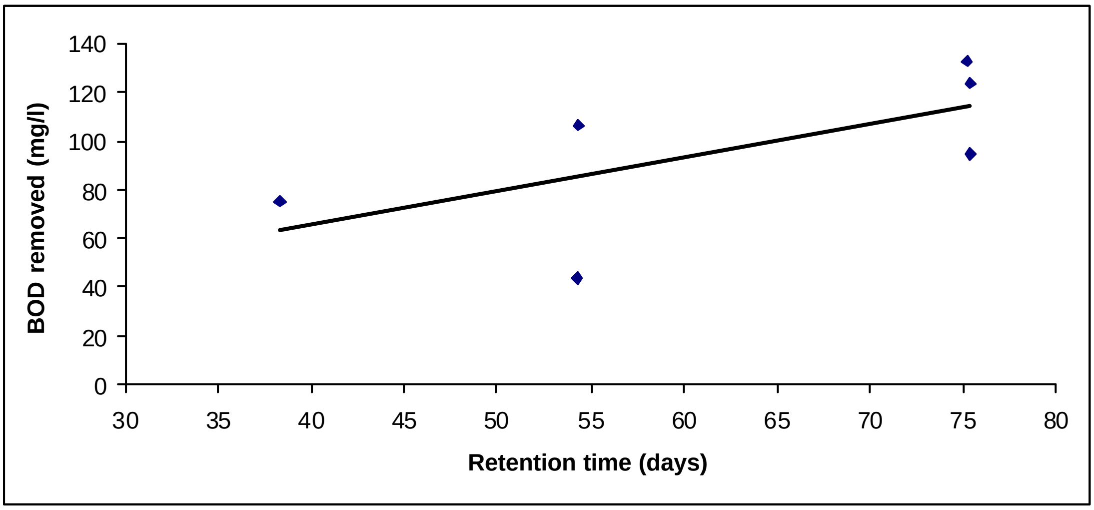

Figure 4.4 The effects of increasing the hydraulic retention time on the amount of BOD removed

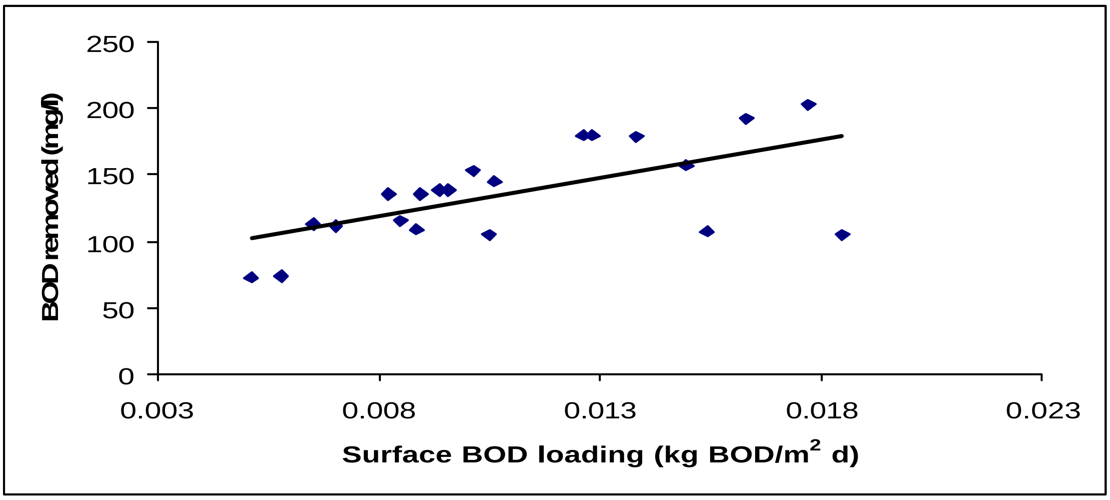

Figure 4.5 Effect of surface BOD loading on the amount of BOD removed

4.5 System Performance (ammonia removal)

One of the main objectives of the Aero-Fac system design was to achieve reductions in ammonia through reducing the overall BOD in the first lagoon and providing a well mixed environment in the second lagoon (NoSWA, 2001). On average; the primary lagoon reduced the influent ammonia concentration by 4.9 mg/l (33%) whilst the secondary lagoon reduced incoming ammonia by 2.2 mg/l (22% comparing final effluent to mid lagoon). Thus, 69% of the amount of ammonia removed was reduced by the reactions in the primary lagoon and the average ammonia removal for the whole system was 48% (comparing final effluent and influent).

The average concentration of nitrate is slightly reduced in the primary lagoon which suggests either that denitrification is occurring or that ammonia removal is not caused by nitrification. However it then increases in the secondary lagoon (figure 4.7). Comparison between figure 4.6 and 4.7 would suggest that the ammonia concentration is reduced by nitrification in the secondary lagoon, but not in the first lagoon because the average level of nitrates only increases in the second lagoon. Possible ways of ammonia loss in the primary lagoon could be volatilisation (loss as a gas to the atmosphere) or, most probably via assimilation into algal biomass. Volatilisation of ammonia is possible in the first lagoon because the lowest mid-lagoon pH reading is 6.9, slightly higher than pH value of 6.6 above which ammonia starts to convert to a gaseous form.

Figure 4.8 shows the average chlorophyll$_a$ in the influent, mid-lagoon and the final effluent. The average chlorophyll$_a$ rises from 0 in the influent to an average of 32.2 (µg/l) in the mid lagoon and then reduces to 7.02 µg/l in the final effluent. This provides support for the idea that ammonia is mainly lost via assimilation into algal biomass in the first lagoon because the rise in chlorophyll a concentration in the first lagoon is accompanied by a corresponding loss of ammonia.

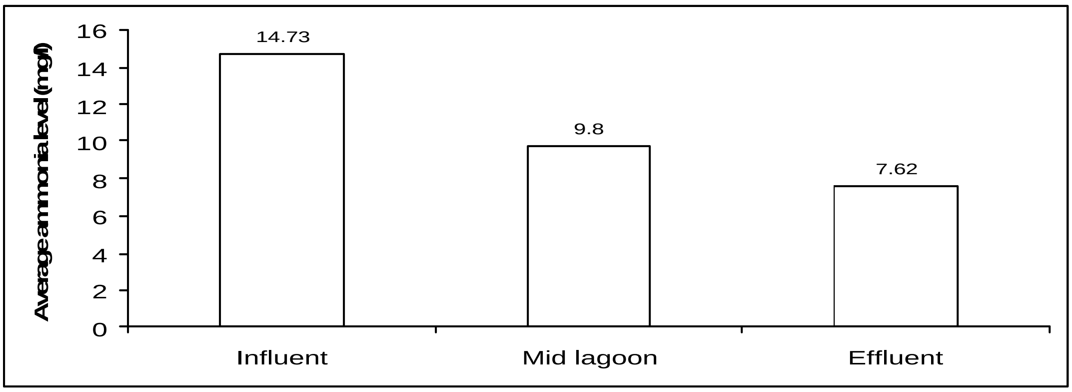

Figure 4.6 Average ammonia in the influent, mid-lagoon and final effluent

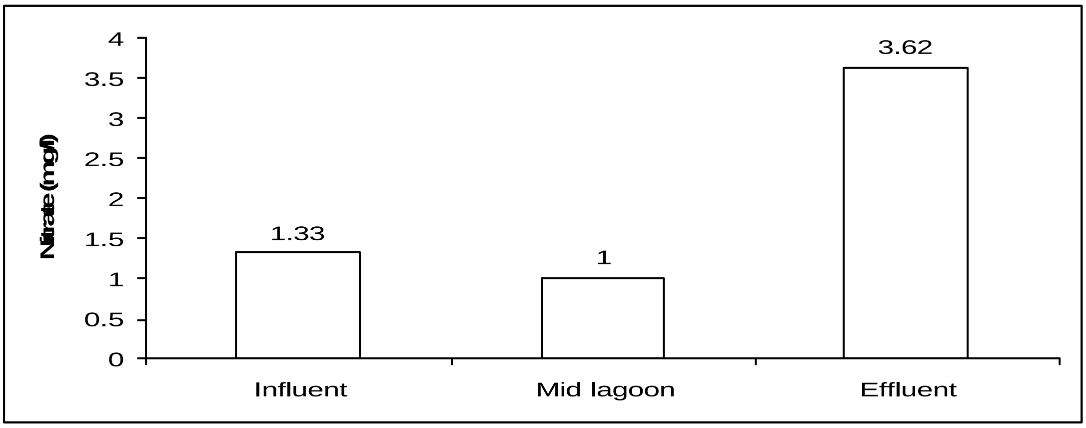

Figure 4.7 Average nitrate in influent, mid-lagoon and final effluent

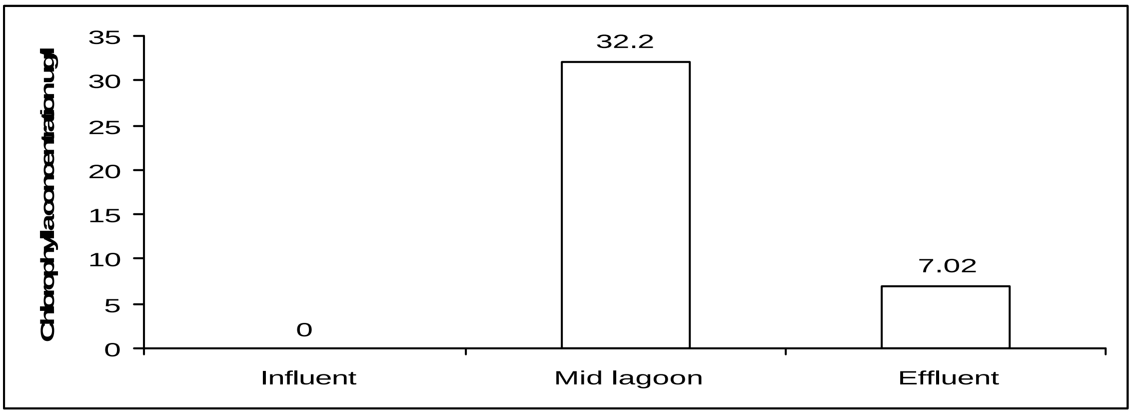

Figure 4.8 Chlorophyll$_a$ concentration in influent, mid-lagoon and final effluent

4.5.1 Final effluent ammonia concentration and ammonia loading

Ammonia removal is known to be closely linked to temperature and pH with the required retention time increasing as temperature decreases and when the pH falls outside of the range 6.6 to 8.2. It is independent of the initial ammonia concentration up to a value of around 50 mg/l, however, it is thought to be highly dependent on the ammonia load. This latter relationship has been confirmed at Errol (figure 4.9) where the effluent ammonia concentration increased in response to the increasing ammonia load. Clearly this correlation would be breached at loadings above 18 kg ammonia /d.

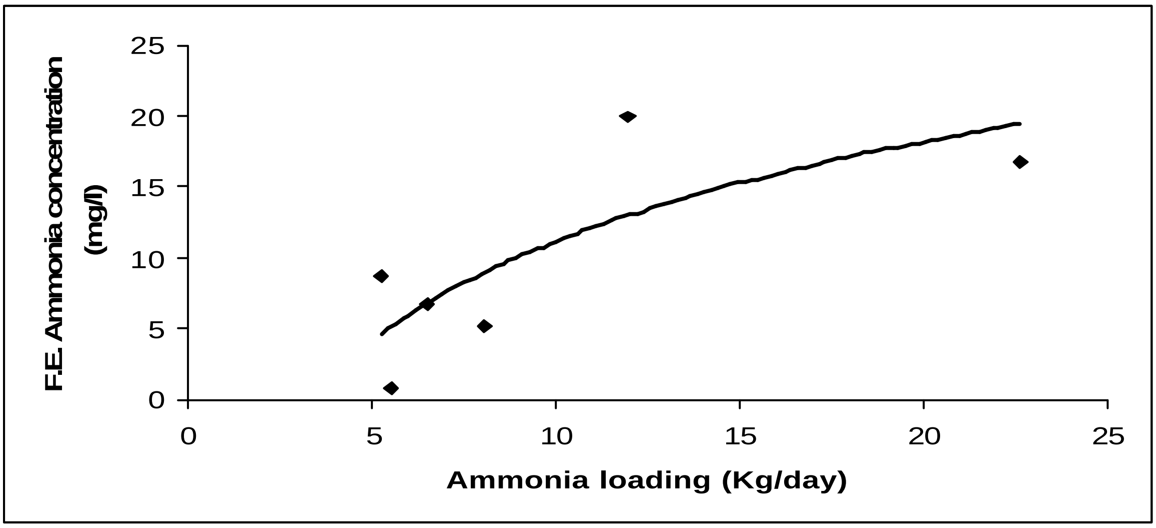

Figure 4.9 The influence of the ammonia loading on final effluent ammonia concentration

4.5.2 Ammonia removal efficiency and retention time

Of course the ammonia loading relates to both the flow and the ammonia concentration and figures 4.10 and 4.11 show that the ammonia removal efficiency is also linked by the system retention time, an observation found by many researchers (Mara *et al*., 1987).

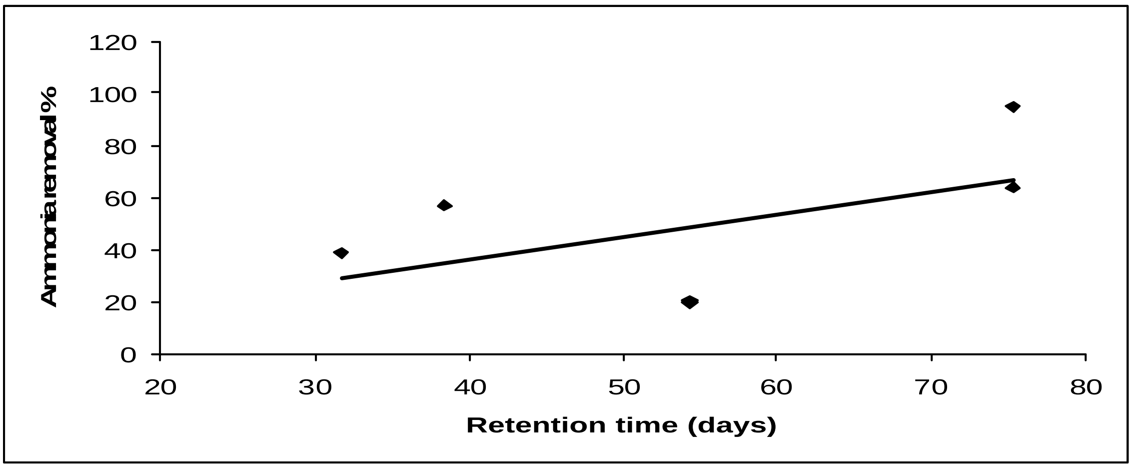

Figure 4.10 The effect of retention time on ammonia removal (%)

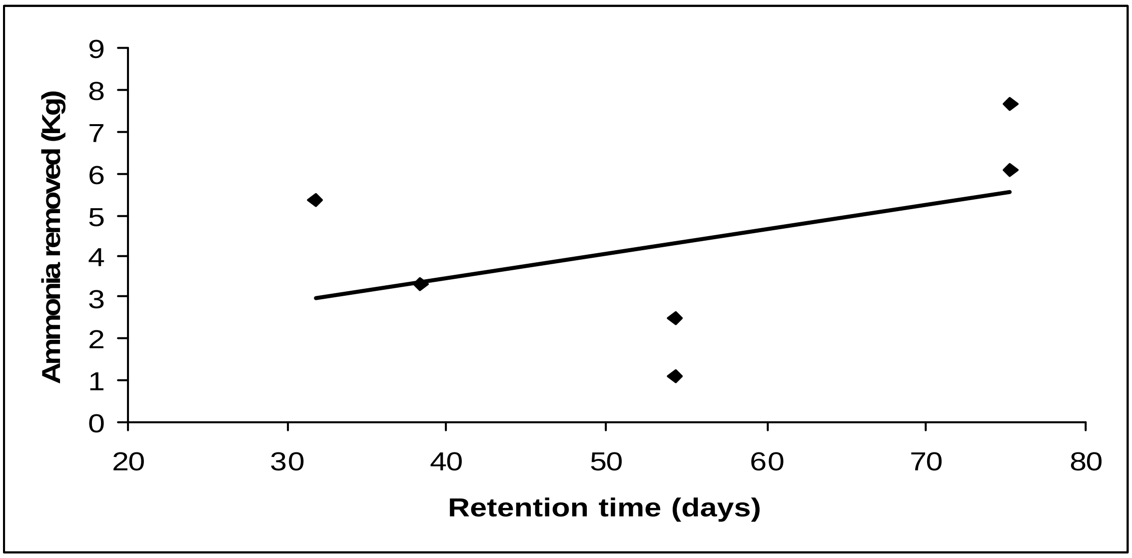

Figure 4.11 The effect of retention time on the amount of ammonia removed

4.6 Seasonal Variation in BOD and Ammonia Removal Efficiency

Seasonal changes in weather would be expected to have a large effect on the performance of the Aero-fac system as they bring changes in light intensity, temperature, flow and BOD concentration. This effect has been investigated using the analytical data obtained by Scottish Water during the lagoon performance tests together with the data obtained during the course of this study. It is clear from figure 4.12 that there are changes in performance of the Errol lagoon but they do not follow a clear seasonal pattern. The worst performance was in spring and autumn 2002 and there has been little difference between summer and winter performance in any one year. This suggests that the contribution of algal photosynthesis to satisfying the oxygen demand of the sewage is negligible and the majority of oxygen is derived from the aeration system.

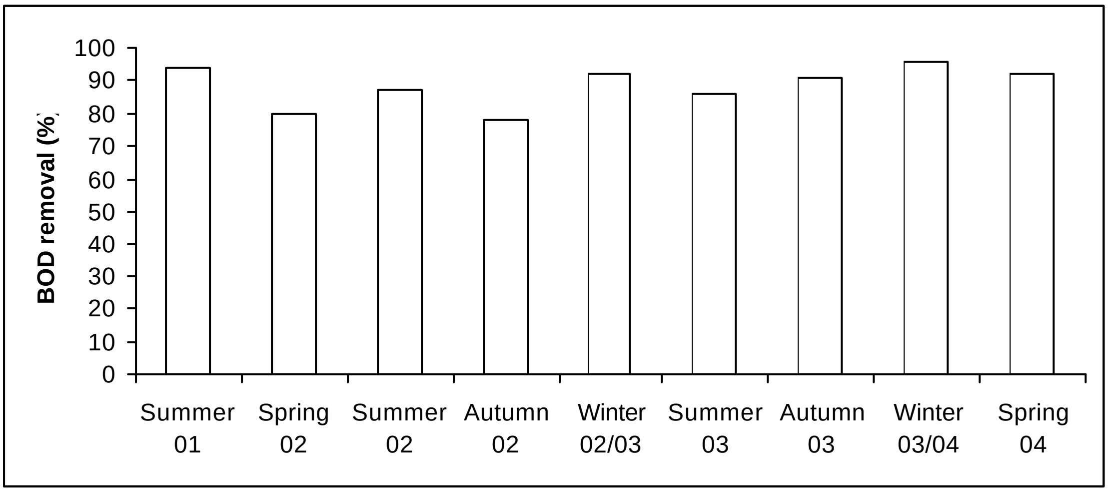

Figure 4.12 Seasonal performance of the lagoons in terms of the percentage BOD removal

By contrast the removal of ammonia is clearly seasonal with much higher removal rates during the summer period. This contradicts the evidence that the mechanism for ammonia removal is by algal uptake and not nitrification. However, there are a lot of reports to the effect that nitrification does not occur in systems which do not have cell recycle (such as aerated lagoons and waste stabilisation ponds) (Horan, 1991) and it would seem that the same applies to the Aero-Fac lagoon.

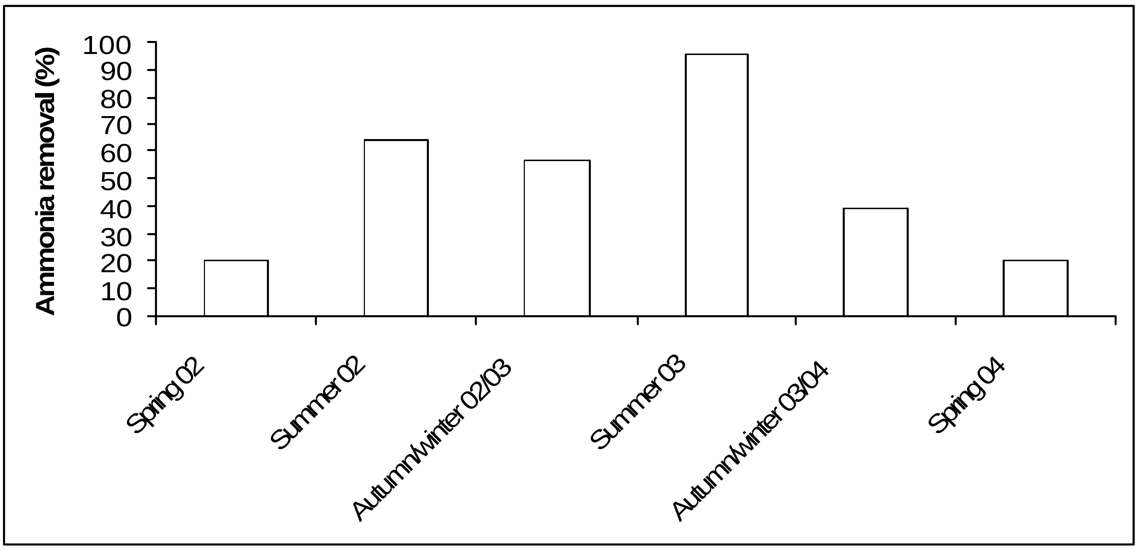

Figure 4.13 Seasonal variations in the removal of ammonia

4.7 Predominant Algal Populations

The contribution of algae in a lagoon system is twofold with both beneficial and detrimental effects. On the positive side algae are said to play an important role in facultative lagoons by generating oxygen through photosynthesis to be used by the aerobic bacteria in breaking down the BOD. A synergistic relationship exists between bacteria and the algae whereby in the daylight, algae produce oxygen for the aerobic bacteria to use and conversely the carbon dioxide produced by bacteria is used for algal growth. In addition the presence and growth of algae in the Errol Aero-Fac facultative lagoons may also be associated with a loss of ammonia as shown earlier.

However the detrimental effects come from deoxygenation when light is not available as both aerobic bacteria and algae compete for oxygen to oxidise organic material matter and obtain energy. In addition the presence of algae can result in more solids in the final effluent and thus compromise final effluent quality. Finally in severe cases production of floating algal mats in the summer months may generate odour problems. Consequently it was thought important to understand the seasonal growth patterns of algae in the lagoon.

4.7.1 Algal species

Quantification of the amount of algae in both lagoons, as well as knowledge of the diversity of the algal population, was undertaken as it was thought this information would be helpful in assessing performance of the Aero-fac system and identifying any potential problems before they became too troublesome. Algae were sampled at mid-lagoon and in final effluent samples and it was noted that they were all non-motile. This is most likely due to the fact that the mixing regime in the lagoons does not allow motile algae any competitive advantage in receiving sunlight and thus energy expended on motility is wasted.

4.7.2 Seasonal variation in the algal species and numbers

Algae are particularly sensitive to sunlight and thus would be expected to show a clear seasonal variation and this was indeed the case at Errol. *Scenedesmus* was the most frequently observed algae throughout the year except for winter with *Pedistrium* closely behind during spring and summer. However *Pedistrium* completely disappeared during autumn and winter. It is clear from the table that diversity was highest in spring and summer with only a negligible population during the winter. Tables 4.9 and 4.10 show the identified algal species in mid lagoon effluent and final effluent during the year's four seasons (summer, autumn, winter and spring) and figures 4.14 and 4.15 illustrate the variation in algal biomass in the mid lagoon effluent and the final effluent during these seasons. The tables and graphs highlight that maximum algal diversity and quantity occur during the summer period.

Table 4.9 Identified algal species in the mid-lagoon effluent at different seasons

Period	**Identified algae**	**Frequency**
Summer	Blue green	Low
	Chlorella	High
	Colestrum	Low
	Diatoms	Low
	Oocystis	Low
	Pedistrium	2nd most frequent
	Scenedesmus	Most frequent
Autumn	Blue green	Low
	Chlorella	High
	Oocystis	Low
	Scenedesmus	High
Winter	*Chlorella*	Low
Spring	*Ankistrodesmus*	Low
	Chlorella	High
	Oocystis	High
	Pedistrium	Most frequent
	Scenedesmus	High

Table 4.10 Identified algal species in the final effluent at different seasons

Period	Identified algae	Frequency
Summer	Blue green	Low
	Chlorella	Medium
	Cladphova	High
	Diatoms	Low
	Nitzshia	Low
	Oocystis	High
	Pedistrium	Most frequent
	Scenedesmus	Medium
Autumn	Blue green	Low
	Chlorella	Low
	Cladphova	Low
	Pedistrium	Low
	Scenedesmus	Low
Winter	*	*
Spring	*Chlorella*	High
	Oocystis	Most frequent
	Pedistrium	High

***** No algae were found

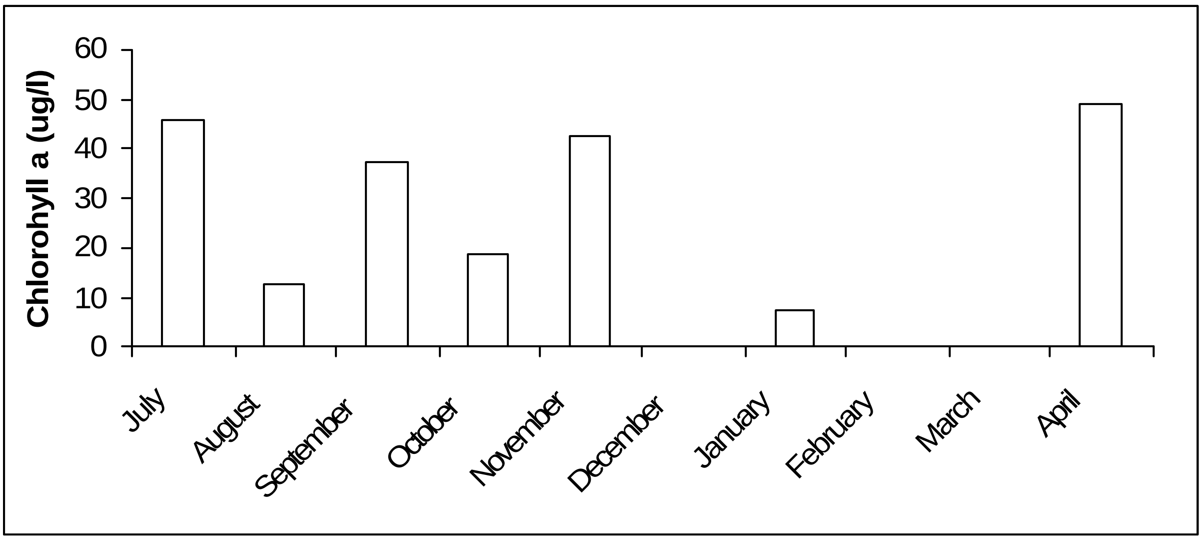

Figure 4.14 Algal biomass variations during the year (mid lagoon)

The algae in the final effluent were also highest during summer and this may account for the higher suspended solids. Figure 4.16 shows the seasonal suspended solids concentration in Errol final effluent.

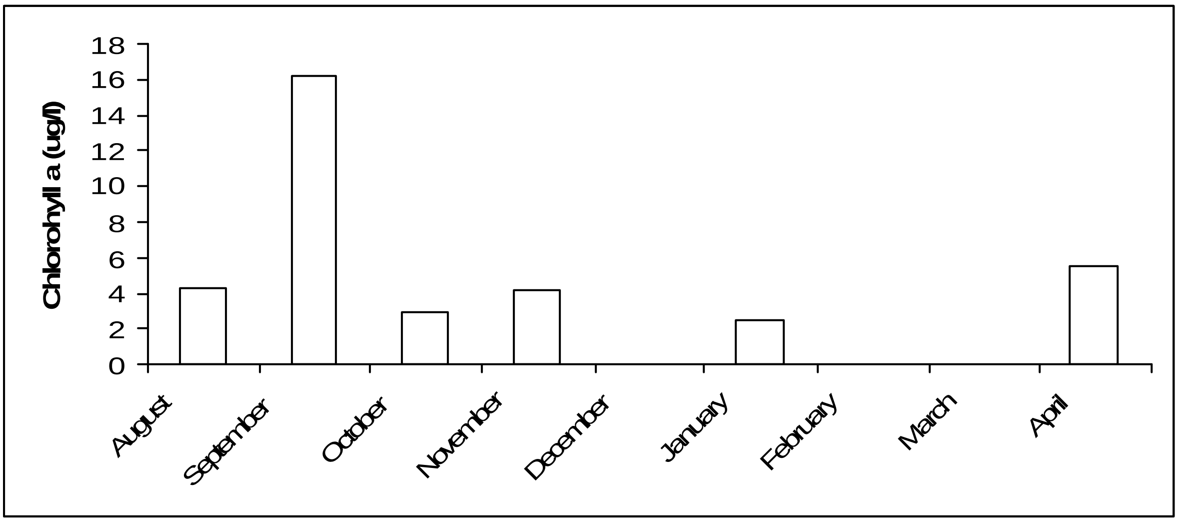

Figure 4.15 Algal biomass variation during the year (final effluent)

Comparison between Figures 4.15 and 4.16 shows clearly that the presence of algae is a major contributor to the suspended solids concentration in the final effluent. It also helps to explain why the pond effluent is always in compliance for BOD but fails the COD standards. Algae will contribute a large COD but a negligible BOD as they are unlikely to degrade during a five day period. This allows the possibility of adapting simple tertiary treatment options (such as a rock filter) (Johnson and Mara, 2002) to polish the final effluent if a COD consent is imposed.

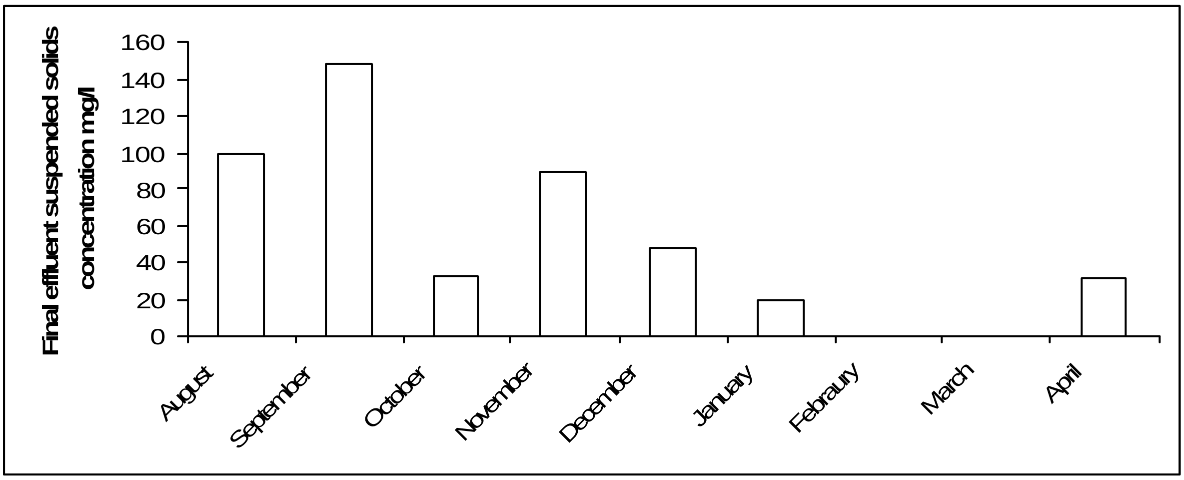

Figure 4.16 Seasonal final effluent suspended solids concentration (mg/l)

4.8 Sludge Accumulation

The performance of the Errol Aero-fac facultative lagoon with regard to sludge digestion was of particular interest to Scottish Water due to the high sludge disposal costs and the limited options available for disposal. Consequently a series of experiments was undertaken in an attempt to understand the deposition and build up of sludge through the lagoon system.

4.8.1 Sludge depth measurement results

Sludge depth measurement on the primary lagoon showed that there was noticeable sludge accumulation only in the area around the inlet of the primary lagoon where it had accumulated to around 25 cm. There was no sludge accumulation greater than 2mm at any point in the rest of the primary lagoon. Sludge depth measurement in the second lagoon was similar with a maximum accumulation of 2mm at any point. Indeed sludge was absent from many parts of the lagoon (Fig 4.17).

In view of the small amount of sludge that had accumulated, accurate measurement was obviously difficult. Consequently it was verified in two ways: measurement of sludge on the same spots by two different people and measurement of the water depth and comparing it with the design water depth. Both methods gave good duplication and provided some confidence in the sludge accumulation data.

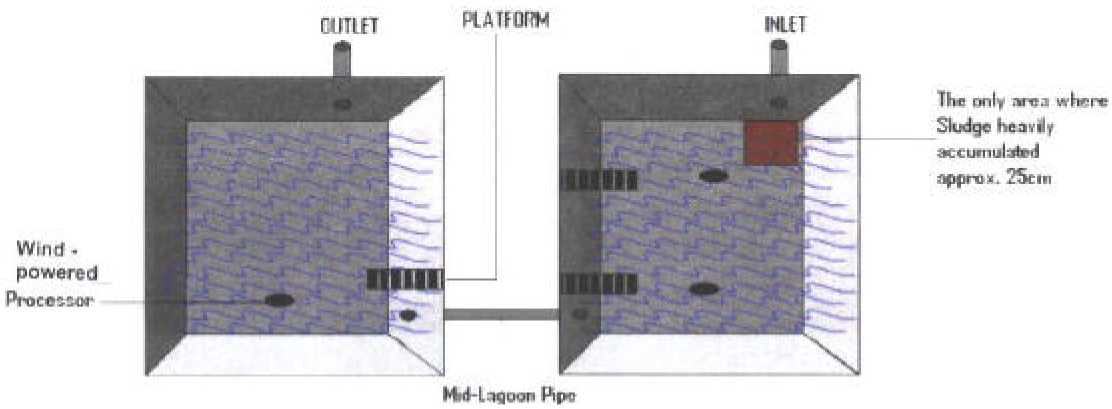

Figure 4.17 Sludge accumulation in Errol Aero-Fac lagoons

4.8.2 Accumulated sludge volume and mass

The total volume of the accumulated sludge in the two lagoons was calculated based on the area and depth measured in those areas in which it had accumulated. The heaviest accumulation where the depth was 25 cm, occupied an area of 25 m^2 and for the rest of the primary and secondary lagoons an overall accumulation of 2 mm was assumed. This gives a volume of 6.25 m^3 sludge in the area where accumulation was at its heaviest and 14 m^3 over the rest of the lagoon area, a total volume of 20.5 m^3. The mass of this volume was estimated using a sludge density of 1003 kg/m^3 to give 2,051 kg sludge accumulated.

Considering the average influent total suspended solids, the final effluent total suspended solids, the average influent flow and the average effluent flow; the total mass of the solids entering the lagoon ponds every day was 82 kg and so over 2.5 years the lagoons have received 74,500,000 kg. Thus taking into account the theoretical available mass of solids, but ignoring any conversion of solids from BOD breakdown, then the lagoons have destroyed 99.99% of the solids entering and accumulated only 0.01% of available sludge.

4.8.3 Future accumulations

Assuming that the population of Errol will increase by an average of 45.7 people per year to reach the maximum design population of 2000 in 2021 and considering that a population of 1200 produced sludge accumulation of 25cm around the inlet and 2mm in the rest of the lagoon in 2.5 years then:

i. The assumed sludge accumulation from a population of 1200 in one year is 25/2.5 + 0.2/2.5 = 10 cm around the inlet and 0.08 cm over the rest of the two lagoons.
ii. The assumed per person sludge accumulation in one year is 10/1200 + 0.08/1200 = 8.33 x 10^{-3} cm around the inlet and 6.667 x 10^{-5} cm over the rest of the two lagoons.

iii. Estimated sludge accumulation after 20 years by the initial population of 1200 is (10 x 20) + (0.08 x 20) = 200 cm around the inlet and 1.6 cm over the rest of the two lagoons.

iv. Assuming in the next 17.5 years there will be an average annual population rise of 45.7 so the population of Errol reaches 2000; the corresponding sludge accumulation is 61 cm around the inlet and 0.5 in the rest of the two lagoons.

v. Thus, the total estimated sludge accumulation after 20 years of operation is 261.4 cm around the inlet and 2.1 cm over the rest of the lagoon.

vi. Desludging should be undertaken when the sludge depth is more than half the mean depth of the water and consequently some desludging may be required around the inlet after 15 years from operation. This will require removal of around 60 to 75 m^3 sludge.

4.9 Retention time

As with all waste stabilisation ponds the Aero-Fac facultative lagoon can be considered as a chemical reactor. The boundary condition of this reactor is best described as open. This means the volume of flow will affect the magnitude of the retention time. For example, a high daily flow will mean the wastewater will be retained for a short period of time in the system, while a low daily flow, will mean the wastewater will remain for a longer period. But, the hydraulic behaviour inside the lagoon will also affect the hydraulic retention time. The random fluctuations in which fluid elements overtake and mix with one another within the lagoons can reduce or increase the retention time. These fluctuations are governed by a number of measurable and indeterminate factors such as the pond dimensions, aerator power, position of the inlet and outlet structures, wind velocity and prevailing direction, dead spaces, short circuits and temperature (Nameche and Vasel, 1998).

According to the Aero-Fac® design guidelines, the minimum retention time of each lagoon in cold climates should be 15 days. The design of the Errol Aero-Fac lagoons for the existing population was based on a retention time of 25.3 days in each lagoon, reducing to a design retention time of 15.3 days in each lagoon at full design population. However lagoon systems always have actual retention times which are far less than design due to short circuiting and dead spaces in the lagoon that receive no mixing. Consequently tracer studies are usually employed to determine the exact retention time and the effect of random fluctuations of fluid movements.

4.9.1 Tracer study

Following the dosing of Rhodamine WT into the inlet and the mid lagoon pipe, fluorescence was determined in mid lagoon and final effluent samples (table 4.11). The table clearly shows that the highest fluorescence in mid lagoon samples was in the sample taken after 16 days whereas the highest fluorescence detected in the final effluent samples was after 17 days.

Based on available fluorescence data plus assumptions; the flow dispersion was calculated for the primary and secondary lagoons using the method of Tomlinson and Chambers (1979):

I. Primary lagoon mean residence time is $? C_t t \, / \, ? C_t$ = 14.26 days.
II. Secondary lagoon mean residence time is $? C_t t \, / \, ? C_t$ = 14.92 days.
III. Normalised time = Time / Mean residence time
IV. normalised concentration $= \dfrac{? \text{ (resultant concentration * } ?t)}{\text{Mean residence time}}$
V. Where ?t is the time interval (1 day).
VI. Primary lagoon normalised rhodamine concentration = 2.91
VII. Secondary lagoon normalised rhodamine concentration = 3.35
VIII. Concentration = Resultant concentration / Normalised concentration
IX. Primary lagoon variance = $?^2 = \dfrac{? \text{ Concentration * normalised time}^2)}{\text{Concentration}}$ = 0.142.
X. Primary lagoon dispersion number = 0.142/2 = 0.071. This is indicative of only a small amount of dispersion in the primary lagoon.
XI. Secondary lagoon variance = $?^2 = \dfrac{? \text{ Concentration * normalised time}^2)}{\text{Concentration}}$ = 0.116.

XII. Secondary lagoon dispersion number = 0.116/2 = 0.058. This is also indicative of a small amount of dispersion in the secondary lagoon. Thus in both cases the lagoons do not behave at all like a completely mixed reactor although the aeration system was predicted to provide a large amount of mixing and thus high dispersion.

Table 4.11 Fluorescence detected by the fluorimeter in mid lagoon and final effluent samples.

Day	Mid lagoon Fluorescence	Final effluent Fluorescence
15	7.14	4.97
16	7.6	4.97
17	4.95	4.97
18	3.775	4.97
19	0.9	2.35
20	0.9	0.9
21	0.9	0.9
22	0.9	0.9
23	0.9	0.9
24	0.9	0.9

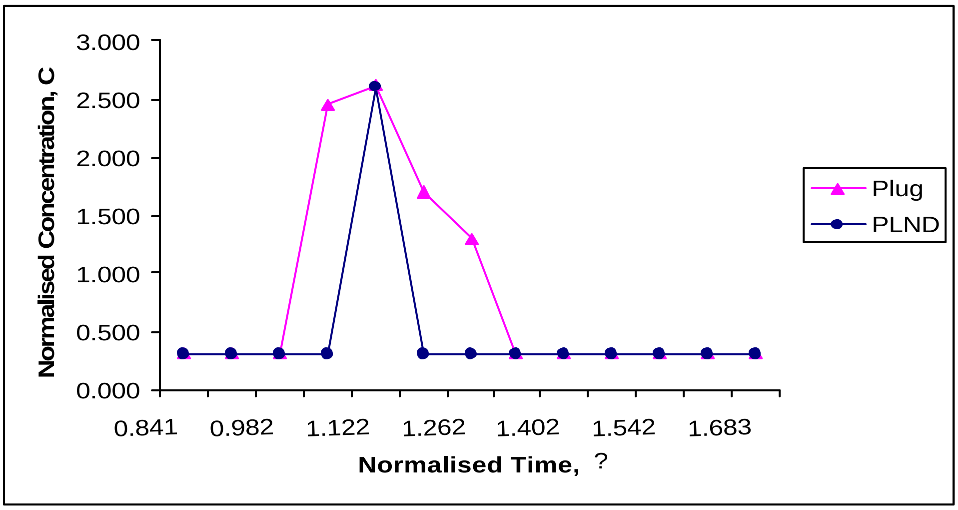

Figure 4.18 Normalised retention time distributions (Primary lagoon)

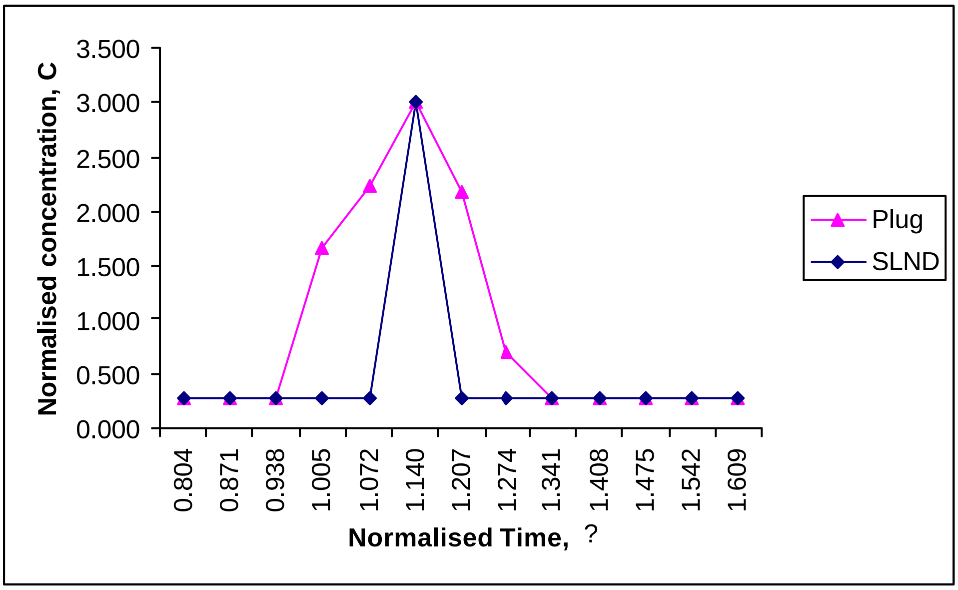

Figure 4.19 Normalised retention time distribution (secondary lagoon)

Figures 4.18 and 4.19 show the normalised retention time distribution of the primary lagoon (PLND) and the secondary lagoon (SLND) along with the expected distribution with a plug flow. It is clear from the difference between the graphs in the two figures that there is only a small deviation from the ideal plug flow.

4.10 Removal of Bacterial Indicators

The ability of the Aero-Fac system to remove pathogens was determined by testing influent and effluent samples for the presence of total coliforms and faecal coliforms. Tables 4.12 and 4.13 show the results of the Bacteriology tests carried immediately after the opening of Errol wastewater treatment plant and after 2.5 years from the opening of the plant.

The lagoons showed similar removals for total coliforms and faecal coliforms with greater than 2-log achieved in all cases. This is not a particularly good rate of removal and a conventional activated sludge plant would achieve similar effluent quality. The poor removal is likely to result from the cold climates with reduced algal activity. Clearly additional tertiary disinfection would be required if it was necessary to meet the requirements of the EC Bathing Water Quality Directive.

Table 4.12 The log numbers of total coliforms and faecal coliforms in the Errol influent and effluent (2001)

Date	Crude		Final		Log removal	
	Coliforms	Faecal	Coliforms	Faecal	Coliforms	Faecal
17/10/2001	7.3	5.88	5.22	2.86	2.08	3.02
18/10/2001	7.3	6.2	5	2.63	2.3	3.57
19/10/2001	6.58		4.92	3.88	1.66	
22/10/2001	6.51	5.26	4.92	3.36	1.59	1.9
24/10/2001	6.28	4.53	5.08	3.18	1.2	1.35
Average	6.794	5.4675	5.028	3.182	1.766	2.2855

Table 4.13 The log numbers of total coliforms in the Errol influent and effluent (2004)

Date	Crude	Final	Log removal
14/04/2004	8.15	4.00	4.15
16/04/2004	7.30	4.64	2.66
18/04/2004	5.20	3.26	1.94
20/04/2004	7.00	4.11	2.89
Average	6.91	4.00	2.91

4.11 Aero-Fac system costs

The capital costs for the Errol Aero-Fac system construction was £1,650,000 which includes the cost of constructing the two lagoons, aerators, diffusers, pumping station, control room and the pipeline from Errol to the estuary. This capital cost equates to £1,375 per person which is extremely high in comparison to the capital costs per person of different sewage treatment options for population less than 2000 in the UK. For instance Mara (1998) has quoted £40-100 for reed beds, £400-1,000 for Rotating Biological Contractor and £333 for a waste stabilisation pond. Furthermore, the operating costs over twenty years were estimated as shown in table 4.14 and thus the total scheme costs after 20 years of operation is estimated to be £2,341,000 (2001 prices). The annual operation costs of other systems used by Scottish Water for similar populations to Errol includes; £18,281 for ST Fillans WWTW, £26,819 for

Pitlochry WWTW and £18,186 for Comric WWTW. These annual operation costs are all lower than Errol annual operation costs, but it must be stated that Errol's exceptionally high business rates were the main reason for the high operation costs. The annual per person operating costs for Errol were estimated to be £29 if the population of Errol stays at 1200, but the per-person cost will reduce when population rises.

Table 4.14 Estimation of the annual operation cost

Description	**Quantity**	**Rate (£)**	**Total cost (£)**
1/ Land rental business rate	1	30,000	30,000
2/ Manual operation			
(a) travel time at 1 hour per week	52	12.00	624.00
(b) Operation time on site at 1 hour per week	52	12.00	624.00
3/ Maintenance			
(a) travel time at 1 hour per week	52	12.00	624.00
(b) maintenance time on site at 1 hour/week	52	12.00	624.00
4/ Materials			
(a) lagoon equipment parts	1	55.00	55.00
(b) pumps parts	1	250.00	250.00
5/ Power (electricity)	1	1750.44	1750
Total operation costs per annum			**34,551**

CHAPTER 5: DISCUSSION

5.1 Influent Wastewater Characteristics

As mentioned in section 4.1, the sewage received at Errol is very weak compared to a typical UK domestic sewage. By contrast the received flow was higher than the predicted flow and in combination these two factors led to a load that was lower than predicted. The contributing factors to explain this are likely to involve high rainfall, high infiltration into the sewers and undersized CSOs which overflow routinely and thus contributed to the loss of load.

The average influent COD:BOD ratio was 3.4:1 and can be described as high for this type of sewage and may be indicative of a large non-biodegradable fraction in wastes which may also help to explain the high effluent COD values. With regard to the guaranteed consents for influent BOD and COD; all influent BOD results passed the consent agreed for BOD (400mg/l), but, several influent COD results illustrated failure as they were above the consent limit for COD (800mg/l) and even exceeded 2000 mg/l in some cases. Bucksteeg (1987) has shown that combined sewerage with runoff from roads of rural villages may have COD concentrations of more than 1000mg/l. Thus, relating the high influent COD at Errol to the combined nature of the sewerage could be a good explanation.

Based on the received sewage characteristics from an estimated population of 1200, the future sewage characteristics from the design population of 2000 were predicted. Comparison between the current sewage characteristics and predicted future characteristics showed there is a possibility that there will be a reduction in the Aero-Fac system performance due to the expected rise in flow and load.

5.2 Effluent Quality

All final effluent (filtered and unfiltered) BOD passed the SEPA and the performance guarantee consents with 95% confidence. The average removal rate of unfiltered BOD was 89% and that is higher than the typical reductions targeted by the system designers (80%) and thus the current performance of the system with regard to BOD removal can be described as excellent. Furthermore, the performance of the primary lagoon was found to be sufficient in most cases to produce a within standards effluent considering the current population, and thus this could acknowledge the practicality of placing a cheaper treatment option instead of the secondary lagoon, especially knowing that most load reductions take place in the primary lagoon.

In her study of the performance of facultative lagoons in the UK, Abis (2002) concluded that BOD removal was mostly affected by the BOD loading. With regard to Errol Aero-Fac evaluation, the BOD removal efficiency was shown to be affected by the BOD loading and the retention time, though only limited retention time data were available to precisely judge the relationship. Seasonal changes and influent flow volumes changes were found not to have an effect on the BOD removal efficiency.

The average final effluent unfiltered COD complied with the UWWTD filtered COD consent, but the 95%ile value did not comply as it was the higher than 125mg/l. However, it must be noticed that unfiltered COD values are expected to be higher than filtered COD values and so the system performance with regard to COD removal should not be criticised especially given that the average COD removal was 73%.

The average removal rate of TSS was 81% which is comparable to reduction rates targeted by the system designers (80%) (LAS International Ltd, 2000). The final effluent TSS level passed the SEPA and the performance guarantee limits with 95% confidence. The 95% value was less than half the SEPA limit for TSS (150mg/l). Furthermore, the primary pond effluent also showed compliance with SEPA's consent. However, accelerated growth of algae in the summer was noticed to increase the average suspended solids concentration in the effluent.

All final effluent pH results were within the range set by SEPA and the performance guarantees. Once more, the primary effluent pH also complied with the SEPA and performance guarantees for pH. On average the pH of the mid lagoon effluent was lower than the final effluent pH.

5.3 Ammonia Removal

With regard to ammonia removal, the average removal rate of ammonia by the system was 48% with 69% removed in the primary lagoon. Furthermore, analysis showed that ammonia removal fluctuated throughout the season (20% to 95%), with the highest removal rates in the summer period. This agrees with the conclusion of most writers on ammonia removal in facultative ponds, including Abis (2002). However, surprisingly, the lowest average rates of ammonia removal were found to occur in the spring period rather than the winter period (Figure 4.13). The higher ammonia removal rates in the summer period is presumably caused by the high temperatures and pH and the longer retention times associated with the likely dry weather flow during this season. While the only explanation for the lowest ammonia removal rates in the spring period could be ice melt that usually occurs in the spring and causes the release of significant amounts of ammonia as a result of anaerobic reactions as well as producing a significant drop in lagoon temperatures (Oleszkiewiez and Sparling, 1987).

The routes of ammonia loss are presumably assimilation in the primary lagoon, possibly nitrification in the secondary lagoon and volatilization in both lagoons. With regard to volatilization; temperature and pH were suggested as the most important factors by König *et al.* (1987). Both König *et al.* (1987) and Ruffier *et al.* (1981) mentioned that a 10 degrees rise in temperature will double the amount of NH_3 in solution, and in solutions of pH < 7.0, ammonia is only present as NH_4 whereas at pH $= 9.2$, half the ammonia is in the NH_3 form. The lowest recorded pH in the Errol lagoons was 6.9 which is nearly equivalent to the minimum pH above which ammonia starts to volatilise. Ammonia removal in the secondary lagoon is evident by a drop in ammonia concentration with a corresponding rise in nitrates concentration, a process presumably executed by the nitrifying bacteria in the lagoon. While the algal population rise in the primary lagoon was assumed to be the reason behind ammonia loss, especially as there was no noticeable increase in nitrate concentration.

The evaluation also aimed to investigate the relation between ammonia removal and retention time (figures 4.10 and 4.11). The figures show that ammonia removal can be linked to the retention time.

5.4 Algal Population

Microscopic identification of mid lagoon effluent samples for algae showed the presence of *Scenedesmus, Chlorella* and *Pedistrium* as dominant populations. The extreme occurrence of *Scenedesmus* and *Chlorella* can indicate there is an obvious presence of ammonia and sulphides as they have good resistance to both chemicals compared to other algae. The large presence of *Pedistrium* might indicate a low solids contents because of their sensitivity to the presence of solids. Furthermore, microscopic identification of final effluent samples for algae showed *Oocystis* and *Pedistrium* were dominant. *Oocystis* are usually found only in aerobic ponds and thus their presence may prove the existence of aerobic conditions in the lagoon, while the large presence of *Pedistrium* indicates low solids content in the final effluent. Blue green algae (*Cyanobacteria)* were identified in both; mid lagoon effluent samples and final effluent samples.

Unexpectedly, all the identified algae were non-motile algae and this presumably results from the mixing in the lagoons which allowed sunlight to reach non-motile algae and thus helped in their ability to photosynthesise and grow. Sukias *et al.* (2003) blamed the reduction in algal numbers in an aerated facultative pond on aeration mixing which made it difficult for algae to maintain their positions in the eutrophic zone and reduced their ability to photosynthesis (Sukias *et al*., 2003). Based on this; motile algae in the Errol lagoons could have lost their competitive advantage because of aeration mixing which frequently moved them from their usual eutrophic zone to lower zones where ammonia and sulphide concentrations are higher and especially known that *Euglena* (the most common motile algae) were found to have very low resistance to ammonia and sulphides (Pearson *et al*., 1987). On the contrary, non-motile algae which usually grow in lower zones in ponds as they cannot move to the surface, gained a competitive advantage through aeration mixing which moved them frequently to the eutrophic zone and gradually reduced the numbers of their competitors the motile algae.

The algal biomass (expressed as concentration of chlorophyll *a*) is higher in the primary lagoon than in the secondary lagoon. This is presumably because of the higher load in the primary lagoon, which offered algae better conditions for growth.

Nevertheless, the average algal biomass in both the primary lagoon (32.2 µg/l) and the secondary lagoons (7.02 µg/l) are much lower than the typical algal biomass range of 500 -1500 µg/l given by Mara *et al.* (1992). As mentioned earlier, this is probably because of aeration mixing which reduced the ability of the algae to photosynthesise by making it difficult for them to maintain their position in the eutrophic zone (Sukias *et al.*, 2003).

As with ammonia removal, figures 4.14 and 4.15 show that algal growth in the Errol lagoons is seasonal, with the highest during the summer period. This is to be expected based on the extended daylight hours and more sunshine during the summer months.

The presence of algae is known to increase the pH of the lagoons (Pearson *et al.* 1987). However, the presence of algae in the Errol lagoon did not prevent the final effluent pH from complying with the SEPA consents. With regard to the effect of algae on the final effluent suspended solids level which is mentioned by many writers; the accelerated growth of algae in the summer period increased the suspended solids concentration in the final effluent, but this was not so great as to stop final effluent complying with SEPA's consent for TSS.

5.5 Sludge Accumulation

Sludge depth measurement was carried out using the white towel test. According to Abis (2002) the test advantages are that it is cheap and reliable while its main disadvantage is its insensitivity to loose sludge.

Nelson *et al.* (2004) concluded that most sludge accumulates around the inlet in facultative ponds based on results of their study on sludge accumulation in four lagoons in central Mexico. Abis (2002) also found in her study of facultative lagoons in the UK that most sludge accumulated around the inlet, though some accumulation was detected around the outlet and at the sides. This is almost identical to the situation in the Errol ponds where sludge accumulated mostly around the inlet of the primary lagoon, with very little accumulation (2mm) in many parts of the primary lagoon and the secondary lagoon. Comparison between the theoretical total solids volume and the volume of sludge accumulated showed a solids removal of 99.9% inside the lagoons. The volume of sludge accumulated in one year is approximately 5.68m^3. This accumulation accounts for a per capita sludge accumulation of 0.0047 m^3/person/year, that is comparable to the estimate given by Nelson *et al.* (2004) of 0.004 m^3/person/year for the rate of sludge accumulation in facultative lagoons in the central region of Mexico (Nelson *et al.* 2004).

However, though sludge accumulation was found to be comparable to the estimate given for sludge accumulation in facultative lagoons, many people within Scottish Water were surprised with sludge depth measurement results as they expected sludge to be evenly distributed and with a larger accumulation. Thus, factors that could have affected sludge accumulation needed to be examined. These were presumed to be sludge accumulation in the pumping station, removal of sludge in the screen or the weak characteristics of the influent. There is no particular problem at the Errol pumping station because of sludge accumulation and in addition most screened materials were papers (i.e. toilet papers) not sludge. The third explanation is more logical as it was shown by influent wastewater analysis that the received sewage at Errol is weak. However, influent analysis also showed a possibility of a large non-biodegradable fraction and so even though the influent wastewater was weak, it has a large fraction that could eventually accumulate in the lagoons. Therefore, the sludge accumulation in Errol's lagoons is normal and not a result of the weak sewage or system failure. In the US, sludge depth is usually measured in similar systems after at least 10 years from the plant opening, thus another sludge measurement will be advisable after 10 years for more noticeable results and to verify the first measurements.

After 20 years of operation; the sludge depth around the inlet is expected to reach 261cm, after taking in to account annual population rise of 48 people. The sludge depth in other parts of the primary lagoon and the secondary lagoon is expected to reach a maximum of 2.1 cm. Desludging was estimated to be required after 15 years, but only around the inlet of the primary pond.

5.6 Retention Time

The retention time of the Errol Aero-Fac® facultative lagoons is not only governed by the daily flow volume but by also the hydraulic behaviour inside the lagoons. However, it is extremely difficult to measure all the factors that govern the hydraulic behaviour and thus most estimation of the Errol lagoons retention time, did not take in to consideration the effect of hydraulic behaviour. For example, the design retention time was estimated by LAS information package by assuming an average received flow of 864 m^3/day as (13,000m^3 (lagoon volume)) / (864(average daily flow)) = 15 days in each lagoon. Using the same method to estimate the current average retention time assuming an average received flow of 647.5 (m^3/day), the retention time in each lagoon equals 20.1 days.

The tracing study carried in Errol Aero-Fac lagoons showed the dispersion number was 0.071 in the primary lagoon and 0.058 in the secondary lagoon. These values are close to plug flow and therefore indicate only a small short degree circuiting with few dead spaces, which has a little effect on the retention time.

5.7 Treatment Cost

Estimation of the annual cost of operation showed that the Aero-Fac® system operation can cost annually £34,552 compared with the initial estimation of £8656 by Montgomery Watson (NoSWA, 2001). The difference is the result of the annual business rate charged by the local council for the land in which the Aero-Fac® system was constructed (£30,000). Comparing the Errol Aero-Fac system annual operation costs to the annual operation costs of other systems used by Scottish Water for similar populations to Errol includes £18281for St Fillans WWTW, £26819 for Pitlochry WWTW and £18186 for Comrie WWTW, shows that the operation cost of Errol Aero-Fac system is comparatively high. However, once the excessive business rate for the plant has been removed the system compares very favourably.

CHAPTER 6: CONCLUSIONS

From the results obtained in this study the following conclusions can be drawn:

1. The current sewage received at Errol wastewater treatment plant can be described as weak and not typical of the UK. It also contains a large non-biodegradable fraction.

2. The final effluent (unfiltered and hence filtered) BOD, TSS and pH all complied with 95% confidence to SEPA and the performance guarantee consents. The 95%ile unfiltered COD did not comply with the UWWT Directive consent for filtered COD; however the removal of COD was 71% and the average unfiltered COD complied with the Directive.

3. The primary pond effluent results show that the secondary lagoon is not required.

4. Sludge accumulated mainly in the primary lagoon around the inlet. The volume of sludge accumulated in 1 year is approximately 5.68m^3 corresponding to 0.005m3/person/year. Desludging would be required after 15 years of operation, but only around the inlet.

5. Ammonia removal is seasonal, with the highest removal during the summer period and surprisingly the lowest removal during the spring rather than the winter period.

6. The capital cost of the Aero-Fac® system is very high compared to other wastewater treatment systems for small populations. In addition, the recent business rate charged by the local council on the land in which the Errol wastewater treatment system was built increased the annual operational costs by £30,000.

7. Presence of algae in the lagoons is seasonal and is higher in the primary lagoon. However, this did not prevent the final effluent pH and suspended solids complying with SEPA consents.

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Appendix

Results Data

Performance Tests Results

The following is a summary of results of the performance tests that were carried by Scottish Water as part of their contract with the Aero-Fac system contractors MWH. (Please note that unfiltered BOD and COD are labelled BOD and COD, while filtered BOD and COD are labelled filtered in the results tables).

Performance test (1) results.

Raw wastewater samples results

DAY	DATE	BOD	Filtered BOD	COD	Filtered COD	TSS	pH(site reading)
1	14-15/7/01	111	51	265	143	105	7.74
2	15-16/7/01	115	61	264	143	105	7.76
3	16-17/7/01	121	88	309	211	121	8.50
4	17-18/7/01	83	32	230	110	81	7.85
5	18-19/7/01	147	91	386	214	115	7.88
6	19-20/7/01	185	97	456	223	153	7.85
7	20-21/7/01	184	81	393	203	134	6.23
8	21-22/7/01	208	51	411	168	188	7.64
9	22-23/7/01	217	101	554	238	240	7.67
10	23-24/7/01	78	51	171	90	76	7.20
11	24-25/7/01	187	118	337	224	95	6.53
12	25-26/7/01	143	44	320	107	101	7.73
13	26-27/7/01	158	71	202	84	144	7.74
14	27-28/7/01	119	32	242	102	99	7.57
15	28-29/7/01	153	74	399	227	133	7.18
16	29-30/7/01	125	80	352	224	81	7.32
17	30-31/7/01	113	42	608	105	102	7.81
18	31-1/8/01	125	152	343	184	108	8.00
20	2-3/8/01	146	116	294	300	75	7.49
21	3-4/8/01	165	175	381	192	70	7.19
22	4-5/8/01	143	78	396	142	119	7.37
23	5-6/8/01	147	59	366	134	107	7.67
24	6-7/8/01	199	55	425	188	110	7.66
25	7-8/801	113	46	321	154	111	7.73
26	8-9/8/01	166	58	413	171	127	7.43
27	9-10/8/01	144	72	356	195	88	7.81
28	10-11/8/01	188	57	439	177	115	7.42

29	11-13/8/01	158	159	396	189	94	7.45
30	13-14/8/01	85	49	298	166	65	7.43
31	14-15/8/01	176	58	411	191	96	7.90
32	15-16/8/01	97	45	238	132	59	7.90
33	16-17/8/01	99	56	268	151	108	7.60
34	17-18/8/01	148	78	377	177	76	7.20
3	18-19/8/01	127	50	308	148	91	7.20
36	19-20/8/01	35	8	91	35	12	7.21
37	20-21/8/01	100	61	257	142	52	7.83
38	21-22/8/01	60	37	178	94	32	7.57
39	22-23/8/01	175	123	362	186	81	7.50
40	23-24/8/01			356	168		
41	24-25/8/01	150	83	374	188	83	7.78
42	27-28/8/01	173	65	423	191	65	7.75
43	4-5/9/01	167	76	394	195	130	
44	12-13/9/01	153	60	373	187	113	7.23
45	13-14/9/01	179	77	415	218	103	7.67

Final effluent samples results

DAY	DATE	BOD	Filtered BOD	COD	Filtered COD	TSS	pH
1	16-17/9/01	6	6	58	49	9	
2	17-18/9/01	7	6	58	40	12	8.10
3	18-19/9/01	10	8	66	50	12	8.00
4	19-20/9/01	9	7	62	42	18	7.80
5	20-21/9/01	8	6	52	44	15	7.80
6	21-22/9/01	5	6	58	41	14	7.80
7	22-23/9/01	5	5	61	48	10	8.10
8	23-24/9/01	5	6	70	47	13	8.00
9	24-25/9/01	25	9	69	35	28	8.10
10	25-26/9/01	6	5	53	43	19	8.00
11	26-27/9/01	7	7	66	54	22	7.90
12	27-28/9/01	7	6	62	46	18	8.10
13	28-29/9/01	5	8	68	45	16	7.90
14	29-30/9/01	6	5	60	37	14	8.00
15	30/9 -1/1/0/01	8	5	61	37	17	8.00
16	1-2/10/01	16	10	72	37	25	8.00
17	2-3/10/01	8	6	68	48	23	7.90
18	3-4/10/01	9	6	64	38	54	7.90
19	4-5/10/01	9	6	66	43	24	7.9
20	5-6/1/0/01	8	6	61	41	18	7.90
21	6-7/10/01	8	4	47	35	18	7.90
22	7-8/10/01	7	5	65	35	19	7.50

Bacteriological results

Date	Crude			Final			Removal Rates		
	Coliforms	E.Coli	Faecal	Coliforms	E.coli	Faecal	Coliforms	E.coli	Faecal
17/10/01	20000000	16000000	750000	165000	6590	730	99.18	99.96	99.9
18/10/01	20000000	13000000	1600000	100000	3840	430	99.5	99.97	99.97
19/10/01	3800000	200000		83000	3400	7600	97.82	98.3	
22/10/01	3200000	740000	180000	83000	13000	2300	97.41	98.24	98.72

24/10/01	1900000	410000	34000	120000	5300	1500	93.68	98.71	95.59
Average	9780000	6070000	641000	110200	6426	2512	97.52	99.04	98.55

Performance test (2) results.

Raw wastewater samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NH3	NO2	SRP	NO3
07-08/01/02	119	279	92	7.2	0.2	26.66	0.02	5.8	0.2
08-09/01/02	61	304	126	7	0.2	24.88	0.02	5.4	0.2
09-10/01/02	95	298	128	7.4	0.2	22.25	0.02	4.9	0.2
10-11/01/02	86	315	143	7	0.3	21.76	0.06	4.7	0.2
12-13/01/02	134	316	115	7.3	0.1	23.26	0.02	6.2	0.1
13-14/01/02	163	385	165	7.3	0.1	23.04	0.02	5.8	0.1
14-15/01/02	137	170	277	7.2	0.2	20.07	0.02	4.5	0.1
15-16/01/02	135	268	106	7.4	0.1	23.03	0.02	5.6	0.1
16-17/01/02	128	326	124	6.9	0.5	24.41	0.16	4.8	0.4
17-18/01/02	116	284	151	7.2	0.2	20.86	0.02	4.2	0.2
18-19/01/02	53	2140	64	7.1	0.2	15.65	0.03	3.3	0.1
19-20/01/02	47	163	91	7.2	0.1	14.51	0.03	3.2	0.1
20-21/01/02	69	231	169	7.1	2.4	10.42	0.18	3	2.2
21-22/01/02	103	147	98	6.9	1.3	8.65	1.08	2.2	0.2
22-23/01/02	*	205	133	6.81	1.6	11.5	0.22	2.4	1.4
23-24/01/02	57	51	65	7	6.7	2.25	0.12	0.9	6.6
24-25/01/02	12	59	49	7.16	8.4	4.38	0.34	1.4	8.1
25-26/01/02	23	69	29	7.2	4.1	6.13	3.9	1.3	0.16
26-27/01/02	19	58	26	7.1	4.7	3.42	0.06	1.1	5.1
27-28/01/02	45	196	112	7.15	3	5.24	0.65	1.9	5.1
28-29/01/02	57	152	57	7.7	2.6	12.51	0.07	3.4	3.1
29-30/01/02	14	79	107	7	2.4	3.5	0.19	1.1	2.2
30-31/01/02	18	74	68	7.3	6.2	2.63	0.09	1	6.2
31/01-01/02/02	1	440	38	7.1	3.1	4.72	1.05	1.1	2.1
01-02/02/02	10	27	39	7	4.6	1.39	0.08	0.8	4.5
02-03/02/02	14	79	50	7.4	5.9	5.05	0.24	1.4	5.7
03-04/02/02	31	60	40	7.1	3.8	4.06	0.55	1.2	3.2
04-05/02/02	12	83	37	7.36	4	5.88	0.26	2	3.7
05-06/02/02	8	58	47	7.3	4.1	6.01	0.17	1.4	3.9
06-07/02/02	50	119	115	7.4	3.3	9.59	0.38	1.9	2.9
07-08/02/02	17	45	56	7.1	2.9	4.04	0.4	1.2	2.5
08-09/02/02	10	72	38	7.2	4	5.05	0.45	1.3	3.6
09-10/02/02	47	86	37	7.4	4.2	6.33	0.41	1.6	3.8
10-11/02/02	30	97	42	7.5	3.1	8.07	0.34	2.1	2.7
11-12/02/02	10	68	44	7.5	3.3	5	0.21	1.3	3
12-13/02/02	35	105	102	7.2	3	3.75	0.14	1.2	2.9
13-14/02/02	48	128	101	7.3	3	5.42	1.59	1.5	1.4
14-15/02/02	43	78	44	7.5	4.1	9.56	0.47	1.7	3.7
15-16/02/02	21	74	21	7.5	3.2	11.3	0.67	2.2	2.5
16-17/02/02	37	104	29	7.5	2.1	13.5	0.48	2.9	1.6
17/03/02	33	119	53	7.4	1.5	10.76	0.34	2.6	1.2

Final effluent samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NH3	NO2	SRP	NO3
08-09/01/02	5	51	10	7.3	2.4	16.86	0.02	4	2.3
09-10/01/02	16	57	19	7.8	2.4	16.59	0.02	4	2.4
10-11/01/02	10	60	13	7.5	2.4	17.06	0.02	4.1	2.4
12-13/01/02	4	49	15	7.6	2.4	14.47	0.02	4.2	2.4
13-14/01/02	5	51	32	7.5	2.5	14.49	0.05	4.2	2.5
14-15/01/02	6	48	23	7.8	2.5	17.19	0.02	4.2	2.4
15-16/01/02	8	50	21	7.7	2.4	17.38	0.02	4.3	2.4
24/01/02	29	62	19	6.9	2	16.36	0.06	4	2
25/01/02	10	48	17	7.4	2.1	16.09	0.07	4	2
28/01/02	6	47	14	7.54	2.5	13.28	0.07	3.6	2.9
29/01/02	7	52	9	8	3	5.71	0.07	1.5	3.4
30/01/02	7	48	114	7.6	2.6	13.6	0.04	3.3	2.6
31/01/02	4	38	14	7.7	2.6	12.89	0.02	3.1	2.6
31/01-01/02/02	7	42	4	7.5	2.8	12.64	0.12	3.1	2.7
01-02/02/02	10	42	14	7.6	3	11.74	0.11	2.9	2.9
02-03/02/02	10	43	12	7.4	3.2	10.65	0.08	2.7	3.1
03-04/02/02	10	36	4	7.7	3.2	9.85	0.14	2.5	3
14-15/02/02	3	45	14	7.3	3.5	6.22	0.14	1.6	3.4
15-16/02/02	4	29	12	7.6	4	6.67	0.11	1.8	3.9
16-17/02/02	4	106	9	7.7	3.3	6.53	0.12	1.8	3.1
17-18/02/02	3	33	10	8	3.8	6.12	0.15	1.5	3.6
18-19/02/02	3	30	7	7.8	1	5.68	0.29	1.5	0.7
19-20/02/02	4	36	7	7.8	3.6	6.12	0.12	1.3	3.5
20-21/02/02	3	32	10	7.7	3.7	6.24	0.13	1.9	3.6
21-22/02/02	10	35	16	7.7	3.6	6.22	0.13	1.8	3.5
22-23/02/02	4	31	26	7.9	3.3	6.27	0.12	1.9	3.2
23-24/02/02	3	37	7	7.9	3.6	6.39	0.12	1.9	3.4
24-25/02/02	5	39	9	7.9	3.4	6.44	0.12	1.9	3.3
25-26/02/02	3	36	16	7.8	4.1	6.22	0.12	2.7	4
26-27/02/02	4	35	10	7.7	3.5	6.44	0.12	1.9	3.4
27-28/02/02	8	38	11	7.7	3.5	6.45	0.11	1.9	3.4
28/02-01/03/02	6	37	12	7.8	3.4	6.63	0.12	1.9	3.3
01-02/03/02	6	43	19	7.7	3.4	6.71	0.02	1.9	3.3
02-03/03/2002	5	41	9	7.8	3.3	6.35	0.11	1.9	3.2
03-04/03/02	10	48	12	7.8	3.4	6.34	0.11	1.9	3.3
04-05/03/02	1	47	12	7.3	3.5	6.5	0.11	1.9	3.4
05-06/03/02	5	38	13	7.8	3.8	6.17	0.07	1.9	3.7
06-07/03/02	4	48	16	7.9	3.9	6.16	0.07	1.9	3.8
07-08/03/02	5	42	14	8	3.9	5.86	0.14	1.9	3.8
08-09/03/02	7	39	14	8	4.1	5.59	0.13	1.9	4
09-10/03/02	7	48	16	8.1	4.2	5.4	0.11	1.9	4.1
10-11/03/02	10	50	17	7.9	3.9	6.29	0.09	1.9	3.8
11-12/03/02	8	58	23	8.3	4.1	5.99	0.08	1.9	4
12-13/03/02	40	54	24	8.2	3.9	5.46	0.11	1.8	3.8
13-14/03/02	4	54	26	8.3	4.1	5.41	0.1	1.9	4
14-15/03/02	3	56	33	8.1	4.1	5.7	0.1	1.8	4
15-16/03/02	4	61	33	8.2	4.1	5.68	0.1	1.9	4
16-17/03/02	5	59	31	8.1	3.9	5.82	0.1	1.9	3.8

17-18/03/02	10	66	36	8.3	4.2	5.47	0.11	1.9	4.1
18-19/03/02	12	63	31	8.3	4.1	5.42	0.11	3.7	4
19-20/03/02	10	70	34	8.2					

Performance test (3) results

Raw wastewater samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NO2	NO3	NH3	SRP
06-07/05/02	158	550	241	7.4	0.1	0.02	0.1	26.5	6.7
07-08/05/02	56	162	95	7.4	0.1	0.02	0.1	23.32	5.1
08-09/05/02	94	277	103	7.5	0.1	0.02	0.1	24.42	4.8
09-10/05/02	149	348	119	7.2	0.1	0.03	0.1	27.65	5.7
10-11/05/02	121	228	52	7.2	0.1	0.02	0.1	24.86	5.7
11-12/05/02	230	394	109	7.1	0.1	0.06	0.1	26.41	8.4
12-13/05/02	101	280	69	7.3	0.1	0.03	0.1	28	7
13-14/05/02	41	133	52	7.2	0.1	0.02	0.1	13.89	2.6
14-15/05/02	118	342	177	7.4	0.1	0.02	0.1	23.39	5.1
15-16/05/02	88	230	74	7.5	0.1	0.02	0.1	26.5	5.2
16-17/05/02	262	436	170	7.3	0.1	0.02	0.1	24.24	5.4
17-18/05/02	114	408	182	7.4	0.2	0.02	0.1	25.42	5.4
18-19/05/02	114	466	231	7.3	0.2	0.11	0.1	22.31	4.4
19-20/05/02	99	367	180	7.3	0.2	0.04	0.1	22.56	5.1
20-21/05/02	84	162	74	7.4	0.4	0.3	0.1	13.88	2.7
21-22/05/02	114	239	111	7.7	0.1	0.02	0.1	15.35	3.7
22-23/05/02	*	194	211	7.3	1.6	0.39	1.2	7.98	1.9
23-24/05/02	129	316	229	7.3	0.1	0.02	0.1	15.44	3.1
24-25/05/02	56	169	77	7.2	0.1	0.02	0.1	14.37	2.8
25-26/05/02	157	340	106	6.7	0.2	0.02	0.1	12.47	4.1
26-27/05/02	149	472	276	7.3	0.1	0.02	0.1	17.83	4
27-28/05/02	85	284	346	7.1	0.1	0.03	0.1	6.79	2
28-29/05/02	75	252	218	7.2	0.2	0.02	0.2	11.92	2.4
29-30/05/02	79	298	127	7.3	0.1	0.02	0.1	14.84	2.9
30-31/05/02	87	298	108	7.1	0.1	0.02	0.1	17.88	3.9
31-01/06/02	81	278	100	7.3	0.7	0.03	0.7	21.59	5
01-02/06/02	102	339	156	7.4	0.2	0.02	0.2	23.79	5.5
02-03/06/02	185	510	263	7.3	0.2	0.02	0.2	20.39	3.9
03-04/06/02	83	239	150	7.3	1	0.26	0.8	10.22	2.2
04-05/06/02	97	361	170	7.5	0.1	0.02	0.1	19.97	4.3
05-06/06/02	113	350	169	7.4	0.2	0.02	0.2	18.13	3.6
06-07/06/02	112	301	127	7.2	0.2	0.02	0.2	21.45	5.1
07-08/06/02	57	306	150	7	0.2	0.02	0.2	15.89	3.4
08-09/06/02	129	280	113	7.3	0.2	0.02	0.2	19.92	4.1
09-10/06/02	98	343	223	7	0.9	0.12	0.7	7.19	1.6
10-11/06/02	10	86	30	7.8	1.4	0.29	1.1	13.77	3.9

Final effluent results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NO2	NO3	NH3	SRP
07/05/02	11				2.5	0.11	2.4	8.27	3.1
08/05/02	10	45	33	8	2.5	0.02	2.4	8.08	3
09/05/02	8				2.2	0.08	2.1	8.32	3.2
10/05/02	11				2.1	0.12	2	8.93	3.4
07/06/02	20	75	30	7.7	1.8	0.41	1.4	12.54	3.8
11-12/06/02	6	111	27	7.9	1.7	0.29	1.4	13.45	3.9
12-13/06/02	9	74	36	7.3	1.5	0.4	1.1	13.24	3.8
13-14/06/02	7	87	28	7.6	1.6	0.3	1.3	13.2	3.8
14-15/06/02	6	62	23	7.6	1.9	0.36	1.5	13.21	3.8
15-16/06/02	1	77	32	7.5	1.8	0.47	1.4	13.37	3.8
16-17/06/02	6	70	16	7.5	1.9	0.49	1.4	13.16	3.8
17-18/06/02	8	55	37	8	1.6	0.56	1	13.93	3.8
18-19/06/02	5	62	27	7.9	2.9	0.52	2.4	12.56	3.7
19-20/06/02	7	55	20	7.9	3.1	0.64	2.5	11.83	3.6
20-21/06/02	9	209	29	7.3	3.7	1.19	2.5	10.49	3.7
21-22/06/02	5	66	19	7.9	2.2	0.87	1.3	16.39	4.2
22-23/06/02	6	59	22	11.6	4.1	1	3.1	11.56	3.7
23-24/06/02	9	80	37	7.6	7.1	1.08	6	7.12	3.6
24-25/06/02	7	75	29	8	3.5	1.02	2.5	9.99	3.6
25-26/06/02	26	78	29	7.4	7.5	0.98	6.5	5.61	3.5
26-27/06/02	10	85	41	7.4	8.6	0.83	7.8	4.78	3.5
27-28/06/02	12	87	41	7.8	4.7	1.33	3.3	8.2	3.5
28-29/06/02	9	74	52	8.1	7	1.37	5.6	6.1	3.4
29-30/06/02	6	77	35	7.4	10	0.33	9.7	2.64	3.4
30-01/07/02	10	94	50	7.8	7.3	0.55	6.8	5.23	3.4
01-02/07/02	15	131	70	7.4	11.5	0.14	11.3	2.41	3.4
02-03/07/02	12	103	52	7.7	7.8	0.17	7.7	4.47	3.4
03-04/07/02	11	83	37	7.7	7.1	0.17	6.9	4.97	3.5
04-05/07/02	9	81	36	7.3	7.8	0.23	7.5	5.29	3.3
05-06/07/02	14	80	33	7.6	8.6	0.4	8.2	4.38	3.3
06-07/07/02	21	94	43	7.4	10.9	0.36	10.6	1.91	3.2
07-08/07/02	17	70	34	7.5	9.7	0.94	8.7	3.11	3.2
08-09/07/02	22	98	44	7.2	11.2	1.3	9.9	1.6	3.2
09-10/07/02	34	94	42	7.2	13.4	0.86	12.5	0.33	3.6
10-11/07/02	17	43	39	7.3	12	0.16	11.8	0.4	3.2
11-12/07/02	10	82	33	7.8	11.2	0.05	11.1	0.46	3
12-13/07/02	8	61	11	7.6	12.4	0.06	12.4	0.55	3.3
13-14/07/02	9	84	34	7.8	11.7	0.15	11.5	0.45	3.2
14-15/07/02	6	82	38	7.6	11.3	0.3	11	0.37	3.2
15-16/07/02	12	103	55	7.6	10.6	0.06	10.5	0.91	3.1
16-17/07/02	7	79	45	7.5	10.3	0.09	10.2	0.82	3.1
17-18/07/02	8	66	26	7.8	10.6	0.72	9.9	0.48	3.2

Performance test (4) results

Raw wastewater samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NO2	NO3	NH3
06/10/02	195	510	208	7.3	0.1	0.02	0.1	29.12
07/10/02	179	704	222	7.2	0.1	0.02	0.1	29.9
08/10/02	124	394	168	7.3	0.1	0.02	0.1	25.97
09/10/02	135	463	179	7.2	0.4	0.02	0.4	27.56
10/10/02	141	420	164	7.2	0.1	0.03	0.1	29.78
11/10/02	28	60	38	7.2	0.3	0.23	0.1	4.54
12/10/02	32	89	72	7.1	1.8	0.3	1.5	2.03
13/10/02	164	454	422	7.1	0.1	0.02	0.1	9.14
14/10/02	60	234	215	7.1	1.9	0.72	1.2	3.3
15/10/02	68	284	183	7.3	4.6	0.19	4.4	4.23
16/10/02	70	46	168	7.5	1	0.95	0.1	10.65
17/10/02	78	238	157	7.3	0.9	0.02	0.9	13.18
18/10/02	221	500	347	7.4	0.2	0.02	0.2	19.34
19/10/02	162	604	331	7.2	0.8	0.02	0.7	18.27
20/10/02	131	416	247	7.2	0.2	0.02	0.2	17.27
21/10/02	47	165	143	7	1.4	0.12	1.3	3.41
22/10/02	8	38	32	7.2	5.2	0.08	5.1	0.99
23/10/02	45	105	100	7.3	5.8	0.32	5.4	5.34
24/10/02	32	173	130	7.3	3	0.25	2.7	8.08
25/10/02	16	90	166	7.8	2.3	0.28	2.1	4.36
26/10/02	50	179	129	7.9	2.3	0.45	1.9	7.96
27/10/02	30	122	27	8.4	0.5	0.06	0.4	5.24
28/10/02	61	185	119	7.6	1.8	0.38	1.4	10.37
29/10/02	80	207	123	7.4	2	0.41	1.6	11.7
30/10/02	58	237	123	7.3	2.2	0.31	1.9	13.06
31/10/02	78	252	142	7.7	2.4	0.34	2	13.85
01/11/02	121	416	228	7.4	0.4	0.1	0.3	11.36
02/11/02	95	311	217	7.4	0.9	0.16	0.8	9.38
03/11/02	38	224	145	7.5	2.9	0.31	2.6	7.75
04/11/02	82	184	141	7.5	2.6	0.23	2.3	12.64
05/11/02	77	372	169	7.5	1.1	0.25	0.9	13.29
06/11/02	93	272	153	7.5	0.3	0.02	0.3	12.2
07/11/02	144	277	167	7.5	0.1	0.02	0.1	14.43
08/11/02	83	264	154	7.6	0.2	0.03	0.2	16.06
09/11/02	121	271	158	7.4	0.1	4.39	0.1	17.52
10/11/02	120	316	211	7.2	0.3	0.03	0.2	8.63
11/11/02	113	438	225	7.4	0.2	0.02	0.2	14.68
12/11/02	95	441	405	7.1	9	0.14	0.8	5.28
13/11/02	59	207	137	7.3	0.2	0.02	0.2	9.58
14/11/02	55	146	173	7.5	1.1	0.17	0.9	2.93
15/11/02	59	354	216	7.3	1.2	0.33	0.8	9.12
16/11/02	75	221	214	7.4	1.9	0.26	1.7	6.1

Mid-lagoon samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	NH3
27/10/02	17	93	64	8	5.5
28/10/02	17		26	7.5	2.7
30/10/02	11	57	27	7.4	2.93
31/10/02	28	75	24	7.5	3.39
01/11/02	11	87	23	7.5	3.78
02/11/02	14	171	30	7.6	3.92
03/11/02	22	29	35	7.6	*
04/11/02	9	55	30	7.6	4.27
05/11/02	10	57	30	7.6	4.62
06/11/02	8	63	23	7.4	4.65
07/11/02	31	59	24	7.5	3.61
08/11/02	11	52	20	7.7	*
09/11/02	7	87	25	7.3	5.34
10/11/02	8	58	20	7.4	6.05
11/11/02	6	63	24	7.6	6.68
12/11/02	14	80	37	7.3	6.95
13/11/02	8		25	7.4	5.83
14/11/02	6		18	8.1	5.83
15/11/02	7	58	36	7.4	5.51
16/11/02	6	51	25	7.6	5.58
17/11/02	20		25	7.5	6.2
18/11/02	9		21	7.6	5.2
19/11/02	7	56	27	7.06	4.9
20/11/02	31	31	22	7.5	6.35
21/11/02	32		13	7.6	8
22/11/02	29	60	17	7.6	6.69
23/11/02	6	45	14	7.5	6.51
24/11/02	6	50	21	7.5	6.4
25/11/02	9		26	7.6	7.09

Final effluent samples results (BOD and COD: unfiltered data)

Date	BOD	COD	TSS	pH	TON	NO2	NO3	NH3	SRP
17/11/02	20	47	15	7.7	2.5	0.24	2.2	0.24	1.9
18/11/02	6	48	17	7.7	2.3	0.17	2.1	3.74	1.9
19/11/02	6	68	17	7.7	2.3	0.13	2.2	2.86	1.9
20/11/02	6	45	21	7.6	2.4	0.2	2.2	3.93	1.8
21/11/02	6	46	23	7.5	2.6	0.3	2.3	4.15	1.9
22/11/02	7	48	14	7.6	2.3	0.27	2.1	4.16	1.8
23/11/02	6	42	14	7.7	2.2	0.08	2.1	4.41	1.8
24/11/02	6	44	15	7.7	2.1	0.09	2	4.58	1.9
25/11/02	3	35	15	7.7	2.2	0.08	2.1	4.66	1.8
26/11/02	10	51	18	7.7	2	0.05	2	4.78	1.8
27/11/02	19	45	17	7.6	2.4	0.19	2.3	4.87	1.5
28/11/02	4	41	13	7.7	2.1	0.12	2	5.28	1.5
29/11/02	20	45	15	7.8	2	0.05	1.9	5.04	1.8
30/11/02	6	41	12	7.6	2	0.11	1.9	5.09	1.9
01/12/02	6	43	12	7.7	1.9	0.1	1.8	4.99	1.9

02/12/02	4	37	11	7.7	1.9	0.09	1.9	5.29	1.8
03/12/02	6	40	15	7.6	1.9	0.06	1.8	5.7	1.9
04/12/02	8	42	11	7.9	1.7	0.09	1.6	6.02	1.9
05/12/02	3	42	10	7.7	1.8	0.09	1.7	5.85	1.9
06/12/02	4	43	9	7.8	1.9	0.04	1.8	5.86	1.9
07/12/02	14	138	11	7.7	1.7	0.06	1.6	6.26	1.9
08/12/02	6	57	10	7.8	1.7	0.05	1.6	6.23	1.9
09/12/02	4	126	13	7.7	1.8	0.08	1.7	6.16	1.9
10/12/02	10	60	31	7.7	2.1	0.07	2.1	6.05	1.9
11/12/02	6	38	16	7.9	2	0.09	1.9	6.09	2
12/12/02	6	36	12	7.7	1.7	0.02	1.7	6.64	2
13/12/02	7	37	10	8.1	1.5	0.02	1.5	6.77	2
14/12/02	6	44	8	7.8	1.7	0.02	1.7	6.79	2
15/12/02	6	46	11	7.8	1.7	0.02	1.6	6.74	2
16/12/02	5	40	11	7.7					2
17/12/02		38	13	7.7					2.1

Errol R&D: BOD, COD and SS Analysis Results

First, it was agreed that samples taken for BOD, COD and suspended solids analysis are to be taken according to the assumed retention. This was possible for most samples but due to the transportation difficulty, tight budget and frequent samplers failures there were many cases where it was not possible to sample according to the retention time. Please note that samples from 14/04/2004 were not taken according to the retention time.

Influent BOD, COD and Suspended solids (BOD and COD: unfiltered data)

Date	BOD mgO2/l	TCODmgO2/l	SS mg/l
02/07/03	67	286	172
09/07/03	224.99	323	185
16/07/03	183	619	192
06/08/03	27	297	217
14/08/03	170	503	239
20/08/03	110	615	328
03/09/03	189	544	258
05/09/03	206	660	341
26/09/03	91	297	74
01/10/03	292	1002	432
07/10/03	123	342	143
10/11/03	204	603	
05/12/03	126	864	695

09/01/04	179	1200	788
16/01/04	163	765	1044
21/01/04	77	326	165
14/04/04	89	573	367
16/04/04	140	455	189
18/04/04	88	295	128
20/04/04	145	409	165

Mid-lagoon BOD, COD and Suspended solids (BOD and COD: unfiltered data)

Date	BOD mgO2/l	TCODmgO2/l	SS mg/l
24/07/03	9.1	97	30
29/07/03	8.7	169	83
01/08/03	15	203	99
06/08/03	98	317	105
14/08/03	33	289	151
20/08/03	30	299	141
27/08/03		144	58
03/09/03	33	158	73
08/09/03	9.6	132	129
26/09/03	17	225	94
01/10/03	27	156	91
07/10/03	25	204	141
15/10/03	14	179	89
10/11/03	27	249	174
18/11/03	22	213	92
05/12/03	9.1	101	65
09/01/04	7.1	104	44
16/01/04	8.1	181	158
21/01/04	7.1	73	19
14/04/04	34	211	127
16/04/04	44	210	165
18/04/04	35	199	109
20/04/04	33	160	101

Final Effluent BOD, COD and Suspended solids (BOD and COD: unfiltered data)

Date	BOD mgO2/l	TCODmgO2/l	SS mg/l
14/08/03	11	144	98
20/08/03	11	145	97
27/08/03	12	169	102
03/09/03	19	240	189
08/09/03	17.7	263	221

26/09/03	3.7	88	34
01/10/03	8.3	94	38
07/10/03	7.2	90	31
15/10/03	4.4	96	29
10/11/03	33	640	236
17/11/03	6.9	212	89
05/12/03	5	107	47
09/01/04	5.4	83	28
16/01/04	4	64	21
21/01/04	4.6	59	9
14/04/04	9.33	93	56
16/04/04	6	70	23
18/04/04	8	68	18
20/04/04	14	71	28

Bacteriological results

Date	Crude		Final		Removal rates	
	Coliforms	E.*Coli*	Coliforms	E.*coli*	Coliforms	E.*coli*
14/04/04	141400000	64900000	10000	10000	99.99	99.99
16/04/04	19870000	10470000	44000	22000	99.78	97.89
18/04/04	160000	70000	1800	600	98.875	99.14
20/04/04	9500000	7400000	13000	7000	99.86	99.91
Average	42732500	20710000	17200	9900	99.62625	99.2325