THE ATTENUATION CAPACITY OF DIFFERENT SUBSOILS RECEIVING DOMESTIC WASTEWATER EFFLUENT

by

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It is the mark of an instructed mind to rest satisfied with the degree of precision which the nature of the subject permits, and not to seek an exactness where only an approximation of the truth is possible.

- *Aristotle*

DECLARATION

I hereby declare that this thesis is entirely my own work unless otherwise stated and that it has not been submitted for a degree at any other university. I agree to permit the library to lend or copy this thesis upon request.

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ABSTRACT

With the domestic wastewater effluent of approximately 400,000 dwellings in Ireland being discharged to groundwater annually and about 25% of all water supplies in the country provided by groundwater a risk of contamination of groundwater resources by domestic wastewater effluent exists. *Treatment Systems for Single Houses*, a wastewater treatment manual produced by the EPA, is aimed at reducing that risk. It recommends a rigorous site suitability assessment procedure, comprising a desk study and on-site assessment, to determine the potential of the subsoil for effluent assimilation and attenuation. It is currently considered that a site with 1.2m of unsaturated subsoil below the invert of the proposed percolation trenches with a T-value falling in the range 1min/25mm and 50min/25m is suitable for the installation of a septic tank treatment system. If less than 1.2m, but greater than 0.6m, of unsaturated subsoil is present, some form of secondary treatment is also required prior to discharge.

A three year project, funded by the EPA, was undertaken to examine the effectiveness of these guidelines. Four test sites with different subsoil characteristics and T-values falling within specified ranges were identified on which domestic wastewater treatment systems were installed. Two of these sites had septic tank treatment systems installed while two had secondary treatment peat filter systems installed downstream of the septic tank. All percolation trenches were constructed to EPA specifications. To assess the attenuation capacity of the subsoil on each site nine lysimeters were installed on each 20m percolation trench at varying depths. The domestic wastewater effluent and lysimeter samples were analysed for COD, ammonia, nitrite, nitrate, orthophosphate and chloride concentrations. Samples were also analysed for the microbiological indicator organisms total coliforms, *E. coli*, faecal coliforms, enterococci and faecal streptococci.

It was found that the installation of the peat filter system downstream of the septic tank greatly reduced the organic and biological load on the percolation areas. However, this reduction in organic load resulted in a hydraulic load greater than the

design hydraulic load being exerted on the percolation trenches, by inhibiting biomat formation along the trench base and side-walls. It also resulted in a greater nutrient load in the percolating effluent at the minimum recommended depth of unsaturated subsoil below the percolation area, i.e. the point of discharge to groundwater, than that recorded on the sites on which septic tank treatment systems were installed. The majority of the attenuation of the organic fraction of the septic tank effluent appeared to occur within the distribution gravel of the percolation trenches while, for all sites, orthophosphate fixation was dependent on subsoil characteristics. It appears from this research, therefore, that the two sites on which septic tank treatment systems were installed out performed the sites on which secondary treatment systems were installed and discharged a better quality effluent at the minimum recommended depth of unsaturated subsoil below the percolation area.

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LIST OF ABBREVIATIONS

AET	Actual evapotranspitation
Al/Al^{3+}	Aluminium
BAF	Biological aerated filtration
BOD	Biochemical Oxygen Demand
BOD_5	Five-day biochemical oxygen demand
BS5930	British Standards Institution Code of Practice for Site Investigations (1999)
C	Carbon
Ca/Ca^{2+}	Calcium
$CaCO_3$	Calcium carbonate
CEC `	Cation exchange capacity
CH_4	Methane
CHONS	Carbonaceous organic matter
Cl	Chloride
COD	Chemical Oxygen Demand
EPA	Environmental Protection Agency
Fe	Iron
GSI	Geological Survey of Ireland
GWPS	Groundwater Protection Scheme
H^+	Hydrogen
K^+	Potassium
Li^+	Lithium
Mg^{2+}	Magnesium
N	Nitrogen
N_2	Nitrogen gas
Na^+	Sodium
NERCC	National Regional Correction Centre
NH_3	Ammonia
NH_4^+	Ammonium
NH_4-N	Ammonium measured as nitrogen

NO	Nitric oxide gas
N_2O	Nitrous oxide gas
NO_2	Nitrite
NO_2-N	Nitrite measured as nitrogen
NO_3	Nitrate
NO_3-N	Nitrate measured as nitrogen
OD	Outside diameter
org-C	Organic carbon
org-N	Organic nitrogen
ortho-P	Orthophosphate
P	Phosphorus
PET	Potential evapotranspiration
PO_4	Phosphate
PO_4-P	Phosphate measured as phosphorus
RBC	Rotation biological contactor
RF	Rainfall
SAF	Submerged aerated filter
SBR	Sequencing batch reactor
SE	Secondary effluent
SMD	Soil moisture deficit
SS	Suspended solids
STE	Septic tank effluent
TKN	Total Kjeldahl nitrogen
Total-N	Total nitrogen
TSS	Total suspended solids

1. INTRODUCTION

1.1 Background

Water is a resource which is under increasing risk from human activities with contamination arising from both 'diffuse' (generally agricultural) and 'point sources', the latter exemplified by farmyards (manure and silage storage) and septic tank systems (Daly, 1993 and Robins and Misstear, 2000). In areas where the subsoil permeability is too low to allow sufficient soakage of the effluent there is a risk to surface watercourses from effluent ponding. Groundwater, on the other hand, is especially at risk in areas where bedrock is close to the surface, where subsoils of high permeability underlie the site and where the water table is close to the surface. Prevention of groundwater contamination is of critical importance as, once contaminated, the consequences are usually longer lasting than for surface water owing to longer residence times; moreover groundwater remediation is often expensive, if not impractical.

In Ireland, the domestic wastewater from over one third of the population, or approximately 400,000 dwellings, is treated by on-site systems (Department of the Environment and Local Government (DoELG) *et al.*, 2000). Both on-site systems and wells are necessary in the absence of public sewerage and water supply systems, yet on-site systems may pose problems on a small site because of the risk of contamination to the water source.

On-site systems can be divided into two broad categories: septic tank systems, which are in the majority, and secondary treatment systems. A conventional septic tank system comprises a septic tank followed by a soil percolation area. As alternatives to a conventional percolation area the effluent from a septic tank can also be treated by a variety of filter systems or package wastewater treatment systems. Removal of most of the suspended solids from the wastewater is achieved in the septic tank and this is accompanied by a limited amount of anaerobic digestion of settled solids in the base of the tank. The effluent then undergoes further physical, chemical and biological treatment in the percolation area before disposal.

In cases where the subsoils render a site unsuitable for conventional septic tank systems, modifications to the treatment system are available. A filter, such as an intermittent sand filter, peat filter or constructed wetland, can be constructed downstream of the septic tank. The effluent would pass through one of these processes and on to a polishing filter prior to disposal. Another alternative is the installation of a package wastewater treatment system. These systems, which produce a good quality of effluent for disposal, generally consist of a primary settlement tank and an aeration tank followed by a secondary clarifier. The aeration tank contains the micro-organisms, either in suspension or attached to an inert media, which are responsible for degradation of organic matter.

In many cases a lack of understanding of the treatment and disposal processes involved in domestic wastewater treatment has led to poor design, siting and installation of on-site treatment systems, resulting in contamination of groundwater and watercourses. These problems result mainly from unsuitable natural conditions being encountered at the site i.e. unsuitable soil and subsoil properties for the treatment and disposal of effluent. Domestic effluents contain many substances that are undesirable and potentially harmful to human health and the environment. Pathogenic bacteria, infectious viruses, protozoa, organic matter, ammoniacal compounds and a variety of toxic chemicals are all found in significant amounts in wastewater (Metcalf and Eddy, 2003).

1.2 Previous Studies

In recent years two major reports have been published by government agencies in relation to on-site treatment systems and groundwater protection. The Environmental Protection Agency (EPA) guidance document *Wastewater Treatment Manual: Treatment Systems for Single Houses* (EPA, 2000) aims to provide guidelines for the selection, design, operation and maintenance of these systems to enable continued sustainable development to take place in Ireland while *Groundwater Protection Schemes (*DoELG *et al, 1999)* aims to maintain the quantity and quality of groundwater, and in some cases improve it, by applying a risk assessment-based approach to groundwater protection and sustainable development. If these two complementary approaches are applied to the conventional *source-pathway-receptor* model for environmental management, Figure 1.1, the receptor being an aquifer, well or spring, the former approach would be seen as contamination prevention by minimising the contaminant source through adequate design of primary treatment and distribution systems while the latter specifically assesses the ability of the subsoil, or pathway, to protect the groundwater.

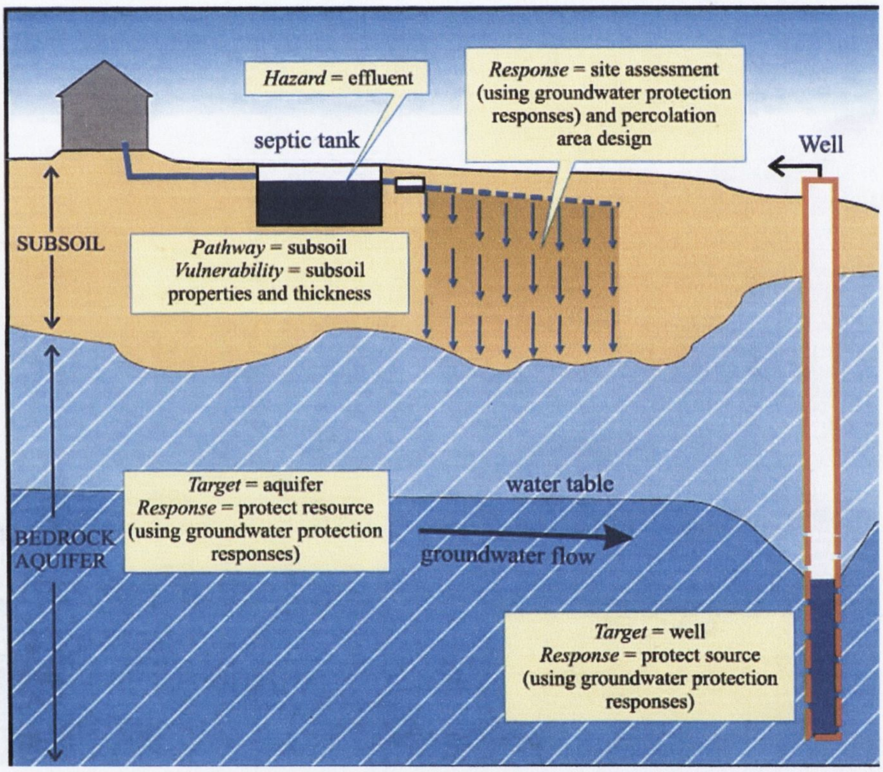


Figure 1.1 Schematic diagram showing how the elements of risk are applied to groundwater protection (DoELG *et al.*, 1999).

1.2.1 Wastewater Treatment Manuals: Treatment Systems for Single Houses

A research study entitled Small Scale Wastewater Treatment Systems, co-ordinated by the Department of Civil Engineering, The National University of Ireland, Galway, was carried out between 1995 and 1997 under the direction of the EPA (EPA, 1998).

The main objectives of this project were to assess small scale treatment systems and to establish guidelines for their future use, so as to ensure sustainable development. Following on from this study, and with regard to S.R.6: 1991 (*Septic Tank Systems, Recommendations for Domestic Effluent Treatment and Disposal from a Single Dwelling House*, which is published by the National Standards Authority of Ireland) the EPA published a guidance manual *Treatment Systems for Single Houses,* (EPA 2000). The manual is intended to provide guidance to planning authorities, developers, system manufacturers, designers, installers and operators on the design, operation and maintenance of on-site wastewater treatment systems for a single house. A single house system refers to a dwelling house of up to 10 people with toilet, living, sleeping, bathing, cooking and eating facilities.

The manual outlines the steps to be taken to assess the suitability of a site for a treatment system and a methodology for choosing the type of system and the optimum discharge route for the effluent. This is followed by information on the design, construction and maintenance of a septic tank, soil percolation area, intermittent filters, constructed wetlands and polishing filters. It also outlines the operation and maintenance, advantages and disadvantages of mechanical aeration systems.

1.2.2 Groundwater Protection Schemes

The Groundwater Protection Scheme (GWPS) framework is based on the concept of 'risk assessment' and 'risk management' (Misstear *et al*., 1998; DoELG *et al*., 1999). According to GWPS '*The risk of contamination of groundwater depends on three elements:*

- *the hazard afforded by a potentially polluting activity;*
- *the vulnerability of groundwater to contamination;*
- *the potential consequences of a contamination event.*

...........The hazard depends on the potential contaminant loading. The natural vulnerability of the groundwater dictates the likelihood of contamination if a contamination event occurs. The consequences to the targets depend on the value of the groundwater, which is normally indicated by the aquifer category (regionally

important, locally important or poor) and the proximity to an important groundwater abstraction source (a public supply well, for instance)' (DoELG *et al., 1999)*

The vulnerability of groundwater depends on (i) the time of travel of the effluent to the target, (ii) the relative quantity of contaminants that reach the groundwater and (iii) the attenuation capacity of the medium through which the effluent travels. Based on this geological and hydrogeological assessment of groundwater vulnerability the Geological Survey of Ireland (GSI) rated vulnerability categories as outlined in Table 1.1. They also defined source protection areas, which are protection areas around major wells and springs, and resource protection areas, based on a classification of Irish aquifers into regionally important, locally important and poor.

The risk management element comprises a series of responses to potentially polluting activities. DoELG *et al.* (2000) also published a paper to be used in conjunction with both *Groundwater Protection Schemes* and *Wastewater Treatment Manuals: Treatment Systems for Single Houses* which is entitled *Groundwater Protection Responses for On-Site Wastewater Systems for Single Houses*. It indicates groundwater protection zones suitable for on-site wastewater treatment systems, and for each of those zones, recommends acceptable treatment systems, conditions and/or investigations depending on groundwater vulnerability, the value of the groundwater resource and the contaminant loading.

Vulnerability Rating	**Hydrogeological Conditions**				
	Subsoil Permeability and Thickness			**Unsaturated Zone**	**Karst Features**
	High permeability	Moderate permeability	Low permeability	(Sand/gravel aquifers only)	(< 30 m radius)
Extreme (E)	0 – 3.0 m	0 – 3.0 m	0 – 3.0 m	0 – 3.0 m	-
High (H)	> 3.0 m	3.0 – 10.0 m	3.0 – 5.0 m	> 3.0 m	N/A/
Moderate (M)	N/A/	> 10.0 m	5.0 – 10.0 m	N/A/	N/A/
Low (L)	N/A/	N/A/	> 10.0 m	N/A/	N/A/

Notes: N/A = not applicable.

Release point of contaminants is assumed to be 1-2 m below ground surface

Table 1.1 Groundwater vulnerability categories (adapted from DoELG *et al.*, 1999)

1.3 Objectives and Scope of Study

This thesis forms part of a larger EPA sponsored study which examines *The Hydraulic Performance and Efficiencies of Different Subsoils and the Effectiveness of Stratified Sand Filters Receiving Domestic Wastewater Effluent.* It is a continuation of previous studies on domestic wastewater treatment systems. While the recent EPA studies focused mainly on construction of treatment systems this project aimed to enhance the understanding of the processes involved and performance of four different subsoils receiving domestic wastewater effluent from septic tanks and secondary treatment systems.

The study consisted of two sets of trials, one of twelve months duration and the other of eight months duration, designed to assess the following parameters:

a) the hydraulic and wastewater treatment performance of two subsoils of known permeability receiving septic tank effluent;
b) the hydraulic and wastewater treatment performance of two subsoils of known permeability receiving secondary treated effluent.

The trials of 12 month duration were carried out at two sites, one with a T-value of between 5 and 25 min/25mm receiving septic tank effluent and the other with a T-value between 25 and 50 min/25mm receiving secondary treatment effluent. For the trials of eight months duration the performance of a subsoil, of T-value between 25 and 50, receiving septic tank effluent and a subsoil, with a T-value between 50 and 60, receiving secondary effluent, was assessed.

Delays experienced at the start of the project due to site restrictions resulting from the risk of foot and mouth outbreak in 2001, allied with the difficulty experienced in identifying suitable research sites, resulted in the second set of field trials being reduced to an eight month study from the desired duration of 12 months.

1.4 Thesis Outline

Chapters 2 to 4 contain a literature review. Chapter 2 describes the design, operation and efficiency of conventional septic tank systems. In Chapter 3 some of the treatment alternative systems are examined including attached growth systems, suspended growth systems and hybrid systems. In Chapter 4 the common contaminants found in septic tank effluent are summarised and the processes by which the subsoil attenuates these contaminants are outlined.

Chapters 5, 6 and 7 describe the site selection, construction and instrumentation processes, respectively. Chapter 7 also deals with the sampling and analysis methodology. Chapters 8 and 9 present the results of the first set of trials while Chapters 10 and 11 present the results of the second set of trials. Finally, in Chapter 12 the results from the four research sites are compared and conclusions drawn. Chapter 12 also includes observations relating to the site assessment and construction processes and recommendations for future studies.

2. CONVENTIONAL SEPTIC TANK TREATMENT SYSTEMS

2.1 Introduction

Wastewater from individual dwellings in unsewered areas, generally rural, is managed by on-site treatment and disposal systems. Although a variety of on-site systems are available, the most common system consists of a septic tank and a subsurface soil disposal field or percolation area (Figure 2.1). A septic tank is a buried watertight container that serves as a combined settling and skimming tank and as an unheated-unmixed anaerobic digester for domestic wastewater effluent (Metcalf and Eddy, 1991). It provides for separation of sludges and floatable materials from the wastewater and an anaerobic environment for decomposition of both retained and non-settleable materials within the scum layer. The septic tank effluent (STE), being highly polluting (containing faecal bacteria and high levels of nitrogen, phosphorus, organic matter and other constituents (Daly *et al.*, 1993)), requires further treatment prior to discharge and is thus directed to a percolation area. In properly designed and constructed percolation areas advanced treatment is achieved for many wastewater constituents of concern through removal (e.g. filtration of suspended solids or sorption of phosphorus), transformation (e.g. nitrification of ammonium or biodegradation of organic matter) and destruction processes (e.g. die-off of bacteria or inactivation of viruses) (Siegrist *et al.*, 2000). Since conventional disposal fields are not suitable for all ground conditions, many alternative systems have been developed (EPA, 2000). The most suitable of these include mechanical aeration systems and sand intermittent filter systems.

Sufficient information must be obtained to determine whether a site is suitable for a conventional septic tank system i.e. septic tank and percolation area, or whether an alternative treatment system is required. The EPA recommends that this site characterisation consists of the following stages:

- a desk study, which collects any hydrological and hydrogeological information that may be available on maps etc. about the site and surrounding water resources (both surface water and groundwater);

- a visual assessment of the site, which defines the site in relation to surface features;
- a trial hole to evaluate the soil structure, depth to rock and water table;
- percolation tests.

(EPA, 2000)

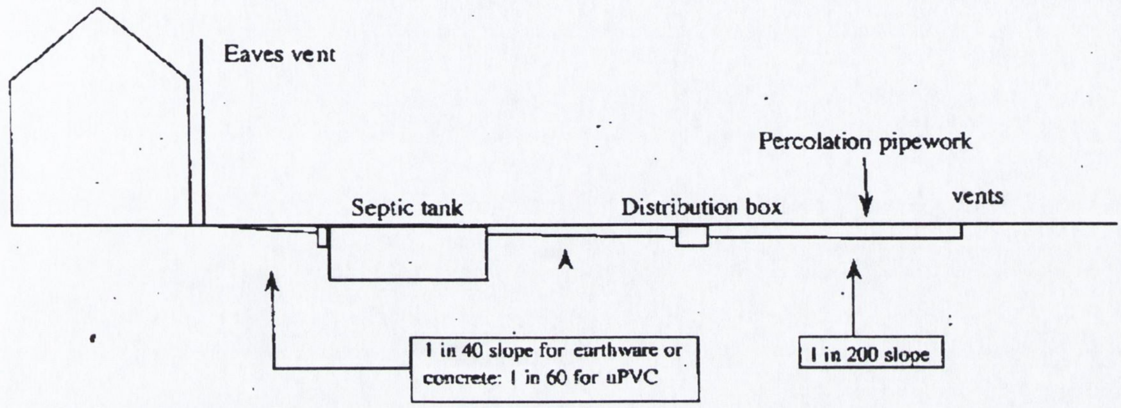


Figure 2.1 A conventional septic tank system (EPA, 2000).

2.2 Septic Tanks

2.2.1 Background

The first reported use of the household septic tank was in France in 1860 when John Louis Mouras and Abbe Moigno discovered that a 'box' placed between a house and its cesspool trapped excrement, reduced the amount of solids and produced a clarified liquid that more quickly entered the soil (Payne and Butler, 1995). It made its first appearance in the United States in 1883 when Philbrick introduced a two-chamber tank with an automatic siphon for intermittent effluent disposal to the residents of Boston. Septic tanks were introduced to England by Cameron in 1895 and the type in use today "are of a form that would be instantly recognisable by those early sanitary engineers" (Payne and Butler, 1995). Modifications to septic tank designs have been incorporated to improve the STE quality, particularly with respect to reducing the suspended solids concentration and to a lesser extent the Biochemical Oxygen Demand (BOD), thereby preventing accelerated clogging of soil adsorption systems.

2.2.2 Design of Septic Tanks

Whatever the mode of construction, in-situ or pre-fabricated, or shape, spherical, cylindrical or cuboid, septic tanks must be designed to:

- withstand corrosion;
- carry safely all lateral and vertical soil pressures;
- accommodate water pressure from inside and outside the tank without leakage occurring;
- remove almost all settleable solids in the effluent wastewater;
- prevent discharge of sludge or scum in the effluent and
- allow for the escape of accumulated gases.

It is important that septic tanks are correctly sized, based on the wastewater to be handled. Tanks that are properly sized and constructed provide highly effective treatment, capable of yielding effluent that is relatively free of fats, oils greases, solids and other constituents that can affect further treatment downstream (Bounds, 1997). Septic tanks are normally sized based on the design flow with 1/3 of the tank volume designed to provide a 24 hour hydraulic detention time while the other 2/3 is set aside for scum and sludge accumulation. This effectively yields a total tank volume equal to 3 times the daily flow volume (Baumann *et al.*, 1978 (cited in Siegrist *et al.*, 2000) and USEPA, 1980). A factor of safety should be provided to allow for variations in wastewater loading and future changes in the character of the waste (Canter and Knox, 1985). Oversized tanks will not be cost-effective while undersized ones will produce poor quality effluent. The septic tank must be of sufficient volume to provide a hydraulic retention time in excess of 24 hours (EPA, 2000) at maximum sludge depth and scum accumulation to facilitate BOD and solids removal from the wastewater. Thus, an important factor in the design of a septic tank volume is rate of sludge accumulation, which is dependent on the number of occupants of the dwelling. It is important, therefore, when designing a septic tank to always consider the potential number of occupants rather than present occupation levels. BS6297 (1983) and EPA (2000) recommend that the tank capacity be calculated using the following equation:

$$C = 180.P + 2000$$

where

C = the capacity of the tank (litres)

P = the design population

This assumes that the tank is desludged at least once in every 12-month period. In Ireland a minimum capacity of 2720 litres should be provided, which effectively equates to a design for a minimum population of 4 people (EPA, 2000). The installation of kitchen grinders increases the sludge load of the wastewater and so, in such situations, the septic tank capacity should be increased by 70 litres for each additional person (EPA, 2000). Table 2.1 outlines typical capacities and dimensions for rectangular tanks:

No. of Persons	Capacity (Litres) C = 180 . P + 2000	Dimensions (m)			
		Length		width	depth
		a*	d*	b*	c*
3	2720	2.2	1.0	1.0	1.2
4	2720	2.2	1.0	1.0	1.2
5	2900	2.4	1.0	1.0	1.2
6	3080	2.5	1.0	1.0	1.2

* refer to Figure 2.2

Table 2.1 Typical capacities of rectangular septic tanks (EPA, 2000)

Septic tanks may be constructed *in situ* of concrete or may be prefabricated from steel, reinforced concrete, glass fibre reinforced concrete or plastic. When steel is used as the construction material it must be treated with bitumen or other corrosion resistant substances. Despite a corrosion resistant coating, tanks still have the tendency to deteriorate at the liquid level (Canter and Knox, 1985). Research by the USEPA (1980) indicates that steel tanks have a short operational life, less than 10 years, due to corrosion. Enhanced quality control is associated with prefabricated tanks since post-construction testing can be carried out on the tanks in the factory. Regardless of the method of construction of the tank, it is essential that all joints are sealed properly to ensure water tightness.

Multi-chamber tanks are superior to single chamber tanks of same size producing effluent with up to 50% less suspended solids and BOD (Laak, 1980). However Laak also found that multi-chamber tanks without inlet and outlet baffles were 10-20% less efficient than single chamber tanks which were baffled. Baffles should be provided on the inlet and outlet of a septic tank to yield quiescent conditions within the tank and limit the disruption and re-entrainment of sludge and scum in the wastewater passing through the tank, thereby minimising suspended solids concentrations in the effluent. It should be noted, however, that a study by the Public health Service in the US cited in Bounds (1997) stated that there was no conclusive evidence to suggest that there was any significant difference in the operation of the one and two compartment tanks. The EPA recommends that septic tanks should comprise two interconnected chambers to limit the discharge of solids in the effluent from the septic tank (Figure 2.2). The benefit of a two-chamber tank appears to depend more on the design of the tank than on the use of two chambers (Metcalf and Eddy, 1991). The benefits of two chamber over single chamber tanks are due largely to hydraulic isolation, and to the reduction or elimination of inter-chamber mixing (Canter and Knox, 1985). Mixing can occur by two means, water oscillation and true turbulence. Oscillatory mixing can be minimised by making chambers unequal in size, reducing the flow-through area and using an L-piece to connect chambers. In the larger first chamber, where most of the sludge accumulates, influent wastewater causes some mixing of sludge and scum with the liquid. More quiescent conditions, which are better for settling of low-density solids, exist in the second chamber as it generally receives the wastewater at a lower velocity. These conditions, along with proper tank maintenance, lead to an effluent low in suspended solids entering the percolation area.

It is argued (Metcalf and Eddy, 1991) that a more effective way to eliminate the discharge of untreated solids involves the use of an effluent filter vault in conjunction with a single chamber tank (Figure 2.3). The vault contains a screen with a large surface area through which the effluent flows before disposal to the percolation area. Clogging of the screen is not generally a problem due to its large surface area but in the event of its occurrence, it is possible to remove and clean, or replace, the screen.

An advantage of the effluent filter vault is that it can be installed in both existing and new septic tanks to limit the discharge of gross untreated solids.

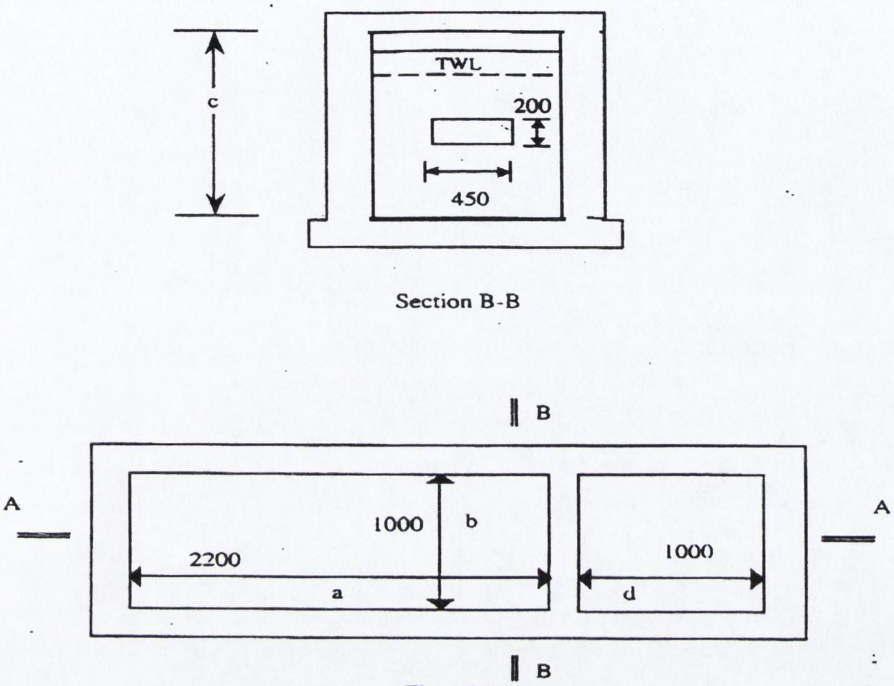


Figure 2.2 Diagrammatic layout of a septic tank (EPA 2000) (refer to Table 2.1)

Although the septic tanks discussed thus far are cuboid in shape, Canter and Knox (1985) reported that "current practices favour rectangular tanks although studies have shown little difference in performance between rectangular and cylindrical designs when sludge storage capacities were similar". Laak (1980) reported on a series of tests using a 27 hour detention time that found that the "outside shape of the tank, circular, cylindrical or rectangular did not appear to have had any significant effect" on the effluent quality.

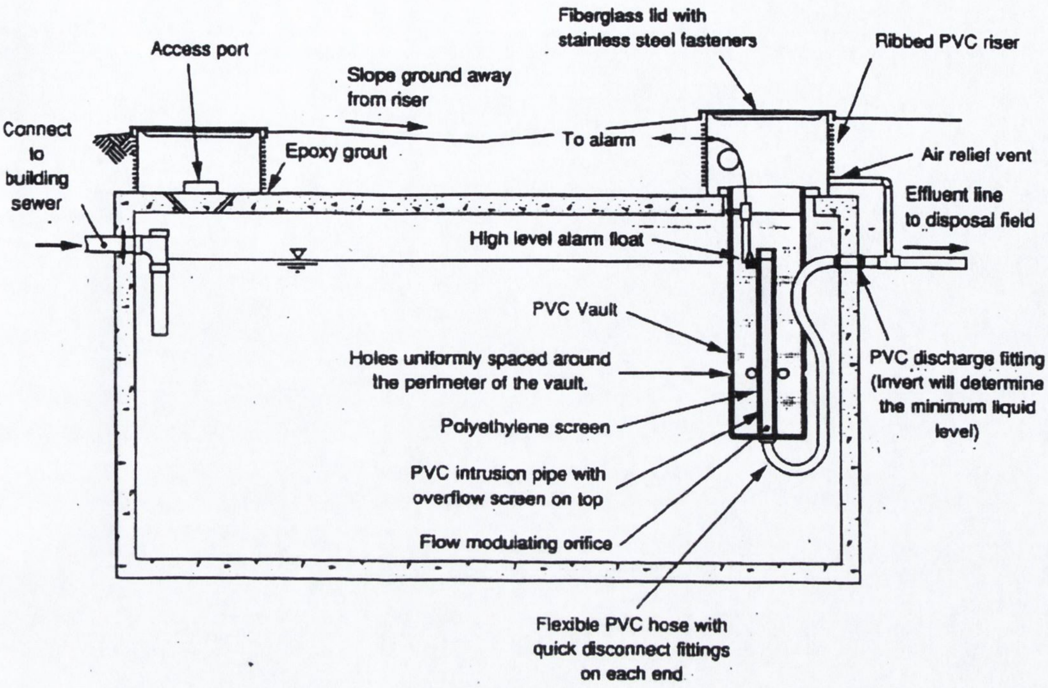


Figure 2.3 A single chamber tank equipped with filter vault (Metcalf and Eddy, 1991)

2.2.3 Septic Tank Operation

A septic tank provides quiescent conditions for settlement of solids and promotes the development of anaerobic conditions for treatment of both stored solids (sludge) and non-settleable constituents in domestic wastewater. The estimated biological and chemical characteristics of domestic wastewater in Ireland, which generally contains toilet flushings (black water or sewage) and washbasin, bathtub washings and kitchen waste (grey water), are highlighted in Table 2.2. The results of quantitative assessment of the physico-chemical quality of 28 septic tanks monitored in Co. Meath are shown in Table 2.3.

Concerns had been aired in the past that high concentrations of detergents and inorganic salts in grey water might upset the treatment processes within the tank and thus, in certain systems, the grey water is piped directly to a separate soil treatment system (Patterson *et al.*, 1971). However, current opinion generally dismisses this premise (Grant and Moodie, 1995) suggesting that the average concentrations of detergents and inorganic salts in typical household effluent will not adversely affect the

Parameter	Typical Concentration (mg/l unless otherwise stated)
Chemical Oxygen Demand COD (as O_2)	400
Biochemical Oxygen Demand BOD_5 (as O_2)	300
Total solids	200
Total Nitrogen (as N)	50
Total Phosphorus (as P)	10
Total coliforms (MPN/100ml)	10^7-10^8

*MPN Most Probable Number

Table 2.2 Characteristics of domestic wastewater from a single house (EPA, 2000)

Parameter	Concentration (mg/l)
BOD	264
COD	500
Suspended Solids	127
Ammonia	55.0
Total phosphorus	16.2

Table 2.3 Performance characteristics of 28 domestic septic tanks, regardless of tank configuration (adapted from Gray, 1995).

proper functioning of a septic tank. Consequently it is recommended that all wastes should be piped to the septic tank, although any type of runoff (rain from roofs, pavements etc.) should be diverted away from the tank in order to minimise the volume of waste to be treated, avoid dilution of the wastewater and prevent hydraulic overloading of the system. It should be noted, however, that research by Corey *et al.*, (1978), cited in Converse and Tyler (1994), reported that it is possible that salts, such as those from water softener backwash, can reduce infiltration in percolation areas where a biomat has not formed. It must be considered that this could lead to better distribution of secondary effluent, which has a low organic load, along the base of the percolation trenches.

There is a popular misconception that septic tanks treat domestic wastewater to a standard that does not further require treatment. They do, however produce a consistent effluent that is easy to treat. Septic tanks act primarily as a settlement chamber reducing the BOD and Suspended Solids (SS) content of wastewater which reduces the probability of clogging in the percolation area (Bouma, 1979). The solids are stored in the tank and the liquid supernatant is allowed to overflow into the percolation area for further treatment. The sludge in the base of the tank is constantly being degraded as a result of anaerobic decomposition and so the net rate of sludge build-up is considerably reduced compared with the theoretical solids accumulation based on a mass balance across the tank. Nevertheless, the volume of sludge does increase with time and requires removal at regular intervals. While the time interval for cleaning varies with factors such as tank size and number of occupants in the house it is usually between 1 and 4 years (S. R. 6: 1991), although once every 12-month period is recommended by the EPA (2000). A well constructed and maintained septic tank can remove between 15 - 30% of the BOD and retain between 50 - 70% solids. A typical effluent from a septic tank contains about 80 mg/l solids (Patterson *et al.*, 1971; Goldstein and Wenk, 1972; TCC *et al.*, 1998; EPA, 2000;). Research in Ireland on two sites by Henry (1990) recorded average suspended concentrations of 160 and 198 mg/l respectively while research by Keenan (1983), also in Ireland, recorded an average suspended solids concentration of 138 mg/l. However Canter and Knox cite research by Viraraghavan (1976) who found that BOD and Chemical Oxygen Demand (COD) removal efficiencies were in the order of 50% while total suspended solids removal was less than 25%. They also refer to a study by Lawrence (1973) which recorded less than a 15% reduction in BOD and a 34-35% reduction of suspended solids. The settled solids (sludge) on the floor of the tank are partially digested by anaerobic micro-organisms with the liberation of gases, principally carbon dioxide (CO_2) and methane (CH_4). Oils, greases, fats and soaps in the wastewater float to the surface aided by these gases, forming a thick scum over the liquid mass and providing an indication that the tank is functioning properly (Daly *et al.*, 1993). The degree of digestion depends on the size of the tank, frequency of cleaning and temperature (Payne and Butler, 1995). While Keenan (1983) and Henry (1990) did not measure the BOD reduction across the septic tank they reported average effluent concentrations of between 268 mg/l and 564 mg/l.

The anaerobic environment within the septic tank is largely ineffective in reducing the nutrient loading of the wastewater. Nitrogen in the influent wastewater is mainly in the form of organic nitrogen and ammonia, measured as total Kjeldahl nitrogen (TKN). Under anaerobic conditions, much of this organic nitrogen is converted to readily oxidisable ammonium ions (NH_4^+). A typical average wastewater influent TKN concentration is 38 mg/l [32% NH_4^+: 68% org-N] whilst average effluent TKN concentrations are recorded at similar levels around 40 mg/l [75% NH_4^+: 25% org-N] (Bauer *et al.*, 1979). Keenan (1983) reported an average ammonium concentration of 51 mg/l while Henry (1990) reported average concentrations 28mg/l and 44mg/l. Therefore, although the septic tank environment promotes the conversion of organic nitrogen to ammonium, it is ineffective with respect to total nitrogen removal across the process. It should be noted that nitrate concentrations in septic tanks are very low and usually zero, due to the lack of aerobic conditions.

The anaerobic digestion process also converts most of the influent phosphorus, in the form of both organic and condensed phosphate (polyphosphate), to soluble orthophosphate which passes out in the effluent. Bauer *et al.* (1979) also reported average total phosphorus concentrations in influent wastewater to septic tank systems serving single houses of 25 mg/l [35% inorganic P (orthophosphate): 65% organic-P]. Salvato (1992) reported average orthophosphate concentrations in septic tank effluent to be 15 mg/l. A typical total phosphorus concentration entering the percolation area is about 15 mg/l (Canter and Knox, 1985) of which about 85% is in the soluble orthophosphate form (Bouma, 1979 and University of Wisconsin, 1978). Keenan (1983) recorded average total phosphorus concentrations of 16 mg/l while Henry (1990) recorded average orthophosphate concentrations of 29mg/l and 50 mg/l. While septic tank influent quality in Ireland differs from the more generally dilute sewage in North America, (Gray, 1995), the processes involved in the septic tank are the same and so it would be reasonable to expect similar behaviour of the nutrients under Irish conditions.

Studies have also shown that removal of viruses, bacteria and micro-organisms within the tank is negligible (Patterson *et al.*, 1971; McCoy and Ziebell; 1975; Canter and Knox, 1985). Particulate organic matter, which contains complex molecules such as carbohydrates, proteins and lipids, are broken down under anaerobic conditions in the septic tank into simpler soluble compounds which will pass out of the tank with the effluent. The biological conversion of organic matter in the anaerobic environment of the septic tank occurs in three stages:

- the hydrolysis of insoluble high molecular-mass compounds into soluble organic compounds suitable for use as a source of energy and carbon,
- the bacterial conversion of these compounds into identifiable lower-molecular-mass intermediate compounds such as organic acids,
- the bacterial conversion of these intermediate compounds into simpler end products, principally methane and carbon dioxide.

It is clear, therefore, that a typical septic tank effluent is of poor quality, with a high pollution potential. It thus requires further treatment prior to discharge to watercourses or groundwater.

2.3 PERCOLATION AREAS

2.3.1 Design and Operation of Percolation Areas

Once the effluent leaves the septic tank it enters a distribution box from where it is channelled into an engineered distribution trench for discharge into the subsoil where it undergoes further biological, chemical and physical treatment. Attempts to distribute the flow equally between trenches or areas of a bed using distribution boxes and 100 mm diameter perforated drain pipe are commonly made, but have been shown to be ineffective (Otis *et al.*, 1978 cited in Siegrist *et al.*, 2000). The soil treatment system, or percolation area, is the most important component of the conventional septic tank system as it is here that the majority of treatment occurs. Research has shown that greater than 90% removal efficiencies can be achieved for organic constituents (BOD, COD and S.S.), micro-organisms and viruses by filtration, sorption and biodegradation processes. However, the removal of nutrients is more limited (USEPA, 1980; Jenssen and Siegrist, 1990; Van Cuyk *et al.*, 2001). Purification efficiencies in soil treatment systems can be very high yielding near complete removal of faecal coliform bacteria and greater than 4 log (99.99%) reduction in viruses (Emerick *et al*., 1997 (cited in Siegrist *et al*., 2000); Stevik *et al*., 1999 and Van Cuyk *et al*., 2001). Viraraghavan and Warnock (1976) reported that the soil had the ability to reduce 75-90% of the total suspended solids, BOD, COD and soluble organic carbon in the STE.

It is important, at this juncture, to distinguish between percolation areas and soakaways as they are often confused. A soakaway or soakage pit is basically a deep hole in the ground into which the STE flows for disposal. Since effluent is released over a small area there is a danger of clogging, causing ponding, or the treatment capacity of the soil being exceeded leading to groundwater contamination by untreated effluent. By the very mode of their construction (i.e. excavation) the depth of the treatment medium (subsoil) between the source and target (groundwater) is being depleted. Soakaways are not, therefore, "a satisfactory alternative to percolation areas" (SR:6, 1991) and should not be used. Typically a percolation area consists of a series of narrow, relatively shallow trenches or mounds filled with a porous medium (usually gravel). The following equation is used for drainage trench area calculations (Payne and Butler, 1993):

$$A = P \times V_p \times 0.25$$

where,

A = Floor area (m^2) of subsurface drainage trench

P = The number of people served by the tank

V_p = the percolation value of the subsoil (sec/mm)

A typical trench system is shown in Figure 2.4. Trenches are shallow, level excavations usually about 800 mm deep and 450 mm wide. The bottom is filled with 20-30 mm washed gravel over which the percolation pipe, maximum length of 20 m per trench, is laid at a 1 in 200 slope. The pipe is then surrounded by more of the gravel which is covered with a suitable semi-permeable geotextile to prevent backfill from clogging the aggregate. About 20 m of 100 mm piping is required per person (EPA, 2000). The porous medium is used to:

- maintain the structure of the trenches;
- provide partial treatment of the effluent by facilitating biomat formation;
- distribute the effluent to the infiltrative soil surfaces and
- provide temporary storage capacity during peak flows.

(Kreissel, 1982)

Research by Amerson *et al.* (1991), cited in Converse and Tyler (1994), reported that it appeared that the dust from the aggregate used in percolation trenches was a major factor in changes in infiltration. Van Cuyk *et al.* (2001) argued that the use of gravel in percolation trenches can have a detrimental effect on the infiltration zone permeability by:

- blocking pore entries;
- becoming embedded in the soil matrix;
- yielding fines that are deposited in pore entries or
- by focusing wastewater constituents as a result of the reduced permeability of the above effects.

Based on these potential adverse effects of gravel on infiltration capacity it was calculated that aggregate free percolation trench area could be constructed with an area 40% to 50% less than that required for gravel systems (Siegrist *et al.*, 2000).

Percolation areas disperse the effluent over a large area close to the ground surface, as far as possible from the watertable and bedrock. Hence, the treatment capacity of the subsoil is maximised and the risk of ponding is reduced where the infiltration and/or permeability is low. Proper design of percolation areas is critical to the

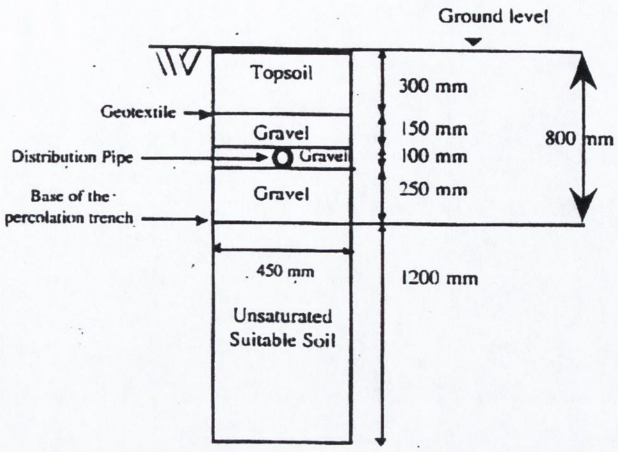


Figure 2.4 Section of a percolation trench (EPA, 2000).

successful operation of septic tank systems. Work by Cotteral and Norris (1969) and Laak *et al.* (1974) identified that the most important factors affecting the performance of a soil absorption system are subsoil properties, biomat formation and loading rate. Subsoil properties will be examined in detail in Chapter 4.

2.3.2 Biomat Formation and Development

The performance expectations of subsurface wastewater infiltration systems include long-term wastewater infiltration and adequate wastewater renovation prior to groundwater recharge (Siegrist and Boyle, 1987). Soil clogging, a phenomenon known to occur as a result of wastewater infiltration, is a function of the organic and solids loading rate from the effluent. Siegrist (1987) lists the main soil and wastewater characteristics that influence the occurrence and rate of progression of soil clogging as: soil temperature, moisture content, aeration status, applied effluent composition, hydraulic loading rate and method of application. Soil clogging has the affect of reducing the hydraulic conductivity and in many cases it is the hydraulic conductivity of the biomat, and not the subsoil, that becomes the controlling variable in wastewater infiltration rates (McGaughey and Winneberger, 1964 – cited in Wilhelm *et al.*, 1994a; Converse and Tyler, 1994). If the native sediments are finer-grained than the gravel surrounding the distribution pipe, a 2 to 5 cm thick mat forms over time below the gravel (Anderson *et al.*, 1982 – cited in Wilhelm *et al.*, 1994a). However, where a subsoil receives a highly treated effluent (BOD and SS < 20mg/l) a biomat will not form because the amount of organic matter is very low (Converse and Tyler, 1997). This affects the infiltration rate with research showing that secondary treated effluent, sand filter effluent in this case, infiltrated at rates 7-12 times greater than septic tank effluent (Loudon *et al.*, 1998). Converse and Tyler (1994) concluded that highly pre-treated wastewaters could be applied at rates 2-6 times greater than that recommended for septic tank effluent and possibly at rates equal to the soil saturated hydraulic conductivity. They also report (1997) that there is undocumented evidence to suggest that infiltration rates may increase due to increased biological activity from worms and other organism activity due to the aerobic nature of secondary effluent.

The clogging zone or biomat forms at the soil–gravel interface along the base and wetted sides of the percolation trench as suspended solids, equal to or larger than the soil pore size, are trapped. While some degree of soil clogging can enhance wastewater treatment through physical/chemical and biochemical process severe clogging can lead to hydraulic dysfunction, anoxic soil conditions and diminished wastewater treatment (Otis, 1985; Siegrist, 1987). Bacteria and other micro-organisms start to grow on the particulate matter due to extensive and lengthy contact between the wastewater constituents and the porous matrix (Figure 2.5). It is within this zone that the majority of the biological activity occurs, and it is here that the processes of decomposition of suspended materials, bacterial build-up and decomposition of organic material by bacterial action continually modify the infiltrative capacity. Mats have been observed to retain as much as 99.9% of the original coliform population over a distance of less than a foot (McCoy and Ziebeil, 1975). The zone is formed by three distinct phases:

- Physical: where solids in the effluent physically clog the soil pores;
- Chemical: where soil colloids swell as a result of chemical processes
- Biological: where bacteria or bacterial breakdown products reduce pore size.

(Patterson *et al.,* 1971)

Research by Orlob and Butler (1955) on five Californian soils concluded that the infiltration capacity of the soil absorption systems was controlled by the nature of the biomat and not the permeability of the soil. Although the biomat penetrates into the soil subsurface, the majority of it is located on the surface of the soil. This leads to reduced permeability, more uniform infiltration and a concomitant unsaturated flow almost regardless of hydraulic loading (Van Cuyk *et al.*, 2001). Wastewater induced clogging increases the soil biogeochemical activity and can enhance, sorption, bio-transformation and die-off/inactivation processes (Siegrist, 1987; Siegrist *et al.*, 1991). The physical method of straining, whereby the movement of material greater than void size is inhibited, also forms an important treatment process within the biomat. The biomat plays an important role in reducing the numbers of faecal bacteria in the percolating effluent (McCoy and Ziebell, 1975) and has also been reported to effectively remove other effluent constituents by various sorption reactions (Laak, 1974; McCoy and Ziebell, 1975 and Miller and Wolf, 1975). Removal of pathogens and other constituents may be less than predicted or desired if the clogging zone development is retarded. Such retardation could occur if the influent to the percolation trench is of a highly treated quality (secondary treatment effluent, for example). Conversely, excessive clogging can be detrimental causing hydraulic dysfunction, anaerobic soil conditions and reduced purification (Van Cuyk *et al*, 2001). It should be noted, however, than localised anaerobic conditions develop due to the high O_2 demand and moist conditions Wilhelm *et al.*, 1994b).

Over time the biomat develops a dynamic equilibrium in properly loaded and maintained systems. The rate of biomat development has been correlated with the amount of suspended solids and O_2 demand in the effluent (Siegrist and Boyle, 1987). As effluent solids accumulate, leading to formation and growth of the biomat, mineralised constituents and particulate material which have been reduced in size are carried away with the percolating water and gases produced from biological conversion of the waste are released (Metcalf and Eddy, 1991). Simultaneously, the development of the biomat can lead to progressive ponding in the trench due to decreased infiltration. The increase in hydraulic head associated with this increase in ponding depth has been shown to compensate for the increased resistance to infiltration (Siegrist and Boyle, 1987). This demonstrated that aggregate depth was important to maintain the hydraulic performance of the percolation trench with increasing ponding. Siegrist et al. (2001), however, argue that most systems operated under continuous use at a design rate of 10 to 50 l/m^2/d will eventually clog to such a degree whereby hydraulic failure will occur.

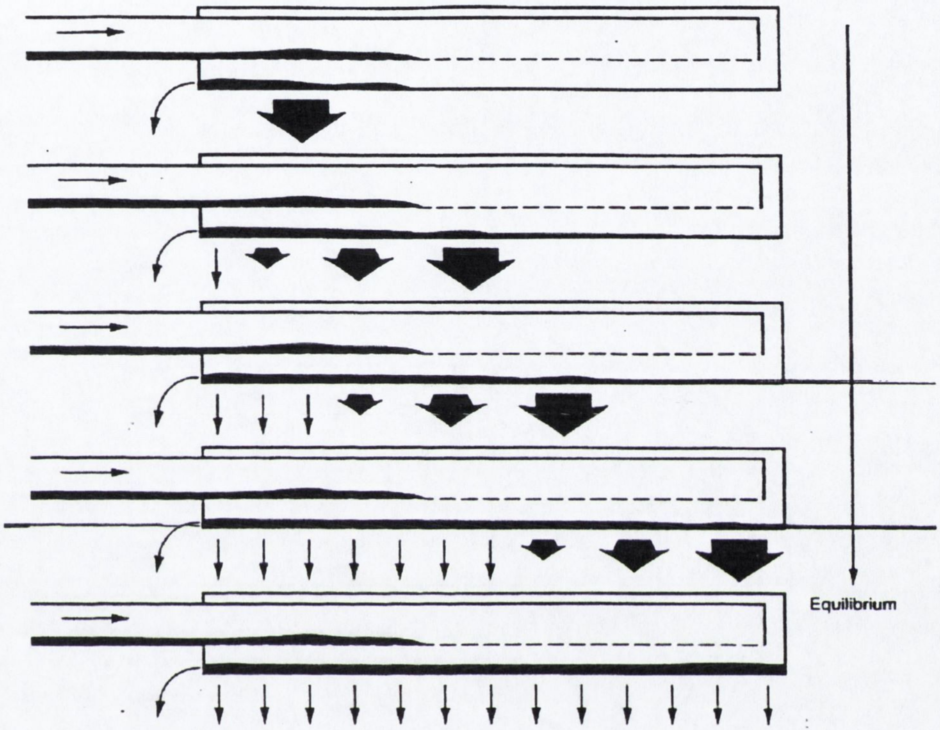


Figure 2.5 Progressive development of a biomat in a percolation trench (Kreissel, 1982)

2.3.3 Loading Rates

Satisfactory performance of percolation systems requires careful consideration of applied wastewater composition and hydraulic loading rate to prevent excessive clogging. The filter field loading rates are limited by either the hydraulic conductivity of the soil or the interaction of the biomat and soil at the trench interface (Bouma, 1975), Lack of consideration of these parameters can lead to hydraulic dysfunction and diminished wastewater treatment. Canter and Knox (1985) report on three types of loading regimes that can be utilised: continuous ponding, dosing and resting, and uniform application without ponding. Continuous ponding has the effect of increasing the effective infiltrative area by submerging the sidewalls of the trench and increasing the hydraulic gradient across the infiltrative surface which may increase the infiltration rate. However, this method leads to anaerobic conditions in the trench which can cause subsequent problems in terms of both hydraulic flow and biological decomposition. Dosing and resting overcomes some of the problems associated with the continuous ponding method. This is where a "reserve" percolation area is built so that it is possible to rest each percolation area in turn for several months at a time. This has the effect of encouraging reaeration of the trench and degradation of the clogging mat thus prolonging the effective life of the system (Otis, 1985). In the uniform application without ponding method, the liquid is distributed over the entire infiltrative surface at a lower rate than the soil infiltrative capacity thus preserving unsaturated aerobic conditions and minimising resistance of the clogging mat. In Ireland, the EPA recommend that the percolation trenches receive an even flow of effluent at a loading rate of 20 $l/m^2/d$ which takes into account the effect of the biomat on subsoil infiltration and ponding.

While a high degree of treatment normally occurs in the biomat at current loading rates higher hydraulic loading rates and non-uniform distribution methods can result in a malfunction in the system. Research by Owens et al. (1997) highlighted the failure of a serially loaded septic tank percolation area due to continuous anaerobic conditions resulting from continuous saturation by effluent. Many studies have shown that a large percentage of bacteria remain near the infiltrative surface when effluents are applied to a porous media. However, if hydraulic loading rates are too high or the dosing frequency is too low, microbes can be transported to lower regions in a soil matrix, posing a treatment concern in systems with a shallow water table (Van Cuyk *et al.*, 2001). Siegrist and Boyle (1987) reported that the findings of research they carried out was consistent with prior research (Jones and Taylor, 1965; Laak 1970; Hargett, 1982; Kristiansen, 1982; Pell and Ljunggren 1984 and Siegrist *et al.,* 1985) demonstrating that soil clogging development was accelerated both: at higher hydraulic loading rates with a constant effluent; and with increasingly concentrated effluents at a constant hydraulic loading rate, leading to system failure.

2.4 Failure of Septic Tank Systems

The ability of septic tank systems to effectively treat domestic wastewater to a degree that minimises risk of contamination of water resources depends on suitable site selection, competent system design and effective system maintenance. Problems associated with septic tank systems are common and generally due to insufficient attention to detail in any one of the areas alluded to above, as summarised in Table 2.4.

Problem	Immediate Cause
Odour	Inadequate ventilation of drains Blocked drainage field Inadequate drainage field
Backing up of sewage and surface flooding	Sagging or blocked inlet drains Blocked drainage field Inadequate drainage field Tank full of sludge
Solids discharged from tank	Tank full of sludge Inefficient or undersized tank
Local watercourse pollution	Blocked drainage field Inadequate drainage field Tank full of sludge Deliberate overflow connection made Proliferation of tanks discharging to land which quickly drains to watercourses
Groundwater pollution	Drainage field operating properly but system in unsuitable location Proliferation of tanks in sensitive area
Tank full of groundwater/tank lifts	High water table

Table 2.4 Symptoms and immediate causes of septic tank system problems (after Payne and Butler, 1995)

3. ALTERNATIVE SECONDARY TREATMENT SYSTEMS

3.1 Introduction

Where a site has been deemed unsuitable for the construction of a conventional septic tank treatment system the installation of a secondary treatment system can be considered. These systems may take the form of mechanical aeration systems, filter systems and constructed wetlands. Of interest to this project, and discussed in this chapter, were the package treatment plants which come in the form of mechanically and passively aerated modules. These provide secondary treatment of septic tank effluent or, as in the case of mechanical aeration systems, are an alternative to the septic tank treatment system. They are always succeeded by a polishing filter and, where used as an alternative to septic tank systems, are preceded by a primary settlement tank (EPA, 2000). The primary settlement tank provides for separation and retention of settleable solids and floatable materials, reduces the size of the subsequent biological treatment stage by up to 50%, reduces oxygen demand and thus the power requirement in mechanical aeration systems and buffers against shock loads (Metcalf and Eddy, 1991). Typical removal efficiencies of 50-70% suspended solids and 25-40% BOD are achievable. The primary purpose of a polishing filter is to reduce the concentration of micro-organisms in the treated wastewater prior to disposal (EPA, 2000).

3.2 Package Treatment Systems

Secondary treatment systems are designed to provide a controlled environment for the accelerated microbial degradation of organic matter and, in some cases, nutrients. In the case of mechanical aeration these systems, where designed for single dwellings, usually contain a primary settlement tank, aeration tank and secondary settlement tank (clarifier) in a fully enclosed compact unit. Passively aerated systems, which generally just contain modules of aerated media, are installed downstream of the septic tank and effluent is discharged to a polishing filter, usually the subsoil. In the aerated chamber carbonaceous organic matter (represented by CHONS) in wastewater is converted by heterotrophic micro-organisms into microbial biomass, thereby releasing both nutrient containing compounds along with carbon dioxide in general accordance with the following stoichiometry :

$$COHNS + O_2 + \text{nutrients} \longrightarrow CO_2 + NH_3 + C_5H_7NO_2 + \text{other end products}$$

$$C_5H_7NO_2 + 5O_2 \longrightarrow 5CO_2 + 2H_2O + NH_3 + \text{energy}$$

(Metcalf and Eddy, 1991)

The microbial biomass is then separated from the treated wastewater by sedimentation in a settlement tank where it can be periodically drawn off as sludge from the base of the tank. Nitrification, the conversion of ammoniacal products to nitrate by autotrophic bacteria, is only achievable in the aeration chamber once most of the carbonaceous matter has been assimilated by heterotrophic bacteria. The nitrifying autotrophic bacteria are much slower growing than the heterotrophs and require a suitable environment downstream of the main carbonaceous removal zone where organic loads are low and both dissolved oxygen and ammonium concentrations are high. Nitrification is considered in more detail in Section 4.2.3.

Phosphorus, once released from organic matter, will remain as soluble ortho-phosphate which will pass out in the effluent without any significant reduction (Fitzgerald, 1995). It is possible to remove orthophosphate from wastewater by either biological or chemical treatment processes, however, both methods require involved engineered treatment design and operational demands unsuitable for such small-scale applications (Stocks *et al.* 1994). Secondary treatment plants do achieve a degree of phosphorus reduction by a combination of sedimentation of particulate constituents and bacterial assimilation. Typical removal efficiencies across such works are 50-65% total phosphorus and 10-20% orthophosphorus depending the type of secondary treatment process employed (Gill, 1999 : Smith, 1999).

Secondary treatment systems are generally classified according to the type of biological treatment process:

- Attached growth — biofilms form as clusters of cells adhered to an inert media, e.g. rotating biological contactor (RBC).
- Suspended growth — the bacterial culture is suspended in a "mixed liquor" and group together to form "flocs", e.g. sequencing batch reactor (SBR).
- Combination — an inert media is present in a flooded reactor resulting in a combination of attached and suspended growth treatment processes, e.g. biological aerated filter (BAF).

3.2.1 Attached Growth Systems

Attached growth systems consist of an inert medium on which micro-organisms grow as a biofilm with typical depths of 60-3000µm (Hawkes, 1983) in a complex ecosystem of bacteria, protozoa, fungi, invertebrates etc. Treatment is achieved by aerobic degradation of the wastewater constituents when they are brought into contact with the heterotrophic bacteria within this microbial biofilm (Figure 3.1). Hence, it is important that the media provides a maximum surface area to volume ratio for optimal

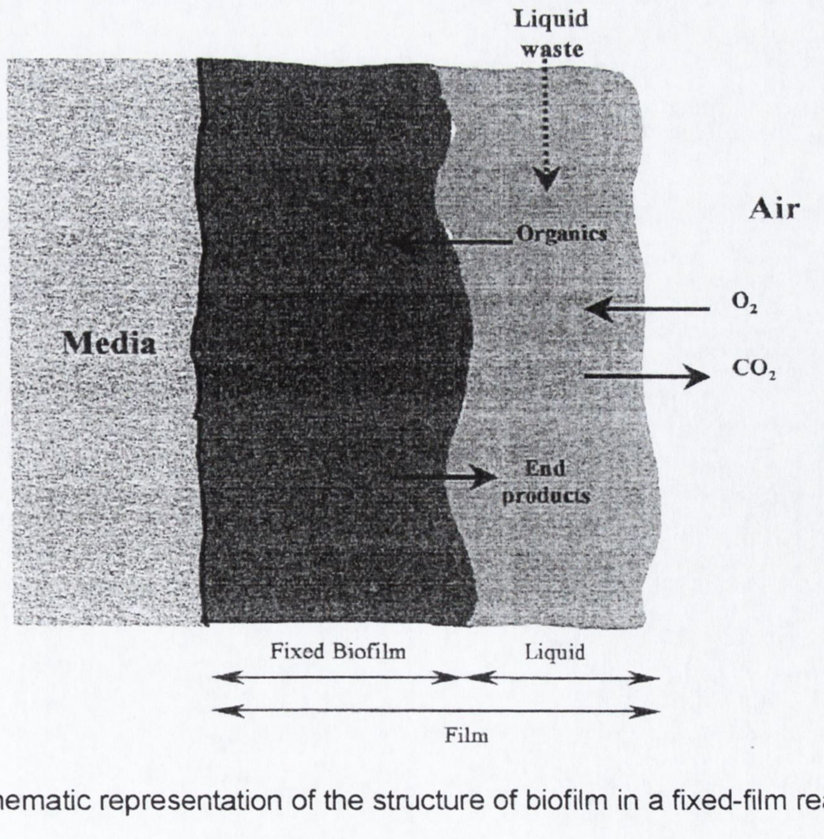


Figure 3.1 Schematic representation of the structure of biofilm in a fixed-film reactor.

biofilm-wastewater contact. As the film only develops on surfaces that receive a constant supply of nutrients, the effectiveness of the system in promoting maximum wetting of the media surface area is an important factor affecting performance (Gray, 1999). The first stage of treatment is the adsorption of organic nutrients onto the film. Fine particles in the wastewater are flocculated by extra-cellular polymers secreted by the attached heterotrophic micro-organisms and adsorbed onto the surface of the film, where, along with organic nutrients which have been physically trapped, they are broken down by extra-cellular enzymes excreted by the bacteria and fungi (Gray, 1999). Soluble nutrients in the influent and also those resulting from this extra-cellular enzymatic activity, are directly adsorbed by the biofilm and synthesised. The by-products of this aerobic degradation process diffuse out into the liquid wastewater phase.

The mass transfer of substrates (organic matter and oxygen) into and within the biofilm regulates the rate of reaction of the micro-organisms. External mass transfer involves the transfer of the substrates from the bulk liquid through a laminar fluid layer into the biofilm. If external mass transfer is the limiting factor in the microbial degradation process (for example, if the wastewater contains only a low concentration of organic matter), the observed reaction kinetics within the biofilm would be first order characterised by the following equation :

$$\frac{dX}{dt} = kX$$

where,

X = concentration of organic matter (mg/l)

k = first order rate coefficient (day^{-1})

(Mihelcic, 1999)

The biofilm development is thus directly proportional to the concentration of organic matter in the influent wastewater, and, increasing the influent organic matter to a level at which it no longer is the limiting factor to growth would result in an exponential increase in the microbial population.

Internal mass transfer is the resistance from the biofilm itself against substrate diffusion from the surface into the cell clusters. The thickness of the biofilm is critical in determining the internal rate of mass transfer with respect to dissolved oxygen which is continually being depleted as it diffuses through the thickness of biofilm. The depth to which oxygen will penetrate depends on a number of factors such as composition of the biofilm, its density and the rate of respiration within the film itself, and has been estimated to be anywhere between 0.06 and 4.00 mm (Gray, 1999). However, it is generally acknowledged that the critical biofilm thickness at which internal mass transfer becomes limiting, is approximately 0.15 mm (Shieh, 1982). Hence, if the biofilm is less than 0.15 mm thick it can be assumed that it is fully penetrated by oxygen and the entire biofilm is not oxygen limited maintaining aerobic conditions. This is the most efficient situation for such a secondary wastewater treatment process whereby the reaction kinetics are defined as zero order as characterised below:

$$\frac{dX}{dt} = k$$

An increase in influent organic matter concentration, therefore, will not result in increased biofilm activity. If, however, the biofilm is greater then 150 μm thick, oxygen becomes the limiting factor in microbial respiration and the reaction kinetics go from zero order to half order as shown.

$$\frac{dX}{dt} = kX^{\frac{1}{2}}$$

In this situation not all the organic matter is utilised and an anoxic, and subsequently anaerobic environment is established inside the biofilm moving towards the media.

As the biofilm increases in thickness, the adsorbed organic matter is metabolised before it can reach the micro-organisms near the media surface leaving no external organic source available for cell carbon. These micro-organisms then enter into an endogenous phase of growth where they are forced to metabolise their own protoplasm without replacement due to shortage of available substrate. They lose their ability to cling to the media surface and are subsequently washed off. As a result a new biofilm layer starts to grow. This phenomenon is known as 'sloughing' and is a function of both hydraulic loading, which accounts for localised shear velocities on the biofilm and variations in organic and nutrient loading, which account for biofilm metabolism (Metcalf and Eddy, 1991; Sawyer and Hermanoniczs, 1998).

An example of an attached film process used for small-scale wastewater treatment applications is the Rotating Biological Contactor (RBC) (Figure 3.2) which consists of a series of closely spaced (20-30 mm) flat or corrugated circular discs, normally plastic, on a slowly rotating horizontal shaft in a closely fitting contoured tank, often referred to

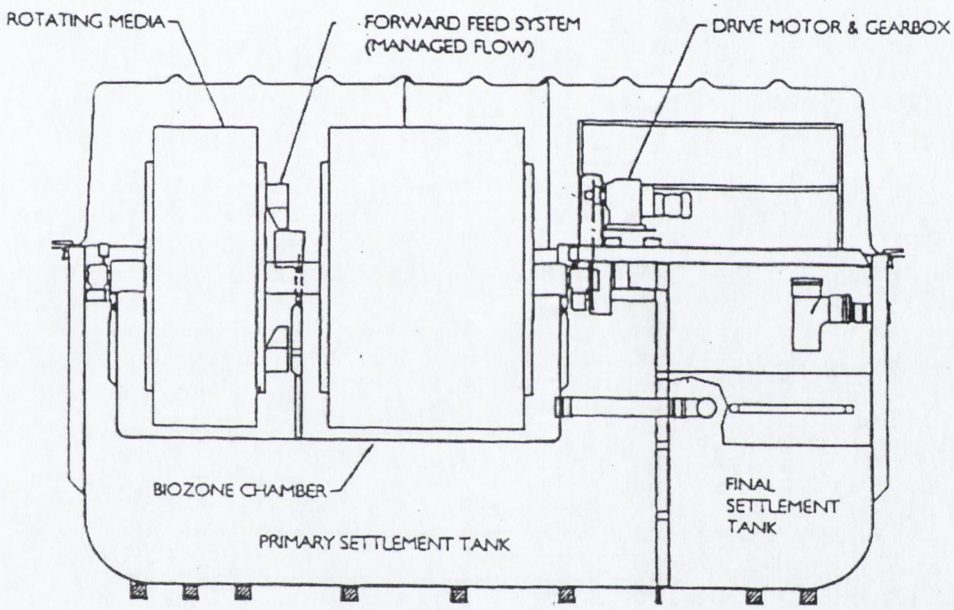


Figure 3.2 Schematic diagram of RBC system (Fitzgerald, 1995)

as the biozone (Metcalf and Eddy, 1991; Fitzgerald, 1995; Gray, 1999). Approximately 40% of the surface area of each disk is in contact with the sewage at any one time (Payne and Butler, 1993). The required surface area of the discs can be determined by using an organic loading rate of 5g BOD/m^2.d of settled sewage and a per capita loading of 40g BOD/d of settled sewage (EPA, 2000). The discs are mounted perpendicular to the effluent flow and rotate at about 1-2 rpm (Fitzgerald 1995). Some RBC designs incorporate primary settlement tanks while all include secondary settlement tanks, which require frequent desludging. Most units provide automatic intermittent desludging whereby the settled solids are pumped back to the primary settlement tank for storage with the primary sludge which should be removed every 2-3 months (Payne and Butler, 1993 and Fitzgerald, 1995). To ensure effective performance, RBC units should be covered to protect the biofilm from the weather, eliminate flies, control odour and reduce the load on the motor turning the rotor due to wind. Covering also insulates the system, reducing heat loss and increasing the rate of oxidation (Metcalf and Eddy, 1991; Gray, 1999).

Once the RBC is operational, micro-organisms attach themselves and grow onto the surfaces of the rotating discs, forming a biofilm. Once the biofilm is established the rotating discs have two main functions:

- they facilitate contact between the biofilm and the organic matter in the wastewater followed by contact with the atmosphere for adsorption of oxygen, and
- they create high hydraulic shear on the biofilm removing excessive solids from the discs – keeping a thin biofilm for maximum efficiency of penetration - and maintaining these sloughed solids in suspension so that they can be carried to the clarifier.

(Metcalf and Eddy, 1991)

The discs are usually arranged in groups in compartments separated by baffles to minimise short circuiting, reduce the effect of surges and to simulate plug flow conditions (Fitzgerald, 1995; Gray, 1999). The first upstream sections receive a higher organic load and thus produce higher levels of solids than later sections. Where nitrification is desired, it can be achieved by extending the number of compartments such that the organic loading is low enough in the final compartment downstream to provide favourable conditions for the slower growing nitrifiers. An example of an RBC available on the Irish market is the Biodisc® supplies by Klargester Environmental Ltd. Other examples of fixed-film small-scale package plants include the Envirocare Pb® and Bioclear® filter system using random packed plastic media and the Puraflo® system which uses a coarse peat mixture media.

PEAT FILTERS

Of especial interest to this project was the performance capabilities of peat based attached growth systems as it was decided to install this type of system on the two research sites specified for secondary treatment of domestic wastewater effluent. There were a number of reasons why it was decided to install this type of system, namely the Puraflo® system:

- It was purported to produce a good quality effluent,
- As it is passively aerated the absence of an aerator suggested a reliable, low maintenance system that had cheap running costs,
- The peat media had a estimated lifespan of 15 years ,
- It was awarded the Agrément Certificate which certifies that it is satisfactory for the purpose defined and meets the requirements of the 1991 Building Regulations,
- It was the system most recommended by consultants and local authorities who were contacted during the site identification phase of the project,
- Bord na Móna assisted greatly in the identification of sites and the provision of information on their system and the project area as a whole, and
- As the systems were purchased for research purposes, Bord na Móna offered a discount on the units.

Peat has been found to be an effective medium for the treatment of septic tank effluent. Rock *et al.* (1982) recorded an 83% reduction in COD and a 90% reduction in TSS across a peat filter. Research by Rock *et al.* (1984) carried out under laboratory conditions found that 30cm of sphagnum peat compacted to a density of 0.12 Mg/m^3 reduced the BOD_5 and SS concentration in STE by greater than 95% and 90% respectively. Further column tests by Viraraghaven and Rana (1991) showed BOD and TSS reductions of greater than 90% respectively. Rock *et al.* (1984) and Viraraghavan and Rana (1991) recorded COD removal of only 72% and 80% respectively but this was due to a COD contribution by the peat itself which was expected to decrease with time. Similarly Talbot *et al.* (1996) recorded an average COD reduction of 78%. Brooks *et al.* (1984) tested three sphagnum peat filters, two lined and the other discharging to the subsoil, under field conditions. They reported reduction in BOD_5 and COD of greater than 90% and 80% respectively. McKee and Brooks (1994) reported reduction of 89% or greater in BOD concentrations on 11 out of 12 peat systems tested. Research carried out by Lindbo and MacConnell (2001) and Monson Geerts *et al.* (2001) on Puraflo® systems recorded greater than 90% reduction in both TSS and BOD. Monson Geerts and McCarthy (1999) reported that peat filters installed at the Northeast Regional Correction Centre (NERCC) had consistently removed greater than 90% BOD and TSS. Talbot *et al.* (1996) recorded an average reduction of 97% and 92% in BOD_5 and TSS respectively. This reduction in organic load occurs in the top of the peat where the more aggressively growing heterotrophic bacteria are dominant while nitrification takes place deeper down in the peat provided aerobic condition prevail (Henry, 1996).

The 90% reduction in NH_4-N measured by Rock *et al.*, (1982) was reflected by a corresponding increase in NO_3-N. Viraraghavan and Rana (1991) recorded a 95% reduction in NH_4-N but never quantified the reduction in total N. While near complete nitrification was achieved within the system examined by Rock *et al.* (1984) there was less than 10% reduction in total N. It was found, however, that substantial (62%) denitrification occurred under anoxic conditions when there was a readily biodegradable organic carbon source present. Lindbo and MacConnell (2001) also found that while almost complete nitrification occurred within the Puraflo® system there was no reported denitrification. However, Monson Geerts *et al.* (2001) recorded a 41% reduction in total N during the winter and a 30% during the summer across a Puraflo® system. Under field conditions it was found that greater than 60% reduction in total N was achieved in three peat filters examined with denitrification suspected as the primary removal process (Brooks *et al.*, 1984). Research by McKee and Brooks (1994) showed an average total N removal of between 21 and 86% depending on the source of the peat. Analysis of the peat filter effluent at the NERCC showed between a 2% and 67% reduction in total N. While the peat medium is a carbon source, relying on this would result in its accelerated decomposition. However, it was suggested (Rock *et al.*, 1984) that STE not only provided the essential nutrients for denitrification under anaerobic conditions but also a more readily source of organic C thereby minimising the need to utilise the peat as a C source. This was corroborated in the field studies of Brooks *et al.* (1984).

Research by Rock *et al.* (1982) recorded an initial reduction in P across a peat filter of greater than 70% but this reduced to 32% in the third year of operation. However, it must be considered that this filter was underlain by sand. Apart from an approximate 10% reduction in the total phosphorus attributed to microbial assimilation no substantial removal of P by the peat medium was recorded in the laboratory trials by Rock *et al.* (1984). They concluded that P removal was dependent on the presence of Fe, Al and/or Ca in the peat. Similar reductions were measured by Viraraghavan and Rana (1991). Lindbo and MacConnell (2001) report little or no reduction in phosphorus concentration across Puraflo® modules examined. Similarly Monson Geerts *et al.* (2001) measured a reduction in P of between 3 and 11% which was probably due to microbial assimilation. Under field conditions it was found that, in three filters tested, a 58%, 62% and 96% reduction in total P was achieved. However, this was not attributed to the peat but to the protective layer of sand above the liner in the first two cases and the characteristics of the subsoil in the other case (Brooks *et al.* 1984). However, research by McKee and Brooks (1994) found that for 12 peat filters examined the peat was responsible for a 60 to 65% reduction in total P. Analysis of the peat filter effluent at the NERCC showed between 23% and 61% removal of phosphorus.

Research by Rock *et al.* (1984) showed substantial removal of total and faecal coliforms in 30, 60 and 90cm columns of peat dosed with STE. Rock *et al.* (1982), Brooks *et al.* (1984), Viraraghavan and Rana (1991), McKee and Brooks (1994), Talbot *et al.* (1996), Monson Geerts and McCarthy (1999), Monson Geerts *et al.* (2001) and Lindbo and MacConnell (2001) reported greater than 99% removal of total coliforms and faecal coliforms by peat filters.

While Henry (1996), Monson Geerts *et al.*, (2001) and Lindbo and MacConnell (2001) highlight the significant reduction in BOD, SS and enteric bacteria across the Puraflo® system it is clear that there is little reduction in the nutrient load. The aerobic environment of the Puraflo® promotes almost complete nitrification but does not facilitate denitrification. While significant P removal was recorded in some peat filters it is dependent on the presence of cations such as Fe, Al and Ca. It should be noted that the Puraflo® modules consist of a fibrous peat media that is the by-product of the fuel preparation process of the peat burning industry in Ireland. In this process the fibrous material in the harvested peat is removed while the rest of the peat matrix is used as a fuel. It is this part of the peat that is most likely to contain the cations required for phosphate removal.

3.2.2 Suspended Growth Systems

In suspended growth systems the microbial population and the wastewater are maintained in a mixed suspension under aerobic conditions. Aerobic conditions in the reactor are achieved by the use of diffused or mechanical surface aerators, which also serve to ensure complete mixing. The combination of adequate dissolved oxygen levels and high concentrations of organic matter in the influent wastewater produces a high rate of microbial activity and inherent organic matter degradation. The relationship between influent organic matter concentration and the concentration of micro-organisms in the reactor is fundamental to the successful operation of a suspended growth system (Figure 3.3). If wastewater with a high concentration of organic matter is placed in a batch reactor, the micro-organisms present require an initial start-up period to acclimatise to their nutritional environment, after which they

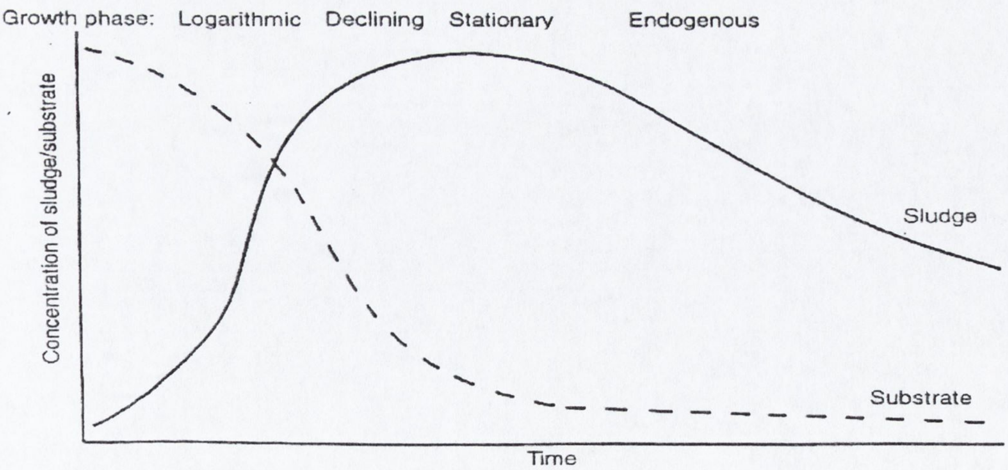


Figure 3.3 The microbial growth curve (adapted from Metcalf and Eddy (1991) and Gray (1999))

multiply rapidly in the presence of oxygen and nutrients (log-growth phase). This growth period coincides with the period of maximum organic matter removal. As numbers continue to increase, the rate of increase in microbial mass starts to decline due to limitations in the food supply. At this stage the micro-organisms enter an endogenous phase since there are not sufficient concentrations of available substrate. During this phase, a phenomenon known as lysis can occur in which the nutrients contained within the dead cells diffuse into the liquor supplying any remaining viable biomass with a food source. It is during this phase that nitrification can be initiated since the heterotrophic organisms are not able to compete effectively. Most suspended growth treatment processes are continuous flow systems and hence an equilibrium concentration of bacteria must be maintained in the reactor to achieve the optimum growth characteristics (and thus organic matter degradation). While it is important that the micro-organisms decompose the organic matter as quickly as possible, it is also important that they form a satisfactory floc, which is a prerequisite for the effective separation of the biomass in the clarifier. It has been observed that the settling characteristics of the biological floc are enhanced as mean residence time of the cells (sludge age) in the system increases (Metcalf and Eddy, 1991; Gray, 1999). If, however, the sludge age is greater than an optimum period, around 10 days, a reduction in settling characteristics can be observed due to the development of filamentous bacteria which effectively reduce the density of the flocs by branching between them and creating a more open structure.

Removal of organic matter by the suspended micro-organisms within the reactor involves three mechanisms:

- adsorption and also agglomeration onto microbial flocs;
- assimilation, and
- mineralisation.

(Gray, 1999)

It is possible, by varying operating conditions, to engineer the treatment system so as to favour either assimilation or mineralisation. With assimilation, which is the conversion of organic matter to new microbial cell material, organic matter is removed by its precipitation as biomass. This results in an increased requirement for sludge separation and disposal. If, however, conditions were to favour mineralisation, which is the complete oxidation of organic matter, the volume of organic matter would be reduced under endogenous respiratory conditions, resulting in lower sludge handling costs but higher aeration costs (Gray, 1999). After the aeration tank the wastewater flows into a clarifier where quiescent conditions allow for settlement of the sludge, most of which is returned to the aeration tank to maintain its microbial concentration. To maintain equilibrium conditions the quantity of sludge drawn off should equal new biomass growth. An example such a system currently available on the Irish market is the Enviropak® system supplied by Simon Allen Ltd.

The Sequencing Batch Reactor (SBR) is a fill-and-draw suspended growth treatment system in which all the unit processes are carried out sequentially in a single reactor (Metcalf and Eddy, 1991). Since the SBR provides batch treatment of wastewater, it can accommodate the wide variations in flow rates which are typically associated with single households (EPA, 2000). It is also possible to vary the length of the reaction cycles and amount of aeration in order to achieve nitrification and also enhanced phosphorus removal by switching between anaerobic and aerobic conditions. Table 3.1 (EPA, 2000) outlines the design criteria for an on-site SBR process for a single dwellings.

Parameter	Range
Total tank volume	0.5 - 2.0 times average daily flow
Number of tanks	Typically 2 or more
Solids retention time (days)	20 - 40
Aeration system,	Sized to deliver sufficient oxygen during aerated fill and react stage
Cycle times (hr)	4 – 12 (typical)

Table 3.1 Design criteria for the SBR process (EPA, 2000)

The SBR treatment process involves a five-step cycle:

(i) Fill Primary effluent enters the reactor allowing the liquid level to rise from about 25% of capacity (at idle) to 100%. Air diffusers can be on or off.

Normally lasts about 25% of the full cycle.

(ii) React During this stage the air diffusers are on allowing for the completion of aerobic reactions initiated in the fill stage. Typically takes up 35% of total cycle time.

(iii) Settle Air diffusers turned off allowing solids separation to occur, providing a clarified supernatant to be discharged. Normally more efficient than in a continuous flow systems as the reactor contents are completely quiescent.

(iv) Draw Removal of clarified wastewater from the reactor. Time required varies from 5 to 30% of the total cycle time.

(v) Idle In a multi-tank system it provides time for one reactor to complete its fill cycle before switching to another unit. Not an essential phase - sometimes omitted.

(adapted from Metcalf and Eddy, 1991)

Another important step that greatly affects SBR performance, although it is not included in the five basic process steps, is sludge wasting. Wasting is essential to maintain a given food-to-micro-organism ratio in the reactor in order to preserve the efficiency of the system. There is no set time period within the cycle dedicated to wasting as the amount and frequency of wasting is determined by performance requirements although, in general, sludge wasting usually occurs during the settle or idle phase.

3.2.3 Hybrid Systems

In a hybrid system the support media is fully submerged in wastewater in an aeration tank which receives oxygen from diffusers at the base of the tank (Figure 3.4). Owing to the high specific surface area of the media, large amounts of biofilm can develop and the biofilm that periodically sloughs off remains in suspension for a time thereby continuing the treatment of the wastewater, making these filters very efficient (Fitzgerald, 1995). In the water industry, there are two main types of hybrid systems referred to using the terminology, BAF (biological aerated filter) and SAF (submerged aerated filter). A SAF is a continuous flow system with settlement of the sloughed

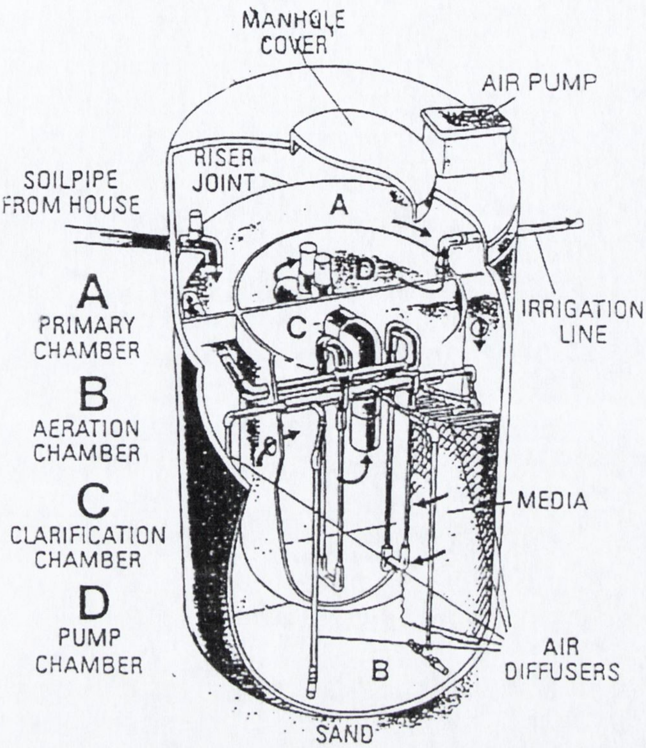


Figure 3.4 A biological aerated filter (BAF).

biomass achieved in a succeeding clarifier from which sludge is periodically drawn off. A BAF has no secondary clarifier and relies on a backwash sequence, typically once per day, to keep the biofilm thin enough to prevent sloughing and hence maintain high quality effluent. The backwash sequence involves taking the filter off-line and pumping clean effluent back through the media at high rates together with vigorous aeration. This has the effect of creating high shear velocities and also knocking the media particles together to loosen the excess biomass. This dirty washwater is then pumped back to the primary settlement tank where the solids co-settle with the influent wastewater. These BAF systems are more suitable for large-scale works where space is at a premium than the small-scale applications of interest in this study due to the period required off-line for the backwash (which requires a balancing tank or parallel BAF for continuous treatment), requirement for clean effluent storage tank, the complicated sequencing of the backwash and the overall extra energy requirement involved. Hence, for small-scale treatment systems (usually propriety package plants), the SAF system is adopted, although it should be noted that in some cases the term

"BAF" is used to describe any generic hybrid system (EPA, 2000). Examples of such package treatment plants include the Biocycle®, Septech 2000®, Biocrete®, Biofilter® and HiPAK® systems. The general recommended design of the surface area of media can be determined using an organic loading rate of 5g BOD/m^2.d of settled sewage and a per capita loading of 40g BOD/d of settled sewage (EPA, 2000) although each propriety system will have its own manufacture's specifications.

4. CONTAMINANTS AND SUBSOIL ATTENUATION PROCESSES

4.1 Introduction

Site selection is fundamental to the successful design and operation of domestic wastewater treatment systems and hence their capacity to contaminate groundwater and surface water, since further treatment of effluent from the engineered section of the system is dependent on the attenuation capacity of the subsoil. This chapter begins with an outline of subsoil characteristics, which includes an outline of the BS5930 classification scheme and a synopsis of the background to Irish subsoils. The factors and processes contributing to the contaminant attenuation in the subsoil and the main contaminants in STE are then examined. Finally, the contamination of groundwater in Ireland by STE is reviewed.

4.2 Subsoil Characteristics

As defined in the Groundwater Protection Scheme (DoELG *et al.,* 1999), *subsoils* are the most important feature in influencing groundwater vulnerability to pollution in Ireland (Misstear and Daly, 2000). They act as a protecting filter layer over groundwater, their effectiveness depending on type, permeability and thickness. Subsoils are the 'loose' unlithified sediments present between topsoil and bedrock. The word 'soil' is the general term used by engineers to describe all 'Quaternary deposits', 'drift' and 'overburden'. However, it is useful to distinguish between the topsoil (the upper metre or so affected by biological and weathering processes) and the underlying subsoil, as the latter is of most relevance in attenuating contaminants from on-site wastewater systems.

There are a number of ways (geological, soil science and geotechnical engineering) of describing properties of subsoils, i.e. the material and mass characteristics which give the subsoil name – particle size distribution, plasticity and dilatancy, density/compactness and discontinuities. The method used to describe subsoils during the course of this project will be the geotechnical approach based on BS 5930 (BSI, 1981).

4.2.1 Using BS 5930 to Describe Subsoils

Under BS 5930, the British Standard Code of Practice for Site Investigations, subsoils are described primarily on the basis of their material characteristics (which give the subsoil name) such as particle size distribution (including texture), plasticity and dilatancy; and the mass characteristics such as density/compactness, bedding and discontinuities (Daly and Swartz, 1999). The material and mass characteristics are factors relevant to the assessment of permeability and attenuation capacity of subsoil and thus to groundwater or surface water vulnerability. Work carried out by the GSI to-date suggests that it will be possible, with further research, to relate the subsoil name obtained from using BS 5930 to a broad permeability class. (Swartz *et al.*, 2003)

Particle size Distribution

A subsoil consists of rock particles, mineral grains and sometimes organic matter, together with variable amounts of air and water. The rock particles and mineral grains are split into groups depending on size, as shown in Figure 4.1. The relative proportion of sand, silt and clay particles in the subsoil is the most important factor influencing the permeability, i.e. fine textured subsoils have low permeability while coarse subsoils have high permeability, all other things being equal.

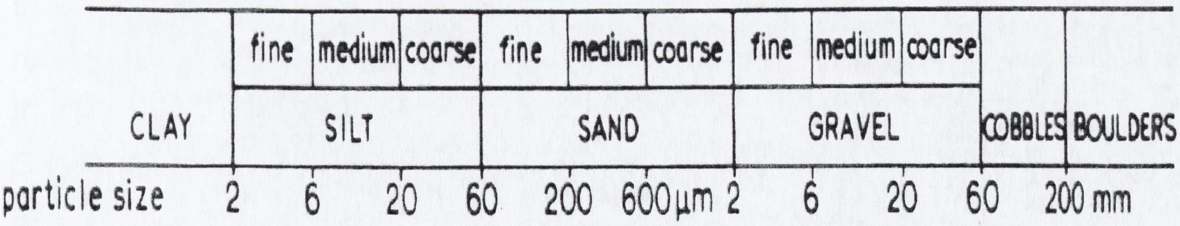


Figure 4.1 Main subsoil size groups described in BS 1377 (BSI, 1990).

Density/Compactness

The more dense or compact the subsoil, the lower the permeability.

Discontinuities

Discontinuities in the subsoil provide preferential flow paths for percolating liquids, reducing the attenuation capacity of the soil and thus increasing the likelihood of contamination. Typical discontinuities include fissures/cracks caused during deposition of the sediment; by weathering; by plant roots; by soil fauna or by parting due to the structure of the subsoil.

Bedding

Far from being a uniform matrix, subsoils often contain beds, laminations or lenses of different sediments. This heterogeneity can have a far reaching impact on water and contaminant movement in the subsurface. For example, if a low permeability clayey sediment underlies a coarse grained sediment of greater permeability the vertical movement of liquid through the subsoil would be hindered resulting in ponding or horizontal spreading of the liquid which could jeopardise water courses.

Plasticity

Plasticity is the ability of a material to deform, or change shape, without breaking when subject to an external force or pressure. The greater the plasticity of a subsoil, the higher the clay content, as one of the main properties of clay is that it is highly plastic.

Dilatancy

Dilatancy, which describes the reaction of subsoil to shaking, can be used to assess the relative silt and clay content of a subsoil. There are three terms that describe a sample's reaction to shaking or patting: rapid, slow and none. A sample has a rapid reaction if water quickly rises to the surface, making the surface shiny and wet looking. If the sample is subsequently squeezed, the shiny appearance goes away quickly. Samples have a slow reaction if it requires vigorous shaking to notice a change and there is little further change after squeezing. Generally sands and silts react rapidly in a dilatancy test while a clay will not react.

4.2.2 Background to Irish Subsoils

A high proportion of the lowland area of Ireland is covered with a significant thickness of Quaternary deposits. These have an important impact on the underground part of the hydrological cycle, either as aquifers or in the way they affect water moving into underlying rock aquifers. The Quaternary period is the most recent geological period, lasting from 1.6 million years ago to the present. It started as a period of global cooling in northern Europe, marking the onset of what is commonly known as the *Ice Age*.

The effect of the Quaternary deposits on the groundwater movement is largely a function of their permeability and, to a lesser extent, their thickness. Owing to their extensive nature and relatively high storage and in spite of their variable permeability, these deposits are an important source of baseflow in the summer, particularly in areas with no rock aquifers (Daly E., 1985). The Quaternary deposits of highest permeability are the various types of sands and gravels, namely glaciofluvial sands and gravels and esker sands and gravels. Where these deposits are sufficiently thick, extensive, saturated and clean they are considered to be aquifers in their own right. Where these deposits are not sufficiently extensive, or perhaps saturated, they are still important as they will allow a high proportion of recharge water to enter an underlying rock aquifer with which they are in hydraulic continuity. The onset of the warmer postglacial period (about 14,000 years ago) saw the disintegration of the ice sheets and the initiation of peat development in the resultant lake basins.

The main subsoils found in Ireland are:

Tills

Till, also known as boulder clay, is sediment deposited by or from glacier ice. It is the most commonly and wide-spread Quaternary subsoil type (TCC *et al.*, 1998). Tills are often tightly packed, unsorted, unbedded, possessing many different particle and clast sizes and types which are often angular or subangular.

Glaciofluvial Deposits

Glaciofluvial sands and gravels are deposited by running water and so represent the stagnation and decay of the ice sheets. The gravels are usually stratified and clasts usually have rounded edges, being polished rather than striated. These deposits, which give rise to a variety of different landforms, including eskers and kames, are generally thickest in areas close to the major halt stages of the various ice sheets. It is in these regions that most of the important sand and gravel aquifers have been located and are likely to be located in the future (Daly E, 1985).

Glaciolacustrine Deposits

Glaciolacustrine deposits, which consist of sorted gravel, sand silt and clay, are normally found in wide flat plains, or in small depressions in the landscape. Due to their mode of deposition they are usually associated with glacial lakes. Deltas, which are formed as sediment is deposited at a river mouth, usually contain interbedded sands and gravels, which dip lakeward, and are left as sand and gravel hills when the ice disappears and the lake drains away. Lacustrine basins, which are distal parts of the lake system, usually contain finer sediments, such as clay and silts.

Alluvium

Alluvium is a product of river flow and flooding and is usually of sand/silt grade. It may however contain gravel beds depending on its location. Because it occurs in river flood plains alluvium is not frequently encountered in septic tank installation, however any house built close to a river may be close enough to the flood plain for it to be considered as a potential site for a septic tank system (TCC *et al.*, 1998).

Peat

Peat, which once covered 16% of the land surface of Ireland (TCC *et al.*, 1998), consists mostly of vegetation which has only partially decomposed. This vegetation fills in marshes, ponds and other lakes carved out and left by Quaternary ice sheets. Thus, in Ireland, peat usually overlies badly drained glaciolacustrine silts and clays.

4.3 Factors and Processes Contributing to Contaminant Attenuation in the Subsoil

4.3.1 Physical Factors and Processes

(i) Permeability

For effective treatment of STE by the subsoil the permeability of the porous medium is critical as it controls flow of the percolating effluent and thus contact time between the STE and soil particles and associated biofilms. Laminar flow through a homogeneous isotropic saturated or unsaturated soil is described by Darcy's law (Domenico and Schwartz, 1998):

$$v = -K\frac{dh}{dl}$$

where,

v = the volumetric flow rate per unit surface area (m/s)

K = the hydraulic conductivity (m/s)

dh/dl = the hydraulic gradient

It should be noted that for unsaturated flow the total head includes the effect of suction.

For convective transport of pollutants in the subsoil, the linear velocity of the water rather than the Darcy velocity should be considered (Bouwer, 1984). The linear seepage of water in a porous medium can be approximated as:

$$v_m = \frac{v}{n_e}$$

where,

v_m = linear seepage (m/s)

v = Darcy velocity (m/s)

n_e = effective porosity, i.e. the porosity available for fluid flow

The hydraulic conductivity, which may be considered as a measure of the ease with which liquid flows through a given medium, is dependent both upon the physical properties of the flowing liquid and the characteristics of the transmitting medium. The physical properties of STE, i.e. viscosity, density and specific weight, are practically constant, so the hydraulic conductivity may be considered as a function of the permeability of the medium alone. The main soil properties that affect permeability are its degree of saturation and subsoil geometry.

Unsaturated flow through the vadose zone, i.e. between the percolation trenches and water table, is important in downward movement and attenuation of microbial and chemical pollutants. These unsaturated conditions, essential in terms of residence time of the STE and promotion of an aerobic environment for treatment processes, can be achieved by application of a suitable loading rate in the percolation trenches which is a minute fraction of the soil's saturated hydraulic conductivity. In the saturated zone the driving force for groundwater flow is hydraulic head, defined as the sum of the elevation head and the pressure head. When a soil is saturated, all of the pores are water-filled, pressure head is positive and conductivity is maximal. When the soil dries out some of the pores become air-filled and thus the conductive portion of the soil's cross-sectional area diminishes. These air-filled pores are assumed to act like solid particles inhibiting fluid flow (Hillel, 1998). As desaturation develops, the first pores to empty are the largest ones, thus confining flow to the smaller less conductive pores. These large empty pores must then be circumvented by the percolating fluid increasing flow path tortuosity. Thus, it is essential to remember that while Darcy's law can be used to describe flow in the unsaturated subsoil, the unsaturated hydraulic conductivity, $K\theta_v$, is not constant. Furthermore, under unsaturated conditions the pressure heads are less than atmospheric, i.e. water is held in suction – capillary forces bind water to the soil particles, and effluent flow, therefore, may occur either as film creep along the walls of wide pores or as tube flow through narrow water-filled pores. The closer proximity of the STE to the solid phase in the unsaturated material and the longer residence time can be expected to enhance removal of pathogens and chemicals from the percolating fluid.

Although there is not a universally accepted working relationship between grain-size and permeability, widespread research (Hazen, 1892; Loudon, 1952; Norris and Fidler, 1965; Masch and Denny, 1966; Summers and Weber, 1984; Shepherd, 1989; Schuh and Cline, 1991; Alyamani and Sen, 1993) has shown that a strong correlation exists between permeability and some representative grain diameter. While permeability of coarse gained sediments is greater than fine grain sediments, all other things being equal, grain-size alone does not dictate the permeability of the sediment as orientation of the particles, degree of sorting within the sediment and presence of preferential flow paths also have an effect. Research has shown that the permeability generally decreases for poorly sorted sediments (Hazen, 1892; Fraser, 1935; Carman, 1939; Krumbein and Monk, 1942; and Beard and Weyl, 1973). It must be noted with respect to unsaturated flow, however, that at low volumetric water contents the relationships that hold true in saturated flow may be invalid. For example, at lower volumetric water contents, coarse materials may have very few saturated pores and could thus have a lower unsaturated hydraulic conductivity than finer grained sediments which would have more saturated pores (Fetter, 1994).

PREFERENTIAL FLOW

Long continuous openings in the soil matrix have long been regarded as very important in the preferential movement of water through the soil profile. Preferential flow is a rapid transient physical phenomena occurring in the larger and predominantly vertical continuous soil pores (Di Pietro *et al.*, 2003). Preferential flowpaths also influence solute transport through natural soils (Larson, 1999). The importance of macropores in providing preferential flowpaths through the subsoil for percolating effluent is illustrated in Table 4.1. Williams *et al.*, (2000) outline various studies that

Soil Type	% Retention	
	Intact	Disturbed
Silt loam	78.0	99.8
Sandy loam	21.0	95.0

Table 4.1 Retention of *E. coli* in Kentucky soil cores (two soil types) either intact (macropores present) or sieved and mixed (macropores absent), based upon the ratio of the concentration of cells in the leachate compared to the concentration in the irrigation water (Wood, 1995).

showed that contaminants appeared faster at a given soil depth than would be predicted if the water flowed through the entire volume of soil due to the prominence of preferential flowpaths. It should be noted, however, that not all large voids are preferential flowpaths as some are hydrologically effective in chanelling flow through the soil while other are not. It is reported (White, 1985 and Kung, 1990) that flow along preferential paths is the cause of rapid movement of dissolved and suspended matter through soil. While the importance of macropores is confirmed by Weiler and Naef (2003) they found that of the macropores present in their study area only a few contributed significantly to preferential flow. Williams *et al.*, (2000) observed that the risk of groundwater contamination is increased by the presence of active preferential flowpaths. Kung (1990) found that preferential flowpaths were the dominant flow pattern in a sandy vadose zone monitored. Öhrström *et al.* (2004) observed dye movements through preferential flowpaths rather than the total soil matrix in an unstructured sandy loam. Beven and Germann (1982) report that the presence of preferential flow paths (pores formed by soil fauna, pores formed by plant roots, cracks and fissures and natural soil pipes) may lead to spatial concentrations of water flowing through unsaturated soil which would have important implications for the rapid transport of STE through the vadose zone with limited treatment. They also found that the impact of macropores is governed by the water supply to the macropores, the water flow in the macropores and the water transfer from the macropores to the surrounding soil matrix. Weiler and Naef (2003) conclude that the volume of water received by macropores alters the percolation depth and transport of solutes. Allaire-Leung (2000) reported on how the increase in tortuosity of macropores reduced the effect of these preferential flowpaths.

(ii) Filtration

Particle size distribution also plays an important part in the removal of suspended solids, including bacteria, from the STE by acting as an effluent filter. There are three filtration mechanisms (McDowell-Boyer *et al.*, 1986):

- Surface Filtration – occurs at soil surface when particles are too large to penetrate the soil resulting in biomat formation,
- Straining – particles small enough to enter the soil pores are removed by mechanical straining as the effluent percolates through the subsoil,
- Physico-Chemical Filtration - this occurs when very small particles, i.e. where the ratio of soil grain diameter to that of the particulate is greater than twenty, are retained if the attractive forces predominate when the particles collide with the soil.

Siegrist *et al.* (2000) refer to research by Updegraff (1983) who found that straining becomes an effective mechanism when the average cell size is greater than the grain size d_5 (d_5 is the particle diameter at which 5% of the particles in mass are smaller and 95% of them are larger). Hagedorn (1984) reports that bacterial travel is limited by physical straining or filtration, with the degree of retention inversely proportional to the particle size of the soil. However, Johnson and Atwater (1988) report on a study which found that a coarser–textured sand is just as effective overall as a loamy sand in removing coliform bacteria, although the fine-textured material is more effective in the first 15 cm. Filtering starts with the trapping of the larger suspended particles at the surface or at some depth. Individual particles may be blocked in the pores or several particles may interact to form a bridge in the pore that prevents further movement of these particles in the direction of flow. Once movement of the larger suspended particles has been blocked, these particles themselves begin to function as a filter and trap successively smaller suspended particles (Canter and Knox, 1985). Bouwer (1984) reported that bridging occurred when the diameter of the suspended particle was larger than 0.2 times the diameter of the particles constituting the porous medium. Depending on how particles in the porous medium were packed, bridging also occurred if the diameter of the suspended material was more than 0.07 times that of the particles in the medium. When the size of the suspended particles was less than 0.07 times the particle size of the medium, the suspended particles moved through the medium without bridging or blocking.

4.3.2 Chemical Factors and Processes

(i) Adsorption

Adsorption is a factor in the removal of phosphates, ammonium, organic compounds, bacteria and viruses from STE. It is an important phenomenon in soils that contain clay as the very small size of clay particles, their generally platy shapes and the occurrence of large surface area per given volume make them ideal adsorption sites (Gerba and Bitton, 1984). The iron, aluminium and hydrous oxides coating the subsoil clay minerals and magnesium-hydroxy clusters or coating on the weathered surfaces of ferromagnesium minerals provide excellent sorption sites (Miller and Wolf, 1975).

Adsorption is the physical and/or chemical process in which a substance accumulates at a solid-liquid interface (Mihelcic, 1999). It results from the differential forces of attraction or repulsion occurring among molecules or ions of different phases at their exposed surfaces (Hillel, 1998). During the process of adsorption a chemical species passes from one bulk phase to the surface of another where it accumulates without penetrating the structure of this second phase. Desorption refers to the reverse of the process of adsorption (Burchill *et al.*, 1981). Because adsorption removes contaminant from the fluid phase, even if only temporarily, it acts to slow the movement of the contaminant through the subsoil. The term "retardation" is thus commonly used to describe the effects of contaminant adsorption and a retardation coefficient assigned to the adsorbate (the substance being adsorbed). It is important to make the distinction between adsorption, which is a superficial attachment or repulsion, and absorption, which involves the transfer of a molecule from one phase to another, via their interface, resulting in the alteration of the composition of the second phase. Adsorption can be divided into two categories: chemical adsorption and physical adsorption - although as both can occur simultaneously in soils it is often impossible to distinguish between them.

Chemical adsorption, or chemisorption, involves valence forces of the type which bind atoms to form chemical compounds of definite shapes and energies (Burchill *et al.*, 1981). It tends to occur at specific adsorption sites, and does not proceed past the monolayer stage, i.e. all of the adsorbed molecules are in contact with the surface layer of the adsorbent. Chemisorption can progress as either an endothermic or exothermic chemical reaction, but strong chemical bond formation is often associated with large exothermic heats of reaction (releasing large quantities of energy to the environment). As an adsorptive molecule approaches the adsorbent surface, an energy barrier has to be overcome for reaction to take place; hence chemisorption, and desorption, usually involve an activation energy. Chemical reaction at a surface may actually prevent the original species ever being recovered.

Physical adsorption is a rapid, non-activated process which occurs at all interfaces. Transport processes, like diffusion or fluid flow to the interface, are rate-determining, the heats of adsorption are relatively low, and the chemical nature of the adsorptive species is essentially preserved in the processes of adsorption and desorption (Burchill *et al.*, 1981).

Clay particles are of colloidal nature and, when dry, neutralising counter-ions are attached to their surfaces. When wetted they display negatively charged surfaces that attract and adsorb cations (Bouwer, 1984). A hydrated colloidal particle of clay or humus forms a micelle, in which the adsorbed ions are spatially separated, to a greater or lesser degree, from the negatively charged particle. Together, the particle surface, acting as a multiple anion, and the "swarm" of cations hovering over it, form an electrostatic double layer (Hiemenez, 1986; Harter, 1986).

The cations of the double layer can be replaced by other cations introduced in the STE through a process called cation exchange which may be regarded as a form of adsorption (adsorption mainly applies to organic compounds while ion exchange applies to inorganic compounds). The smaller the cation and the higher its charge, the more strongly it is adsorbed and the more readily it replaces more weakly held cations (Bouwer, 1984). Of the more common cations in soil systems, Li^+ is the most weakly held, followed in order of increasing bond strength by Na^+, H^+, K^+, NH_4^+, Mg^{2+}, Ca^{2+} and Al^{3+}. Thus, if a percolating solution contained more Ca^{2+} than that in the original subsoil, the Ca^{2+} would replace K^+, Na^+, and the other more weakly bound cations held in the adsorptive mantle surrounding the clay particles until the cations adsorbed to the clay were again in equilibrium with the cations in solution. Cation exchange reactions are rapid and reversible; therefore, the composition of the exchange complex responds to frequent changes in the composition and concentration of a percolating solution. The composition of the soil's exchange complex in turn governs the soil's pH, as well as swelling and flocculation-dispersion tendencies (Hillel, 1998).

The total cation-exchange capacity (measured in milliequivalents of cations per 100 grams of soil) of a soil depends not only on its clay and organic matter content but also on the type of clay present. The cation-exchange capacity of clay minerals ranges from around 10 meq/100g for kaolinite to about 90 meq/100g for montmorillonite while the cation-exchange capacity of stable organic matter in the soil may exceed 200 meq/100g. The cation-exchange capacity of soil materials, therefore, generally ranges from essentially zero for sands and a few meq/100g for light textured soils to about 60 meq/100g for clay soils (Bouwer, 1984).

(ii) Precipitation

Precipitation is the separation of an insoluble product when two solutions are mixed together. It occurs in soils when the soluble ortho-phosphate ions (PO_4^{3-}) present in percolating STE, or sorbed onto soil colloids, react with ions in the soil solution. The nature of the product and the efficiency of this precipitation process, described as phosphate fixation, depends on the cations present and the pH of the soil.

In strongly acidic soils there are sufficient aluminium, iron and manganese ions in solution to cause the precipitation of all dissolved phosphate ions. Zanini *et al.* (1998) report that constant nitrification also generates acidity which can increase the number of cations present – in theory nitrification uses 7.14 moles of alkalinity (as $CaCO_3$) per mole of NH_4 oxidised. Because of its large specific surface (m^2/m^3) the freshly precipitated phosphate has a degree of solubility; this solubility decreases over time due to the growth of larger particles of precipitate at the expense of smaller ones as a result of the dynamic exchange between the precipitate and its dissolved ions.

In alkaline soils, phosphates quickly react with calcium to form a sequence of products of decreasing solubility, e.g. phosphate can react with calcium carbonate to form monocalcium phosphate, dicalcium phosphate and then tricalcium phosphate which is less soluble. Although tricalcium phosphate is quite insoluble, it may undergo further reactions to form even more insoluble compounds such as hydroxy-, oxy-, carbonate- and fluorapatite compounds (Brady and Weil, 2002).

4.3.3 Biological Factors and Processes

Anaerobic conditions can exist in the vadose zone, an otherwise aerobic region, due to its unhomogeneous anisotropic nature. This enables both aerobic and anaerobic biological transformations to occur, such as organic matter decomposition, nitrification and denitrification.

(i) Organic Matter Decomposition

Microbial decomposition of organic matter proceeds most rapidly in the aerobic zones where micoorganisms in the subsoil use the oxygen present as an electron acceptor during the decomposition of the substrate (McCarty *et al.*, 1984). Organic compounds in the percolating STE are subject to enzymatic oxidation resulting in carbon dioxide, water and energy production in a process that can be represented by:

$$R-(C,4H) + 2O_2 \longrightarrow CO_2 + 2H_2O + \text{energy}$$

Aerobic organisms cannot function in localised anaerobic zones within the subsoil treatment system so anaerobic or facultative organisms, such as methanogenic bacteria, become dominant. Since anaerobic decomposition is a slower process, pockets of partially decomposed organic matter can often accumulate in the subsoil. Anaerobic decomposition releases relatively little energy for the organisms involved and produces a wide range of partially oxidised organic compounds, such as organic acids, alcohols and methane gas (Brady and Weil, 2002), some of which have a detrimental effect on the subsoil "micro-environment" by inhibiting flora and fauna growth while others, notably methane gas, contribute to the greenhouse effect. Anaerobic decomposition is a three step process as follows :

(i)	Hydrolysis	conversion of insoluble high molecular-weight organic matter to soluble organic matter by hydrolytic bacteria,
(ii)	Acidogenesis	conversion of soluble organics to a range of organic acids and alcohols, hydrogen and carbon dioxide by acidogenic bacteria,
(iii)	Methanogenesis	conversion of hydrogen and ascetic acid to methane and carbon dioxide by methanogenic bacteria

(Brady and Weil, 2002)

The rate limiting step is considered to be the methanogenesis due to the slower growth rate and higher sensitivity of the methanogenic bacteria.

(ii) Mineralisation, Nitrification and Denitrification

The nitrogen content in STE (as described in section 2.2.3) is typically 70% to 90% ammonium and 10% to 30% organic nitrogen (Lance, 1972; Nilsson, 1990 (both cited in Siegrist *et al.*, 2000) and Gold and Simms, 2000). Organic nitrogen contains amine groups which are broken down by soil micro-organisms, by a process called mineralisation, into simple amino compounds which are then hydrolised releasing nitrogen in the form of ammonium (NH_4^+). The reduction in the ammonium concentrations in the STE as it percolates through the unsaturated subsoil is accompanied by an increase in nitrate (NO_3^-) concentration brought about by the process of nitrification. Nitrification can be limited by low temperatures, insufficient oxygen or by lack of alkalinity (Van Buuren *et al.,* 1999). Nitrification is a process used by only a few genera of autotrophic bacteria as a means to generate energy as the energy yield for the reaction is low resulting in very slow growth rates (Wood, 1995). It is a two step process with the first step, the rate limiting step, consisting of the conversion of ammonium to nitrite (NO_2^-) by *Nitrosomonas* bacteria, a specific group of autotrophic bacteria. This reaction is followed closely by the conversion of nitrite to nitrate by another group of autotrophic bacteria, *Nitrobacter*, resulting in a net increased soil acidity through the production of hydrogen ions (H^+).

If the percolating nitrate solution enters an anaerobic pocket, or zone of reduced oxygen concentration, where there is appropriate bacteria and a supply of readily available carbon in the form of organic substrate present, it will undergo denitrification. Denitrification involves a series of reactions used by a wide range of facultative anaerobic heterotrophs in which oxidised forms of nitrogen are used as alternative electron acceptors and nitrate is reduced to gaseous nitrogen (NO, N_2O or N_2). The proportion of the three main gaseous products seems to be dependent on the prevalent pH, temperature, degree of oxygen depletion, and concentration of nitrate and nitrite ions (Brady and Weil, 2002). N_2 is generally produced under anaerobic conditions while under conditions of low pH, reduced oxygen and high nitrite and nitrate it is generally nitric oxide gas (NO) and nitrous oxide gas (N_2O) that are produced.

4.4 Key Contaminant Groups in Septic Tank Effluent

4.4.1 Organics and Suspended Solids

Biodegradable organics in either dissolved or suspended form can be characterised by biochemical oxygen demand (BOD) or chemical oxygen demand (COD) which measure the amount of oxygen required for biochemical and chemical oxidation respectively. Volatilisation and adsorption, followed by microbial degradation are the main processes for removal of soluble biodegradable organics in the subsoil (Siegrist *et al.*, 2000). Suspended solids, including organic and mineral matter, can be removed through a combination of physical straining and biological degradation processes (Reed *et al.*, 1994). Laboratory columns simulating the unsaturated zone often remove 80 to 90% of organic C in STE when aerobic conditions are maintained suggesting aerobic oxidation and retention to be the main removal processes (Wilhelm *et al.*, 1994b). As outlined in Section 4.3.1, subsoils act as effective porous media biofilters in removing suspended matter within a specific particulate size range. The large specific surface of individual soil particles and subsoil organic matter provide high potential for biofilm development which is of great importance in the breakdown of organics in the percolating STE (Bouwer, 1984). Micro-organisms use O_2 as the electron acceptor in the oxidation of organic C to CO_2 (Wilhelm *et al.*, 1994a). Van Cuyk *et al.* (2001) report that the optimum biological degradation potential of the system is only achieved after an acclimatisation period of 2 to 3 months.

4.4.2 Inorganic Constituents

(i) Nitrogen

The fate of the nitrogen introduced in the STE is dependent on its initial form as well as biological and chemical activity in the subsoil treatment system. The removal mechanisms for nitrogen include mineralisation, volatilisation, adsorption, cation exchange, incorporation into microbial biomass, nitrification and denitrification. In a properly installed and operating system the predominant nitrogen retention reaction is ammonium adsorption while the predominant transformation reaction is nitrification (Siegrist *et al.*, 2000). The two forms of nitrogen that are of concern to the pollution of groundwater and surface water are ammonium and nitrate. Ammonium is toxic when present in high concentrations while nitrate presence in drinking water has been linked to methaemoglobinaemia (blue baby syndrome) in infants and it is also assumed to promote eutrophication in estuarine environments (Harman *et al.*, 1996).

Ammonium ions can be discharged directly from a septic tank to the percolation trench or they can be formed by the mineralisation of organic nitrogen, contained in the STE, in the upper layers of the soil system. Prolonged adsorption of NH_4-N appears most effective under anaerobic conditions, where nitrification is inhibited with Canter and Knox (1985) reporting that under the anaerobic conditions normally prevailing directly below the percolation trench, ammonium ions are readily adsorbed onto negatively charged soil particles. They are also adsorbed by organic colloids in the soil (Jenssen and Siegrist, 1988). After the adsorption capacity of the first few inches of soil is reached, the ions in the percolating STE will travel further to find unoccupied sites if anaerobic conditions persist. Cation exchange may also be involved along with adsorption in the retention of ammonium ions in the subsoil; however, just as the adsorption capacity of the soil can be exceeded, there are only a finite number of exchange sites available in the subsoil. The adsorption of NH_4-N can range from 2mg/100g in sandy soil to 100mg/100g in fine grained soils with a 30% clay content (Jenssen and Siegrist, 1988). Vermiculite and similar clays are shown to strongly fix NH_4-N (Wilhelm *et al.*, 1994a). The fraction of the cation exchange capacity (CEC) of the subsoil that may be used to adsorb ammonium depends on the concentration of the other cations in the STE as these cations, especially divalent cations such as Ca^{2+} and Mg^{2+}, compete with ammonium for exchange sites. It is important to note that the ammonium adsorbed by the soil CEC is only temporarily immobilised because it can be readily oxidised to nitrate in the presence of oxygen (Lance, 1984, Jenssen and Siegrist, 1988). However, several experiments have demonstrated that approximately 50 to 85% of fixed NH_4-N may be unavailable or only slowly available to nitrifying micro-organisms (Nommik and Vahtras, 1982). It can be difficult to quantify NH_4-N retention as it can be masked by oxidation and other interactions (Wilhelm *et al.*, 1994b). Ammonium can also be incorporated into microbial cell tissue or, if the pH becomes alkaline, volatilisation may occur, i.e. ammonia (NH_3) gas released, although nitrogen removal by these mechanisms is only minor (Lance, 1984; Canter and Knox, 1985).

Under the aerobic conditions of the subsoil treatment system ammonium will undergo nitrification. Micro-organisms in the subsoil use O_2 as the electron acceptor in the oxidation of NH_4-N to NO_3-N. Wilhelm *et al.*, (1994a) outline both field and column experiments which indicate that almost complete nitrification occurs within 1m of the distribution pipe and within a few hours of exposure to O_2. In some well-aerated systems, complete nitrification of effluent occurs within 0.5m of the biomat below the percolation trench (Whelan and Barrow, 1984). Jenssen and Siegrist (1988) report that nitrification, which is normally very rapid and is pH (7-8.5 desirable) and temperature (8-10°C desirable) sensitive, can occur in the first 30 cm of the subsoil. In properly constructed systems complete nitrification can be achieved (Siegrist *et al.*, 2000); however, one to two months are required from system start-up to generate a full population of nitrifiers (Van Cuyk *et al.*, 2001). As nitrification is temperature dependent, it is reasonable to assume that complete nitrification might not occur year round due to seasonal effects. Research carried out by Harman *et al.* (1996) in a sandy subsoil receiving STE from a school showed nitrate concentrations increasing from less than 0.1 mg(NO_3-N)/l in the septic tank to 112 mg(NO_3-N)/l at the water table while ammonium concentrations decreased from 128 mg(NO_3-N)/l in the tank to less than 1 mg(NO_3-N)/l at the water table. In this case, because nitrate levels observed at the water table were similar to effluent ammonium levels, almost complete nitrification of ammonium appeared to occur with little ammonium being removed by other processes. Wilhelm *et al.,* (1994b) report on research they carried out in which the concentration of nitrate at the watertable approximated 70% of the original N in the STE. Since nitrate is a negatively charged ion it is not attracted to the negatively charged soil colloids and as such is readily leached to the groundwater. Nitrate can be removed from the subsoil by denitrification, as discussed in Section 4.3.3. The occurrence of this process requires the presence of an anaerobic zone and an adequate biodegradable carbon source (Jenssen and Siegrist, 1988). However, due to aerobic oxidation the wastewater is usually devoid of sufficient organic C to promote denitrification in properly functioning septic tank treatment systems (Wilhelm *et al.*, 1994a and 1994b).

(ii) Phosphorus

Phosphorus is generally the limiting nutrient for algal growth in many aquatic ecosystems. The main source of phosphorus in STE is household detergents. Elevated phosphate levels can stimulate plant and algae growth which can eventually result in the eutrophication of surface water-bodies (Schindler, 1977). As most phosphorus is retained in the subsoil by adsorption and precipitation it is not generally a problem to groundwater in Ireland and is not likely to be in the future (Kilroy *et al.*, 1998). This is because the maximum admissible concentration for drinking water (2.2mg/l P) is rarely approached, except in areas of gross contamination. However, as P concentrations of only 20µg/l may trigger eutrophication groundwater may act as a pathway for P to surface water targets. As a result septic tanks have been implicated as a source of phosphate in some lakes where dwellings are located nearby although the evidence suggests that a significant amount of STE phosphorus is immobilised in the vadose zone sediments (Zanini *et al.*, 1998).

Phosphorus is mainly present in the STE as orthophosphate, dehydrated orthophosphate and organic phosphorus (Siegrist *et al.*, 2000). Bouma (1979) reported on studies that found that more than 85% of total phosphorus in the STE was in the soluble orthophosphate form. The organic phosphorus in the effluent can be subject to mineralisation, in a way similar to organic nitrogen, resulting in the production of soluble inorganic phosphate (Brady and Weil, 2002). Phosphate is generally immobilised within a few metres of the percolation trench and significant movement of phosphate is rare (Reneau *et al.*, 1989). As a result contamination of groundwater with phosphorus is seldom investigated or reported. However, studies have identified phosphorus leaching in areas with sandy soil, shallow water tables and in phosphorus saturated soils (Breeuwsma *et al.*, 1995). Phosphorus attenuation in the subsurface is controlled by soil adsorption and mineral precipitation reactions which can be considered in two general categories: initial adsorption reactions and much slower precipitation reactions that regenerate additional adsorptive surfaces occurring both from phosphate in solution and from phosphate previously sorbed (Lance, 1984).

The types of reaction that fix phosphorus in relatively unavailable forms differ from soil to soil and are closely related to soil pH. In acid soils these reactions involve mostly Al, Fe or Mn, either as dissolved ions, as oxides, or as hydrous oxides. In alkaline and calcareous soils the reactions primarily involve precipitation as various calcium phosphate minerals or adsorption to the iron impurities on the surfaces of carbonates and clays. At moderate pH values, adsorption on the edges of kaolinite or on the iron oxide coating on kaolinite clays plays an important role (Brady and Weil, 2002). Arias *et al.*, (2001) report that at pH levels greater than 6 the reactions are a combination of physical adsorption to Fe and Al oxides and precipitation as sparingly soluble calcium phosphates. At lower pH levels they suggest that precipitation as Fe and Al phosphates becomes increasingly important. It is clear, therefore, that the capacity of a subsoil to fix phosphate is dependent on the presence of Fe, Al, Mn and Ca. Robertson (2001) found that the organic matter in STE causes iron in soils to become soluble. As a result the phosphorus is precipitated out of solution forming a stable coating on the soil particles.

Soils vary greatly with respect to P-sorption capacity with a quartz sand receiving STE probably becoming saturated after a few months while the sorption capacity of a weathered sand or fine grained soil may hold for a period of 10 years or more (Jenssen, 2001). With respect to sands the P-removal efficiency is often high initially but then decreases after some time as the P-sorption capacity of the sand is used up (Arias *et al.*, 2001). In calcareous terrain, acidity generated by the oxidation reactions stimulates carbonate mineral dissolution resulting in near-neutral pH and Ca enrichment in the plume (Robertson, 2003). Analysis by Wilhelm *et al.*, (1994b) on STE in a calcareous subsoil recorded the sorption of PO_4-P followed by its precipitation with Ca as an amorphous mineral that later crystallised. As a result most of the P in the STE was removed in the first three months after leaving the septic tank. In general, phosphate retention is greater in acidic settings than in neutral or basic settings so the possible lowering of soil pH is not expected to increase phosphate mobility (Wilhelm *et al.*, 1994a). They therefore concluded that calcareous sand seemed to have a good ability to limit PO_4-P mobility. This corroborated previous findings by Whelan (1988). Phosphate attenuation on noncalcareous sites monitored by Robertson (2003) appeared to be as a direct result of the development of acidic conditions and elevated Al concentrations which subsequently caused the precipitation of Al-P minerals. The potential for P sorption of a porous medium is dependent on the mineral composition, which determines the metal types and content, and the degree of weathering of the particle surfaces, which renders the metals in an oxide or hydrous state where they are able to react with phosphorus compounds (Jenssen, 2001). It is also dependent on the grain size of the medium. Analysis by Zhu *et al.*, (2003) on the phosphorus sorption characteristics of a light-weight aggregate of expanded clay found that the finer the grain size the higher the P sorption. They also found that the influence of temperature on P sorption was greater for the coarser grained aggregate. While capacity for P sorption is finite, a given soil, however, does not have a fixed capacity to remove phosphorus as these reactions are dependent upon many factors such as the phosphorus concentration in the soil solution, soil pH, temperature, time and the concentration and type of ions in the STE (Lance, 1984).

Mineral precipitation reactions may be affected by flow velocity but are less likely to be influenced by loading history (Harman *et al.*, 1996). Lance (1984) found that by decreasing the hydraulic loading rate from 0.45 m^3/m^2d to 0.1 m^3/m^2d, phosphate removal in sand columns was increased from about 10% to about 80%. Harman *et al.* (1996) report on research carried out at three test sites in sandy subsoil, two in operation for six years and the third for twelve years. Phosphate attenuation in the unsaturated zone of both six year old sites was nearly complete with less than 0.1 mg/l being recorded at the water table. At the older site, however, removal of phosphate was found to be less than 50%. The research, carried out on a system in operation for 44 years, led them to the conclusion that the attenuation of phosphate in the subsoil was controlled by mineral precipitation rather than sorption because:

- a comparison of the maximum sorption capacity of the soil and the phosphate sorbed suggested that the sorption capacity of the soil had been reached,
- the phosphate concentration recorded at the water table over a three year period was relatively constant and significantly lower than the STE phosphate concentration suggesting steady-state conditions. If adsorption was the main

attenuation process phosphate concentration at the water table would increase as sorption sites were used up,

- sorption capacity did not change significantly within or below the phosphorus peak in the subsoil suggesting that the high concentration of phosphorus was more likely a result of a zone of phosphorus precipitation.

Precipitation may provide unlimited capacity to fix phosphate, provided that the solution and soil chemistry is conducive to the reaction (Jones and Lee, 1979).

4.4.3 Pathogens

With an estimated 200,000 wells and springs in use in Ireland (Wright, 1999) the survival of pathogens under unsaturated and saturated conditions is a major concern in the protection of groundwater resources. Pathogens commonly found in STE include enteric bacteria, at sustained concentrations, and viruses and protozoa, at highly variable and episodically released levels (Cliver, 2000). The most important pathogenic bacteria and viruses that might be transported to groundwater include Salmonella sp., Shigella sp., Escherichia coli and Vibrio sp., and hepatitis virus, Norwalk virus, echovirus and coxsackievirus (Abu-Ashour *et al.*, 1994). Viruses and protozoa are not continuously present at high densities, but rather are shed during disease events and thus the concentration in the STE can vary from zero to values of the order of 10^6 organisms/100ml or more (Siegrist *et al.*, 2000). While some reference is made to viruses and protozoa in this literature review they were outside the scope of the project as the main concern of the project was the persistence of enteric bacteria with subsoil depth.

Bacteria are single-cell prokaryotic organisms containing a colloidal suspension of proteins, carbohydrates and a cytoplasm. The cytoplasm contains ribonucleic acid (RNA) and deoxyribonucleic acid (DNA) which control protein synthesis and reproduction respectively. Reproduction is usually by binary fission, although some species reproduce sexually or by budding (Metcalf and Eddy, 2003). Pathogens commonly found in domestic effluent include some enteric bacteria which are gram-negative, facultative anaerobes with a wide variety of metabolic capabilities, including anaerobic respiration using NO_3^- as the electron donor, that inhabit the intestinal tracts of humans and animals (Campbell, 1993). Their presence in groundwater, therefore, is not conclusive of anthropic contamination. Where present, however, bacterial pathogenic organisms of human origin typically cause diseases of the gastrointestinal tract, such as typhoid, gastroenteritis and cholera (Table 4.2). As it is not feasible to test samples for the presence of all pathogenic bacteria samples are tested for the presence of indicator species under the premise that their presence is suggestive of the presence of human pathogens (Section 7.4.2).

Bacteria	Disease	Concentration in raw wastewater (MPN/100ml)	Infective dose	Survival time in soil at 20-30°C (days)
Campylobacter spp.	Gastroenteritis	10^0-10^3		
Escherichia coli (enteropathogenic)	Gastroenteritis	10^6-10^8	1-10^{10}	<120 but usually <50
Legionella pneumophila	Legionnaires' disease			
Leptospira spp.	Leptospirosis			
Salmonella spp.				
S. typhimurium	Salmonellosis	$10^2 10^4$	10-10^8	<120 but usually <50
S. typhi	Typhoid fever	$10^2 10^4$	10-10^8	<120 but usually <50
Shigella spp.	Shigellosis	10^0-10^3	10-20	<120 but usually <50
Vibrio cholerae	Cholera	10^0-10^3		
Yersinia enterocolitica	Yersiniosis			

Table 4.2 Bacterial pathogenic organisms present in domestic wastewater (adapted from Gray (1999), Kiely (1997), Mason (2002), Metcalf and Eddy (2003) and Viessman and Hammer (1998)).

The fate of pathogens in subsoils is primarily governed by their transport and persistence. Bacteria and virus survival in the subsurface environment is in the order of a week to several months (Wilhelm *et al.*, 1994a). Reneau *et al.* (1989) report that pathogenic bacteria and viruses in soil are generally reduced to minimal numbers within two to three months although enteric bacteria have been known to survive up to five years under some unusual environmental conditions. In subsoil treatment systems inactivation/die-off, filtration and adsorption can be extremely effective attenuation processes in the removal of pathogens from percolating STE. The development of a biomat improves the retention of pathogens (Hagedorn, 1984 and Sélas *et al.*, 2002). Hagedorn (1984) outlines research that found that in a properly functioning percolation area the indicator bacteria were almost completely removed after 38cm. He also outlined other research that found that approximately 30-90cm of soil beneath the percolation areas monitored was adequate for complete bacterial removal provided that the subsoil had both a layer permeable to effluent flow and another region adequately restrictive to form a clogged zone. Reneau *et al.* (1989) report that while most treatment occurs in less than 3m of subsoil under unsaturated flow conditions bacteria can be adequately removed within 0.9 to 2.0m of effluent flow through soils.

The main factors affecting the growth and survival of bacteria and the survival of viruses (which require a living host for reproduction) in the subsoil are summarised in Table 4.3 (a) and (b). The type of pathogen and its physical state, type and characteristics of the subsurface soil, wastewater quality, temperature and hydrogeological conditions influence the fate of pathogens in the subsurface environment (Bitton and Harvey, 1992 and Scandura and Sobsey, 1997). For

Factor	Comments
Moisture Content	Longer survival in moist soils.
Moisture holding capacity	Survival time is less in sandy soils with lower water-holding capacity.
Temperature	Longer survival at lower temperatures.
pH	Shorter survival time in acidic soils (pH 3-5) than in alkaline soils.
Organic matter	Increased survival and possible growth when sufficient amount of organic matter is present.
Biological interactions	Increased survival in sterile soil where there is a lack of competition from indigenous microflora. The presence of antibiotics also have a detrimental affect on their survival.

Table 4.3 (a) Factors affecting the growth and survival of bacteria (adapted from Abu-Ashour *et al.*, 1994 and Gerba and Bitton, 1984).

Factor	Comments
Moisture Content	Increased virus reduction in drying soils.
Temperature	Viruses survive longer at lower temperatures.
pH	Most enteric viruses are stable over a pH range of 3-9; survival may be prolonged at near-neutral pH values.
Cations	Certain cations have a thermal stabilising effect on viruses. May also indirectly influence virus survival by increasing their adsorption to soil (viruses appear to survive better in sorbed state).
Biological factors	No clear trend with regard to the effect of soil microflora on viruses.

Table 4.3 (b) Factors affecting the survival of viruses in the subsoil (adapted from Abu-Ashour *et al.*, 1994 and Gerba and Bitton, 1984).

example, the average penetration rate of a motile strain of *E. coli* through sand has been shown to be four times faster than that of a nonmotile strain of the bacterium (Reynolds *et al.*, (1989). Persistence of enteric bacteria in the subsoil is generally highest at low temperature, high soil moisture content and abundant organic matter that may allow the growth of certain bacteria (Bitton and Harvey, 1992), although the USEPA (1993) dispute this latter point. The USEPA (1993) cite the availability of nutrients and biological factors as most important for the survival of pathogenic bacteria with elimination faster at higher temperatures (37°C), at pH values of around 7, at low oxygen concentrations and at high levels of dissolved organic carbon, conditions that promote naturally occurring antagonistic bacteria. Soil temperature and moisture are the primary factors affecting virus survival in soils receiving STE with viruses surviving best in moist soils under low temperatures (Bitton and Harvey, 1992).

Bacterial transport through percolation areas is controlled by soil porosity and the degree of saturation with water (Bitton and Harvey, 1992). They may be transported over much greater distances under saturated conditions than under unsaturated conditions due macropore flow and to higher pore water velocities (Hagedorn *et al.*, 1981). As the moisture content of the subsoil decreases the maximum water-conducting pore size decreases. As a result organisms larger than 8μm are unlikely to move through water-filled pores in unsaturated subsoils, and viruses, therefore, are most likely to be transported (Powelson and Gerba, 1995). Removal of pathogens from the percolating STE is mainly due to filtration for bacteria, although adsorption does also occur, and adsorption for viruses (Gerba and Bitton, 1984). As most bacteria and soil materials are negatively charged the electrostatic repulsion present between them must be overcome for adsorption to occur. The sorption of cations present in the percolating effluent onto subsoil particles can neutralise the negative charge and bacteria can be retained by van der Waals forces (Peavy, 1978 and Powelson and Gerba, 1995). Adsorption appears to be significant in soils having pore openings several times larger than typical sizes of bacteria (Canter and Knox, 1985). In flow systems bacterial transport through porous media has been largely described as a function of pore entrance size (Reynolds *et al.*, 1989). While cell size and ionic strength of the effluent were shown to be important variables controlling bacterial transport through porous media grain size was seen to be the most important factor (Fontes *et al.*, 1991). Physical straining occurs when bacteria are larger than the pore opening in the soil. Bacterial removal by straining is inversely proportional to the particle size of soils. (Hagedorn *et al.*, 1981 and Bitton and Harvey, 1992). When these bacteria accumulate within the subsoil the pore space is reduced which has the effect of removing smaller bacteria from the percolating effluent. Research (Gerba and Bitton, 1984) in which E. coli suspended in distilled water was allowed to percolate through sand columns suggest that once the capacity for removal by straining has been satisfied, sedimentation is the only effective process.

Straining as a removal mechanism is less affective for viruses than for bacteria due to their small size, 18 to 25nm as opposed to 750 nm for bacteria (Reneau *et al.*, 1989; Bitton and Harvey, 1992 and Powelson and Gerba, 1995). Nicosia *et al.* (2001) report that soil characteristics affect virus removal and in general, virus sorption capacities increase as clay content, cation exchange capacity and specific surface area increase, and as organic content decreases. Adsorption of viruses onto soil particles is also dependent on the pH of the system. At low pH values below 7.4, virus adsorption by soils is rapid and effective but higher pH values greatly decrease the effectiveness of virus adsorption because of increased ionisation of the carboxyl group of the virus protein and the increasing negative charge on the soil particles (Canter and Knox, 1985). While the actual mechanism of viral adsorption is unknown, two general theories exist, both based on the net electronegativity of the interacting particles:

(i) that a clay-cation-virus bridge operates to link the two negatively charged particles, or

(ii) that the fixation of multivalent cations onto the ionisable groups on the virus particle is accompanied by a reduction of the net charge of the particle.

(Gerba, 1975)

In research carried out by Nicosia *et al.*, (2001) in fine sand using the bacteriophage PRD1 as a human enteric virus surrogate it was found that the sorption of PRD1 was reversible as the phage particles slowly desorbed over time. Desorption was also reported when solutions of lower ionic strength were flushed through soils (Reneau *et al.*, 1989). Brown *et al.* (1979) described a 2-year study where STE was monitored for coliphages in three undisturbed soils. Regardless of application rates the coliphages were rarely detected in soil leachates and 120cm of any of the soils tested appeared to be sufficient to minimise any possibility of groundwater contamination by coliphages.

Protozoa are a topical issue in relation to the use and treatment of surface water for human consumption although little has been written on their persistence in the subsoil or the threat they present to contamination of groundwater resources. They are of lesser concern than bacteria and viruses however since they are relatively large and therefore are removed more efficiently by subsoil filtration (Reneau *et al.*, 1989). Certain protozoa, excreted from mammals into the environment, form protective "oocysts" which allow them to survive for long periods (generally several months) in damp cool situations whilst waiting to be ingested by another host before continuing its life cycle (Gray, 1994). The two pathogenic protozoa oocysts most frequently found in surface drinking water sources are Cryptosporidium (4-7 µm in diameter) and Giardia lambia (8-14 µm long and 7-10 µm wide). While the transport of these specific protozoa in subsoil systems has not been studied directly, the transport of an indigenous strain of flagellated protozoa in a sandy aquifer was studied by Harvey *et al.* (1995). A tenfold decrease in the numbers was observed over the first metre of flowpath and numbers of protozoa became undetectable 3.6 metres down gradient from the injection point. This suggests that transport of protozoa through the unsaturated subsoil would be inhibited.

4.5 Irish Experience of Groundwater Contamination by STE

Septic tank systems, together with farmyards, are considered to be the two main point sources of groundwater pollution in rural areas (Daly *et al.,* 1993; Daly, 1993; Dames and Moore, 2000). Groundwater is an important national asset that accounts for about 15% of total water volume supplied by public authorities and about 25% of all water supplies (Environmental Research Unit, 1993). The degree of microbial contamination of groundwater in Ireland is very high with, in many areas, at least 30% of private domestic and farm wells polluted by faecal bacteria and in some highly vulnerable areas more then 50% polluted. It is likely that there are areas in Ireland where more than 70% of private wells contain faecal bacteria at some time during their use (Daly, 2003). Groundwater contamination from septic tank treatment systems is mainly a problem where dwellings are not serviced by public sewerage and where water supplies are obtained from poorly constructed wells in vulnerable aquifers. In 1993 there were over 300,000 septic tank systems in Ireland serving a population of about 1.2 million people and discharging 80 million m^3 of effluent into the ground annually (Daly *et al.*, 1993); this figure increased to approximately 400,000 dwellings in 1999 (DoELG *et al.*, 2000). Of the estimated 200,000 wells and springs in use at present in Ireland those in unserviced areas are therefore often at risk of STE contamination because of their location.

E. coli, an enteric bacteria, when discovered in groundwater is an indicator of faecal contamination but unfortunately its presence is not specific to STE contamination as it is also present in the gut of warm blooded animals. The absence of farmyards in the locality, therefore, could be indicative of contamination from septic tanks. In the 1980s, Daly (1987) argued that septic tanks, rather than farmyards, were the more likely cause of groundwater contamination because:

- they are more numerous than farmyards,
- contaminated wells are more often located closer to septic tanks than to farmyards, and
- the use of soakage pits allows the STE to enter groundwater more readily than farmyard effluent.

Daly (1987) also reported that of samples taken from 146 groundwater sources in one local authority area 84 (58%) contained *E. coli*. Out of 39 high yielding wells and springs in the same county, 29 (74%) were contaminated. In another county 22 of 41 group schemes surveyed contained *E. coli*. Thorn *et al.* (1986) examined the groundwater quality in south Co. Sligo and found that of the 42 sources examined, 28 (67%) were contaminated with faecal coliform and/or faecal streptococci. They concluded that septic tanks were the main source of this contamination. The presence of *Cryptospiridium* in Lough Owel, the water supply for Mullingar, in 2003 caused 26 people to become ill. As the lake is groundwater fed and as the vulnerability of the catchment is mostly 'extreme' and 'high' groundwater was considered to be a potentially significant pathway for *Cryptospiridium* to enter the lake from septic tanks and/or farmyards (Moran, 2003).

A study sponsored by the Department of Environment and Local Government examining the methods available for the reduction of nutrient inputs to Lough Leane from all sources included a section on the possible impacts of STE on water quality in the Lough Leane catchment (DoELG, 2000). It was found that the location of domestic wastewater treatment systems in areas of shallow overburden resulted in contamination of surface streams by STE. However, a survey of part of the catchment area revealed the most representative type of wastewater treatment system to be a one-chamber, blockwork septic tank greater than 20 years old with effluent discharge going to the soakaway area. It could be hoped, therefore, that the situation with respect to STE contamination will improve, both in the Lough Leane catchment and nationwide, with the implementation of the EPA guidelines (2000).

5. SITE SELECTION

5.1 Introduction

There were a number of essential criteria that had to be satisfied during the site selection process in order for the four sites eventually selected to be deemed suitable. T-values of between 1 and 25, 25 and 50, and greater than 50 had to be identified, site location and trial hole inspections had to satisfy EPA (2000) guidelines and there had to be a sufficient number of residents in each dwelling to ensure that at least 20m of percolation trench could be used. It was decided that, due to time constraints, the potential research sites would have to be fully occupied and that dwellings only at the planning stage and works in progress would not be considered. Therefore, as an incentive to interested parties, the project team offered to pay for and install, to EPA (2000) specifications, an on-site treatment system on the suitable sites.

5.2 Site Selection Criteria

5.2.1 Desk Study

The main aims of the desk study were to obtain hydrogeological data relevant to assessing site suitability and to identify potential targets at risk (i.e. water resources) from the proposed installation of an on-site treatment system. While it is possible to obtain data on bedrock type, soil type and subsoil type on a small scale, i.e. on a regional or countywide basis, from relevant publications, the GSI, as part of the Groundwater Protection Scheme (GWPS) framework (Chapter 1), has produced individual Groundwater Protection Schemes for 14 counties/regions (Clare, Kildare, Kilkenny, Laois, Limerick, Meath, Monaghan, North Tipperary, Offaly, Roscommon, South Cork, South Tipperary, Waterford and Wicklow) from which it is possible to obtain detailed information (1:50,000 scale maps) on the soil type, subsoil type, bedrock type, aquifer category and vulnerability class within these counties. This information, used in conjunction with the Groundwater Protection Response Matrix for Single House Systems (Table 5.1), an integral part of the GWPS, was an essential part of the assessment of site suitability for the installation of a treatment system and also of the type of system to be installed.

VULNERABILITY RATING	RESOURCE PROTECTION AREA Aquifer Category					
	Regionally Imp		*Locally Imp*		*Poor Aquifers*	
	Rk	Rf/Rg	Lm/Lg	Ll	Pl	Pu
Extreme (E)	R2[2]	R2[2]	R2[1]	R2[1]	R2[1]	R2[1]
High (H)	R2[1]	R1	R1	R1	R1	R1
Moderate(M)	R1	R1	R1	R1	R1	R1
Low (L)	R1	R1	R1	R1	R1	R1

R1 Acceptable subject to normal good practice (i.e. system selection, construction, operation and maintenance in accordance with EPA (2000).

R2[1] Acceptable subject to normal good practice. Where domestic water supplies are located nearby, particular attention should be given to the depth of subsoil over bedrock such that the minimum depths required (EPA, 2000) are met and that the likelihood of microbial pollution is minimised.

R2[2] Acceptable subject to normal good practice and the following additional condition:

- There is a minimum thickness of 2m unsaturated soil/subsoil beneath the invert of the percolation trench of a conventional septic tank system;

OR

- A treatment system other than a conventional septic tank system as described in EPA (2000) is installed, with a minimum of 0.6m unsaturated soil/subsoil with P/T values from 1 to 50 (in addition to the polishing filter which should be a minimum depth of 0.6m), beneath the invert of the polishing filter (i.e. 1.2m in total for a soil polishing filter).

Table 5.1 Response Matrix for on-site treatment systems (adapted from DoELG *et al.*, 1999).

While in the methodology outlined in *Wastewater Treatment Manual: Treatment Systems for Single Houses* (EPA 2000) a desk study should precede any on-site assessment, this was not a practical option in the identification of suitable sites for the project; whereby the on-site evaluation procedure was used to eliminate unsuitable sites and a subsequent desk study confirmed potential site suitability.

5.2.2 On-Site Assessment

There are three stages to the on-site assessment process: visual assessment, trial hole inspection and P/T-tests. The visual inspection was a very valuable tool in the selection process and enabled a prompt decision to be made on the suitability of a site for further investigation or its elimination from the selection process prior to any disruptive excavation. Tables 5.2 and 5.3 outline the main factors considered during the visual inspection.

FACTOR	SIGNIFICANCE
Water level in ditches and wells	Indicates depth of unsaturated subsoil
Shape, slope and form of site	May indicate whether water will collect at a site or flow away from the site
Presence of watercourses	May indicate low permeability or a high watertable
Presence and types of rock outcrops	Insufficient depth of subsoil to treat wastewater allowing it to enter the groundwater too fast
Proximity to adjacent percolation areas and/or houses	May indicate too high a loading rate for the locality and/or potential nuisance problems
Land use and type of grassland surface (if applicable)	Indicator of rate of percolation or groundwater levels
Vegetation type	Indicator of the rate of percolation or groundwater levels
Proximity to wells on-site and off-site, water supply sources, groundwater, streams, ditches, lakes, surface water ponding, beaches, shellfish areas and wetlands	Indicates targets at risk

Table 5.2 Factors to be considered during a visual assessment (EPA 2000)

Type of System	Watercourse/ Stream	Wells/ Springs	Lake	Any Dwelling	Site Boundary	Road	Slope Breaks/ Cuts
Septic tank; Prefabricated intermittent filters; mechanical aeration systems	10	10	50	7	3	4	4
In situ intermittent filters; percolation area; polishing filters	10	30	50	10	3	4	4

Table 5.3 Minimum separation distances in metres (EPA, 2000).

Trial Hole

On completion of a successful visual assessment a trial hole was excavated to ensure that there was a sufficient depth of unsaturated subsoil below the invert of the percolation trench along its full length (1.2m for subsoil receiving septic tank effluent and 0.6m for subsoil receiving secondary effluent). It was essential that the trial hole was not excavated within the boundaries of the proposed percolation area as any subsequent consolidation of the backfilled soil could lead to the buckling of the perforated pipe in the percolation trench, thus inhibiting effluent dispersion along the base of the entire trench. The excavation and replacement of the subsoil can also create preferential flowpaths within the subsoil matrix which increases the risk to groundwater contamination from percolating wastewater effluent (Section 4.2.1).

The soil and subsoil characteristics, outlined in Table 5.4, were also examined in the trial hole as part of this integrated approach of on-site assessment. They provide a better understanding and description of the subsoil matrix and an insight into future behaviour of the soil on receipt of wastewater effluent.

CHARACTERISTIC	IMPORTANCE
Soil texture	Affects physical and chemical processes within the soil
Structure	Influences pore space, aeration and flow conditions
Preferential flowpaths	Influence the percolation rate of effluent, level of treatment and subsequently the risk to groundwater
Soil density	Influences percolation rate
Colour	Indicative of state of aeration of soil
Layering	Effects percolation rate

Table 5.4 Subsoil characteristics considered during trial hole inspection

T-Test

The T-test is used to ascertain the suitability of a subsoil to receive septic tank effluent. It calculates an average of time for water to drop 100mm in two pre-soaked, 300mm square holes, 400mm deep below the invert level of the percolation pipe, therefore giving an indication of the hydraulic assimilation capacity of the subsoil surrounding the base of the percolation trench under saturated conditions. As with the trial hole, it was essential that the T-test holes were excavated outside the boundaries of the proposed percolation area. While the proposed construction of a percolation area for the treatment of effluent from secondary treatment systems allows for a similar test to be carried out from ground level (known as a P-test), it was decided that the percolation trenches on both sites should be constructed to the same specifications, for continuity, and therefore T-tests would be required at both sites.

T-test procedure

Two T-test holes were excavated adjacent to the proposed percolation area. The bottom and sides of the holes were rubbed with a wire brush to remove any smearing or compaction caused during their excavation. Both holes were filled with clear water at 10:00 and again at 17:00. The following day the silt was cleaned out of both holes and a bar with rubber rings was inserted into each hole. The rubber rings were then placed at 100mm intervals up to 400mm from the base of the hole. The holes were refilled with clear water up to the 400mm mark and the time noted. The water was allowed to drop to 300mm and the subsequent time required for the water to drop to 200mm was recorded. The hole was then refilled to the 300mm mark and the time required for it to drop to 200mm recorded again. This procedure was then carried out a second time. The average time of the three recordings, i.e. the time required for the water to drop from 300mm to 200mm, was divided by four to give the time required for a fall of 25mm or the *t-value* for each hole. The average of the t-values is then calculated to give the T-value. A proposed percolation area with a T-value less than 1 minute/25mm, or greater than 50minutes/25mm, is deemed to have failed the test. However, one of the aims of this project was to examine the assimilation capacity of a subsoil with a T-value greater than 50minutes/25mm.

On completion of a satisfactory desk study and site assessment, a site can be considered to be suitable for the installation and construction of an on-site treatment system. However it should be appreciated that both the results from the trial hole inspection and the T-test are location-specific: the subsoil is not an isotropic homogeneous medium and therefore results from both methods of suitability assessment will vary across the site. There can also be a large difference in the two t-values. It is possible for one, or both, t-value(s) to be outside the 1-50 range and yet for the T-value to be acceptable (for example t_1=0.5 and t_2=52). This occurrence should necessitate a more comprehensive site assessment. It is also worth considering the size of the range into which a successful T-value must fall (Box 5.1).

Box 5.1

For example a T-test that results in T-value of 1 could be completed in 0.5 hours while a T-test resulting in a T-value of 50 could take18 hours, yet both sites are deemed suitable for the same treatment systems. Saturated conditions in a trench of high T-value subsoil would promote anaerobic conditions which impede the aerobic chemical and biological processes (outlined in Chapter 4) that are essential in the attenuation of the wastewater effluent. Similarly, a highly permeable subsoil would mean a short residence time thereby reducing effluent attenuation.

In extreme situations a high T-value could forewarn of surface ponding due to the inability of the subsoil to assimilate the wastewater. While a design loading rate attempts to account for this impedance (EPA 2000) it fails to distinguish between subsoils of different permeability receiving septic tank effluent. A figure of 20l/m^2.d is recommended for all subsoils with an acceptable T-value receiving septic tank effluent while loading rates of 25l/m^2.d and 50l/m^2.d are recommended for subsoils of T-values between 21and 50, and 1 and 20 respectively, receiving secondary effluent. While the increased loading rate acceptable for secondary effluent is justified by the decrease in organic and nutrient loading brought about by the additional treatment step the size of this increase and the definition of two broad ranges is questionable.

5.2.3 Occupancy

The EPA manual calculates the typical daily hydraulic loading to an on-site system for single houses as 180 litres per person. While no reference for this figure is cited in the manual it appears to be base on research in the US. Research by WS Atkins (2000) calculated the average per capita water consumption for Ireland in 1997 as between 130l/d and 139l/d with this figure rising to between 146 and 158l/d by 2018. A wastewater loading rate of 20l/m^2.d for septic tank effluent and 25 l/m^2 for secondary effluent on subsoils with a T-value between 21 and 50 are recommended (EPA, 2000). With a maximum permissible trench length of 20m and the standard distribution box containing 4 outlets, it was therefore decided that the desired occupancy of the test sites should be 4 people on the sites receiving septic tank effluent and 5 people on the site with a T-value between 21 and 50 receiving secondary effluent. With respect to the site with a T-value greater than 50 receiving secondary effluent it was decided that a minimum occupancy 4 was required.

5.3 Site Selection

The initial site identification process from Sites 1 and 2 took the form of a networking exercise and involved the canvassing of personal contacts such as family, friends, colleagues, students, local builders and members of the GAA in Wicklow. Professional contacts such as the GSI, Teagasc, EPA and various independent consultants were also approached. This method was productive, resulting in 23 site investigations – but only yielded one site which fulfilled the specified criteria. Contact was then made with the relevant department of numerous local authorities in the Leinster region (Carlow, Fingal, Meath, Louth, Kildare, Laois, Wexford and Wicklow). While contact was also initiated with Limerick County Council the main focus of attention was concentrated on the other local authority areas, especially those bordering Dublin as proximity to laboratory facilities in Trinity College was of utmost importance for sample analysis. It was also desirable from the point of technical support that, if possible, the research sites were within commuting distance of college.

Contact was also made with manufacturers of, and agents for, various secondary treatment systems (Evirocare, Bord na Móna, Bio-clear, Biocycle, Biocrete and Molloy Engineering). It was envisaged that, as the installation process of such systems should involve a comprehensive site assessment, such companies would have a database of potential sites. While this was true, most of the information related to green-field sites or work in progress, save for two sites, and was therefore of little relevance to the project.

The most successful site identification strategy adopted was a media campaign which involved newspaper advertisements, radio interviews and advertisements, a poster campaign and presentations. This yielded 37 potential sites for investigation from over 100 responses. Over 70 sites were rejected due to distance from Dublin and it is worth noting that information gleaned from conversations with these house owners revealed that 15 of these sites experience a high watertable and another 5 experience drainage problems. Figure 5.1 gives a county by county breakdown of the sites investigated.

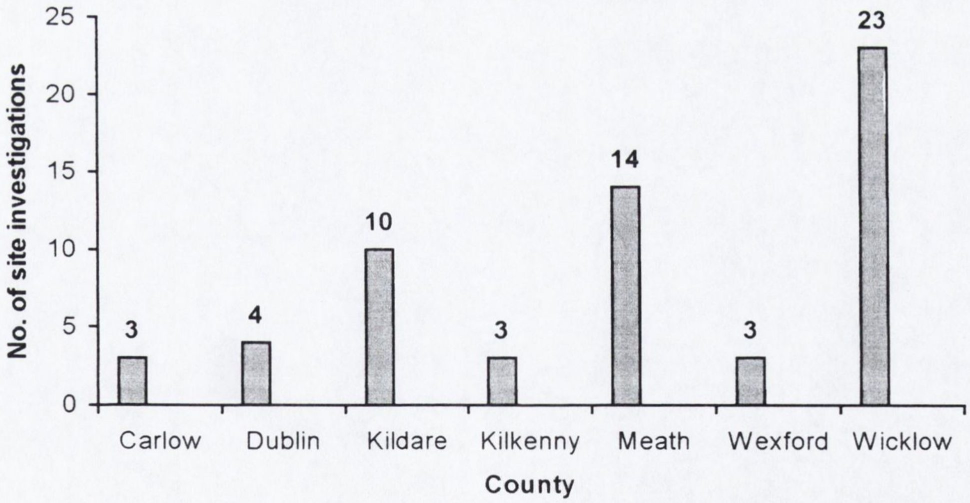


Figure 5.1 Location of site investigations.

Of the total of 60 sites with the desired occupancy rate investigated over a 7 month period only 2 (3.33%) satisfied the site selection criteria, ergo the EPA recommendations. Of a further 14 sites investigated during the site identification process for Sites 3 and 4 only 2 (14.29%) satisfied site selection criteria. When these data are combined it shows that while all sites had some form of on-site treatment system only 5.41% were deemed suitable for such systems. It is clear from Table 5.5, which outlines the reasons for site unsuitability, that the most common reason for site rejection was the presence of a high watertable.

REASON FOR UNSUITABILITY		QUANTITY	PERCENTAGE
T-value	Too high	12	16.4
	Too low	9	12.3
Watertable		36	49.3
Shallow bedrock		4	5.5
Confined site		9	12.3

Table 5.5 Reasons for site unsuitability.

While proximity to Dublin determined the catchment area for potential research sites the location of the selected sites within this area was based solely on completion of the dwelling and suitable occupancy levels. Should the results of this assessment of a random sample of sites be indicative of the regional unsuitability for on-site treatment of domestic wastewater, the pressure being exerted on water resources in these regions, especially groundwater given the high percentage of sites deemed unsuitable due to the presence of a high watertable, could be a cause for concern.

5.4 Test Sites

As a result of the successful networking and advertising campaigns and the subsequent site investigations four suitable sites were identified two in County Kildare, one in Rochestown (28890E, 20860N) and the other in the Curragh (27890E, 21290N), and two in County Wicklow, one in Killaveny (30680E, 17390N) and one in Three Wells (31430E, 18160N). The location of these sites is highlighted in Figures 5.2 to 5.5. The Rochestown site was a recently renovated cottage which had no form of on-site treatment system installed. Both surface run-off and domestic effluent were discharged into an open pit of 1.5m depth. As a planning condition relating to the construction work on the Curragh site, Kildare County Council specified that the present septic tank and percolation area had to be

Figure 5.2 Location of Site 1 at Rochestown.

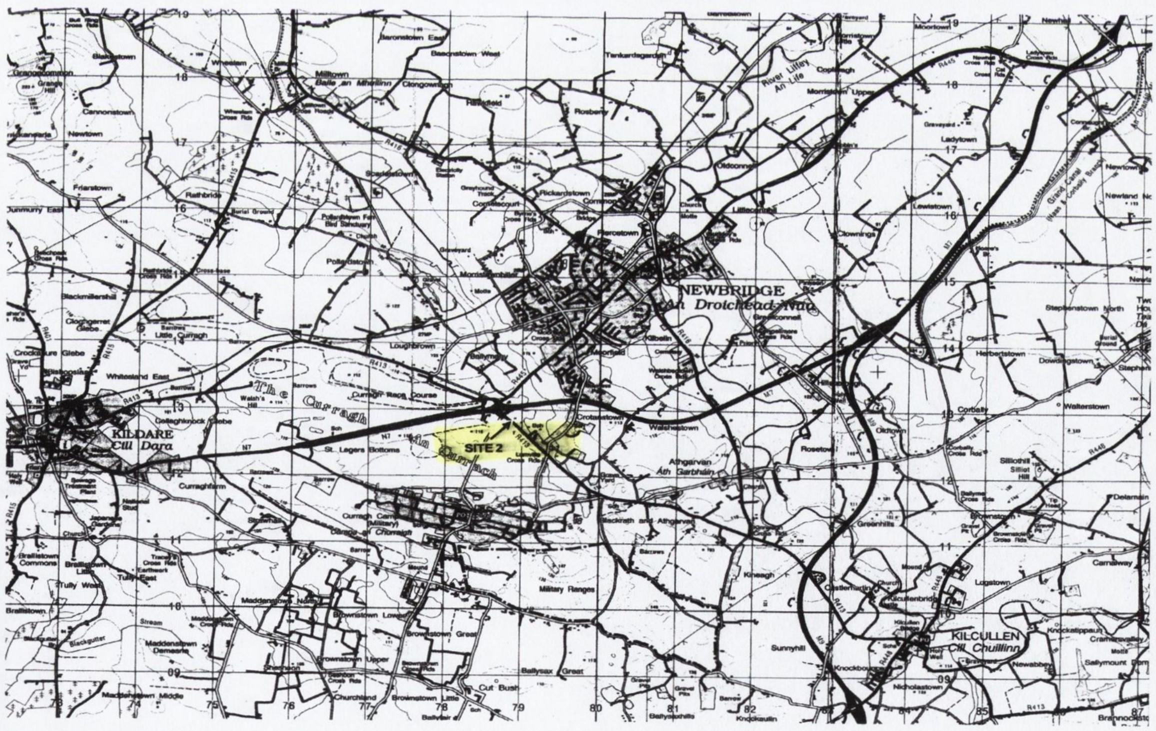


Figure 5.3 Location of Site 2 at the Curragh.

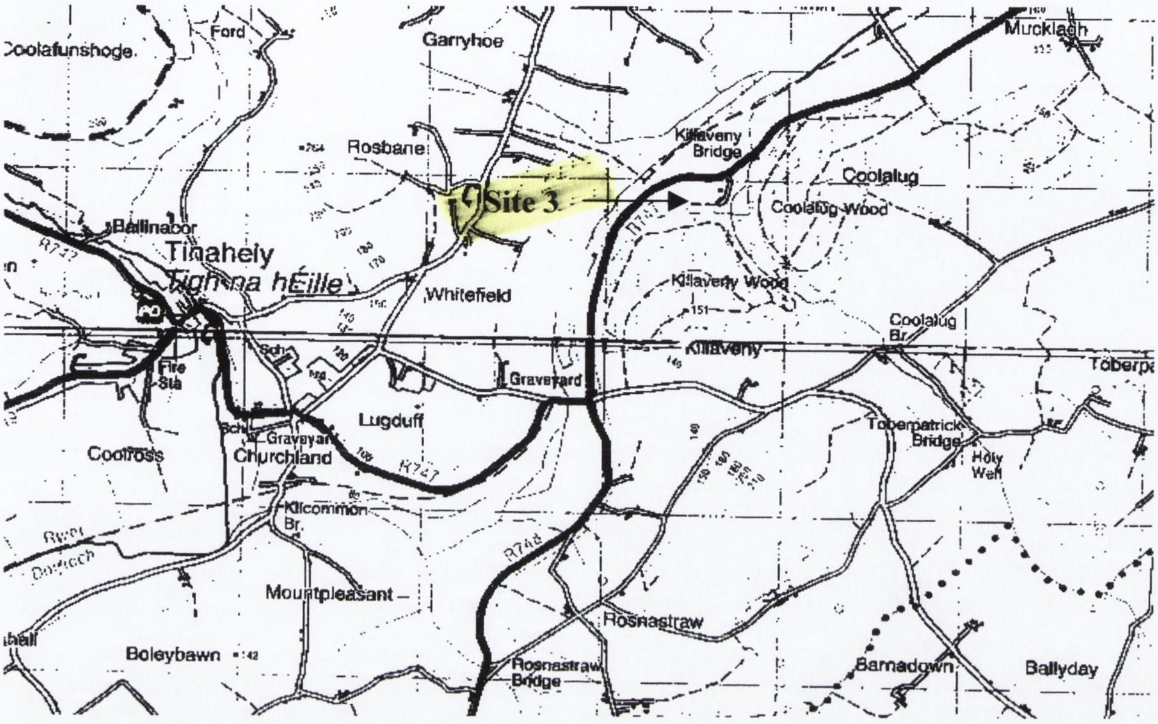


Figure5.4 Location of Site 3 at Killaveny.

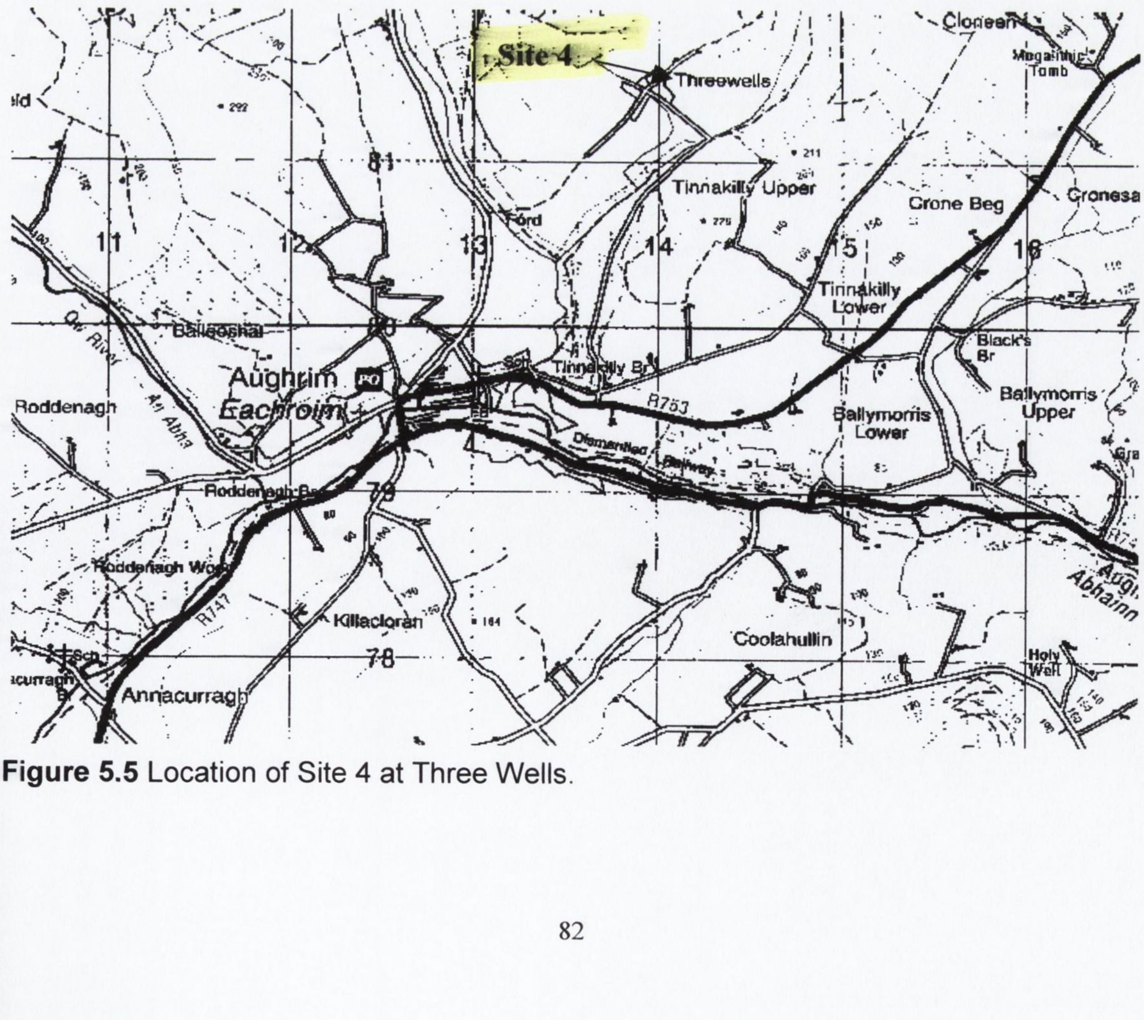


Figure 5.5 Location of Site 4 at Three Wells.

upgraded. Similarly a planning condition in relation to an extension on the Killaveny site required the upgrading of the existing domestic wastewater treatment system. The site at Three Wells was a recently constructed bungalow on which the septic tank treatment system had yet to be installed. As Figures 5.2 to 5.5 show all sites were located in undeveloped areas, so it was possible to construct four new on-site treatment systems in undisturbed subsoil.

5.4.1 Site 1: Rochestown

Rochestown lies within a region of grey-brown podzolic soil described as the Elton Series, a series that occurs most extensively in the northern and eastern portions of County Kildare (Conroy *et al.*, 1970). These soils normally occur at elevations less than 150m, on undulating relief with slopes of 2 to 6°. They are derived from dominantly limestone drift with a small admixture of shale and sandstone. The soils of this series are deep, well-drained, of loam texture and high base status.

The excavation of the trial hole to ensure sufficient depth of unsaturated subsoil (Figure 5.6) also facilitates the examination of the subsoil profile and characteristics

Figure 5.6 Trial hole on Site 1.

and the completion of the required site characterisation form (Appendix A). While the installation of a secondary treatment systems only requires the presence of 0.6m of unsaturated subsoil below the invert of the percolation trench, it was desired, in order to allow direct comparison between sites with septic tank treatment systems and those with secondary treatment systems, that 1.2m of unsaturated subsoil below the invert of the percolation trench was present.

Soil/subsoil texture classification was carried out in accordance with BS5930 (1981) with the aid of a flowchart produced by the Groundwater Section of the GSI (Appendix A). It was found that the soil profile consisted of three distinctive layers as outlined in Table 5.6. While no particle size distribution analysis was carried out on the layer between 0.3m and 0.9m, it showed the characteristics of a subsoil with a higher clay content than the layer below it. This is consistent with the regional description of the soil as a grey-brown podsol.

	Soil / subsoil Texture & Classification	Soil Structure	Density	Colour	Preferential Flowpaths
0.1M	A Horizon	Crumb	Medium	Dark brown	Roots
0.2M					
0.3M					
0.4M	sandy Clay (w/silt)	Structureless – massive	Medium	Reddish brown	Some roots and macropores
0.5M					
0.6M					
0.7M					
0.8M					
0.9M					
1.0M	sandy Silt (w/clay)	Structureless – massive	Medium	Brown	None evident
1.1M					
1.2M					
1.3M					
1.4M					
1.5M					
1.6M					
1.7M					
1.8M					
1.9M					
2.0M					
2.1M					
2.2M	Base of hole				

Table 5.6 Characterisation of soil profile at Site 1.

In general, the selection of sampling points within a trial hole for BS5930 classification is dependent on the visual assessment of the exposed material, as it is not feasible to examine the subsoil texture at small intervals over the entire hole depth. Therefore, difference in subsoil colour is often used to distinguish between different layers, i.e. layers that appear on visual inspection to be of uniform colour and texture. The flaw in this method is highlighted by comparing the particle size distribution curves of samples taken from a layer that has been described as homogeneous in Table 5.6. Soil samples taken at 1.0m, 1.5m and 2.0m below groundlevel were analysed for particle size distribution (Appendix A). These were the depths to which it was proposed to install the suction lysimeters and tensiometers. Table 5.7 summarises the graphs produced in Appendix A.

Depth of sample below groundlevel	% Gravel	% Sand	% Silt	% Clay
1.0m	10.6	50.3	32.8	6.3
1.5m	27.5	61.1	11.4	0
2.0m	10.1	59.3	26.8	3.8

Table 5.7 Particle size analysis of soil samples from Site 1.

While Table 5.7 shows that the samples taken at 1.0m and 2.0m are similar in particle size distribution the sample taken at 1.5m has a substantially lower fines content, which is indicative of a layer of greater permeability. Such a layer, if it extends laterally, could be beneficial in the treatment of percolating wastewater should it act as a distribution medium enabling the treatment process to be effected over a greater area. As the T-test (Table 5.8) was carried out in the B horizon it is not representative of the more extensive C horizon below, which in this case would appear to be of higher permeability. This has led to the suggestion that the T-test hole should be excavated to a depth of 400mm below the invert of the percolation trench. However, this does not take into account the infiltration capacity of the side walls of the percolation trench. As the test stands, the infiltration capacity of an area 150mm above and below the percolation trench is examined thus giving an indication of the hydraulic behaviour of the subsoil along the trench sides and below its base under saturated conditions. However, it is questionable as to whether or not a head of 300mm would build up in the trench. A refinement that might be suggested, therefore, is that the T-test hole is excavated to the depth of the projected invert of the trench and that a smaller volume of water might be more representative of future conditions. The driving force provided by a smaller head could also be seen to better represent the impedance that would be expected by the formation of the biomat along the base and walls of the trench.

	Test Hole No. 1			**Test Hole No. 2**		
Fill No.	**Start Time** (at 300mm)	Finish Time (at 200mm)	Δt (minutes)	**Start Time** (at 300mm)	Finish Time (at 200mm)	Δt (minutes)
1	11:45	13:05	80	12:13	14:25	132
2	13:08	14:33	85	14:26	16:53	147
3	14:33	16:10	97	16:53	19:22	149
		AverageΔt (minutes)	87.3		AverageΔt (minutes)	142.7
		AverageΔt/4 = t_1(minutes)	21.8		AverageΔt/4 = t_2(minutes)	35.7
T-value = $(t_1+t_2)/2$ = 28.8 (minutes/25mm)						

Table 5.8 T-test results from Site 1.

Research carried out in Galway (Mulqueen and Rogers, 2001) has produced an adapted T-test which derives an equivalent T-value from time factors obtained through laboratory experiments. The main difference between the two tests is that while with the standard T-test the time is recorded at 100mm intervals, it is recorded at 50mm intervals in the adapted T-test. With the adapted T-test the time is initially recorded when the test hole is filled to 400mm and the test is completed when the water level in the hole reaches 100mm above the base. Unlike the standard T-test the adapted T-test only requires the test hole to be filled once. As the data for the adapted T-test were being obtained during the standard T-test it was not possible to record time data below 200mm. While the adapted T-test carried out on Site 1 was therefore incomplete, it is worth noting the similarity between the T-value obtained by this method and that obtained using the standard T-test (Table 5.9). It can be seen that the adapted T-test, for both Sites 1 and 2, under-predicts the standard T-test but if it had been continued to 100mm it is likely to returned a higher average value, due to the lower head, thus resulting in a closer approximation.

Fall of water in hole (mm)	Time Factor (T_f)	Time (min) of fall (T_m)		Field-Saturated hydraulic conductivity $K_{fs} = (T_f/T_m)$		Equivalent percolation T-value in minutes/25mm $T = 4.45/K_{fs}$	
		Hole1	Hole 2	Hole1	Hole 2	Hole1	Hole 2
400-350	5.3	25	39	0.21	0.14	21.2	31.8
350-300	6.9	30	48	0.23	0.14	19.3	31.8
300-250	8.1	32	51	0.25	0.16	17.8	27.8
250-200	9.7	48	81	0.20	0.12	22.3	37.8
200-150	11.9	N/A	N/A	N/A	N/A	N/A	N/A
150-100	14.1	N/A	N/A	N/A	N/A	N/A	N/A
					Average T	**26.2**	

Table 5.9 Adapted T-test results from Site 1 (see Mulqueen and Rogers, 2001, for explanation of terms).

SITE OCCUPANCY

For the duration of the project there were five people resident at Site 1, a husband and wife, their daughter and two sons. All the children were students at primary and secondary schools, generally leaving the house at between 08:00 and 08:30 and returning between 15:30 and 16:30, Monday to Friday. The father ran a business from home and the mother generally left home between 09:30 and 10:00 for work, returning between 16:30 and 17:00. It must be considered that in modern society it is common for both spouses to work outside the home, reducing daily loading rates on the wastewater treatment system.

5.4.2 Site 2: The Curragh

Site 2 lies within a region of soil described as the Athy Complex (Conroy *et al.*, 1970). Soils of this type, which generally occur between elevations of 46m and 275m, are found scattered throughout the county but occur predominantly in the southern part. The topography of the regions where soils of this complex are found varies between flattish and undulating, with many areas described as hummocky with sharp slope changes ranging from 0° to 20°. The parent material of these soils consists of calcareous, fluvioglacial coarse gravels and sands of Weichsel Age composed mainly of limestone with a small proportion of sandstone, schist, shale and conglomerate (Conroy, *et al.*, 1970).

As was the case with Site 1, a trial hole (Figure 5.7) was used to examine the subsoil profile and characteristics, and the results obtained used to complete the required site characterisation form (Appendix A). On inspection of the trial hole four distinct layers were observed and classified (Table 5.10). Grain size distribution curves were also produced for samples taken at depths of 1.0m, 1.5m and 2.0m below ground level (Appendix A), and Table 5.11 again highlights the difficulty in identifying textural differences in an apparently homogenous layer. As the soil at this site has a higher clay content than the soil at Site 1, it was expected that Site 2 would show a lower percolation rate. However, this was not borne out by the results of the T-test (Table 5.12).

Figure 5.7 Trial hole on Site 2.

Under high potential levels the hydraulic conductivity of sands is higher than clays while the opposite is true at low potential levels. It is possible, therefore, that on Site 2 the pre-soaking step of the T-test failed to create saturated conditions in the subsoil surrounding the test holes and that unsaturated conditions prevailed. It must also be considered that the presence of preferential flowpaths in the form of macropores, cracks and voids around cobbles were observed across the soil profile and that this could explain the higher than expected percolation rate (Table 5.10)

	Soil / subsoil Texture & Classification	Soil Structure	Density	Colour	Preferential Flowpaths
0.1M 0.2M 0.3M	A Horizon	Crumb	Medium	Dark brown	Roots
0.4M 0.5M 0.6M 0.7M	Silt/Clay	Structureless-massive	Low	Brown	Some root ends and macropores present
0.8M 0.9M 1.0M 1.1M	sandy Clay (w/silt) interspersed with rounded cobbles	Structureless-massive	Medium	Reddish Brown	Macropores, cracks & voids around some cobbles
1.2M 1.3M 1.4M 1.5M 1.6M 1.7M 1.8M 1.9M 2.0M 2.1M 2.2M	sandy Clay (w/silt) interspersed with rounded cobbles	Structureless-massive	Medium	Brown	Macropores, cracks & voids around some cobble
2.3M	Base of hole				

Table 5.10 Characterisation of soil profile at Site 2.

Depth of sample below groundlevel	% Gravel	% Sand	% Silt	% Clay
1.0m	3.3	23.3	50.1	23.3
1.5m	8.8	18.9	51.1	21.2
2.0m	24.9	23.8	36.8	14.4

Table 5.11 Particle size analysis of soil samples from Site 2.

	Test Hole No. 1			Test Hole No. 2		
Fill No.	**Start Time** (at 300mm)	Finish Time (at 200mm)	Δt (minutes)	**Start Time** (at 300mm)	Finish Time (at 200mm)	Δt (minutes)
1	12:04	13:11	67	11:38	12:15	37
2	13:11	14:30	79	12:15	13:00	45
3	14:30	15:46	76	13:00	13:47	47
		AverageΔt (minutes)	74		AverageΔt (minutes)	**43**
		AverageΔt/4 = t_1(minutes)	18.5		AverageΔt/4 = t_2(minutes)	10.8
T-value = $(t_1+t_2)/2$ = 14.6 (minutes/25mm)						

Table 5.12 T-test results from Site 2.

During the T-test the time taken per 50mm fall in water level between the top of the test hole and 200mm above its base was recorded to enable a T-value to be calculated by the adapted T-test method (Table 5.13).

Fall of water in hole (mm)	**Time Factor (T_f)**	**Time (min) of fall (T_m)**		**Field-Saturated hydraulic conductivity $K_{fs} = (T_f/T_m)$**		**Equivalent percolation T-value in minutes/25mm $T = 4.45/K_{fs}$**	
		Hole1	Hole 2	Hole1	Hole 2	Hole1	Hole 2
400-350	5.3	18	5	0.29	1.06	15.3	4.2
350-300	6.9	27	12	0.25	0.58	17.8	7.7
300-250	8.1	32	12	0.25	0.58	17.8	7.7
250-200	9.7	35	25	0.28	0.39	15.9	11.4
					Average T	**12.2**	

Table 5.13 Adapted T-test results from Site 2 (see Mulqueen and Rogers, 2001, for explanation of terms).

While this value gives a good approximation of the standard T-test value, as was the case at Site 1, it is again an underestimation. However, as has been proved under laboratory conditions, a completed adapted T-test may give a better approximation of the standard T-test value and thus greatly reduce the time required in assessing the percolation rate of the subsoil.

SITE OCCUPANCY

On initiation of the project there were four adults resident at Site 2; however, this number fluctuated throughout the year. In November one female resident left although she continued to use the laundry facilities in the house. Over the rest of the project duration the number and profile of residents continued to fluctuate with two females, an elderly lady and her carer, leaving in February to be replaced by two males. A housekeeper also called on Mondays to Fridays throughout the year.

There is a livery stable attached to the house in which three people were employed. The employees started work at approximately 07:00 and left for lunch at between 12:30 and 13:00.They sometimes returned for a few hours in the evening. While the employees did not partake in the daily routine of the house they did have access to an external toilet that was connected to the wastewater system of the main house.

5.4.3 Site 3: Killaveny

Killaveny lies within a region of lower Palaeozoic sandstone and shale till known as the Ribband group. This group was deposited after the Bray Group, a formation dominated by greywacke sandstones interbedded with slates, shales and distinctive massive quartzite in the north of the county, and consists of mudstones, siltstones, quartzites and volcanic rocks. These rocks outcrop on both sides of the Leinster Granite and are divided into seven formations one of which is the Maulin formation within which Three Wells is located. The Maulin formation consists predominantly of fine grained sedimentary rocks which have been metamorphosed by the intrusion of the granite (GSI, 2003).

The opening of a trial hole (Figure 5.8) and the subsequent classification of the exposed material (Table 5.14) revealed the subsoil below the percolation area on Site 3 to be a clayey Sand. As can be seen from Table 5.15, which summarises the particle size distribution graphs in Appendix A, there is a reduction in the silt content

Figure 5.8 Trial hole on Site 3

	Soil / subsoil Texture & Classification	Soil Structure	Density	Colour	Preferential Flowpaths
0.1M	A horizon	Crumb	Medium	Dark brown	Roots, some evidence of macropores
0.2M					
0.3M					
0.4M	clayey Sand with some rounded cobbles	Structureless -single grain	Dense	Light brown	Some root ends
0.5M					
0.6M	gravelly clayey Sand interspersed with gravel and rounded cobbles	Structureless - single grain	Dense	Dark brown	None obvious although pockets of cobbles create macropores
0.7M					
0.8M					
0.9M					
1.0M					
1.1M					
1.2M					
1.3M					
1.4M					
1.5M					
1.6M					
1.7M					
1.8M					
1.9M					
2.0M					
2.1M					
2.2M					

Table 5.14 Characterisation of soil profile on Site 3.

with depth and an increase in the gravel content. While the results of the particle size analysis alone would generally suggest a high permeability subsoil the high density of the matrix resulted in a T-value greater than 50 (Table 5.16).

Depth of sample below groundlevel	% Gravel	% Sand	% Silt/Clay
1.0m	13.9	53.7	32.4
1.5m	23.0	55.3	21.7
2.0m	35.0	45.9	19.1

Table 5.15 Particle size analysis of soil samples from Site 3.

As can be seen from Table 5.16 one of the t-tests carried out on Site 3 was not completed and the calculated T-value therefore was an average of five rather than six time intervals. This was due to time taken for the 100mm drop in water level to be achieved in test hole number two. It should be noted, however, that experience has shown that there is generally very little difference between the second and third time intervals in a t-test and that the T-value calculated for Site 4 is similar to the T-value that would have been calculated should the T-test have been completed.

	Test Hole No. 1			Test Hole No. 2		
Fill No.	Start Time (at 300mm)	Finish Time (at 200mm)	Δt (minutes)	Start Time (at 300mm)	Finish Time (at 200mm)	Δt (minutes)
1	12:12	15:07	175	13:20	17:15	235
2	15:07	18:07	180	17:15	22:15	240
3	18:07	21:14	187	N/A	N/A	N/A
		Average Δt (minutes)	180.6		Average Δt (minutes)	237.5
		Average Δt/4 = t_1(minutes)	45.2		Average Δt/4 = t_2(minutes)	59.4
T-value = $(t_1 + t_2)/2$ = 52.3 (minutes/25mm)						

Table 5.16 Results of standard T-test from Site 3.

SITE OCCUPANCY

There were four people resident at Site 3, a mother, father and their two children. The two children were in school, generally leaving the house at 08:30 to return at 16:00. The mother was a housewife while the father worked locally and returned home for lunch most days. Another daughter was in college and returned home some weekends.

5.4.4 Site 4: Three Wells

Located approximately 10.5km north east of Site 3 in the same valley, the subsoil at Site 4 was also described as a till with lower palaeozoic schists, sandstones, greywackes and shales dominant (GSI, 2003). As with Site 3 it is part of the Maulin formation of the Ribband group. Site 4 was an elevated site located at approximately 205m OD while Site 3 was located near the floor of the valley at approximately 90m OD.

A trial hole (Figure 5.9) was again used to examine the subsoil profile and characteristics and facilitate the completion of the site characterisation form (Appendix A). In Figure 5.9 the 2m depth mark is just above the top of the septic tank. Table 5.17 shows that the subsoil exposed by the trial hole on Site 4 was

Figure 5.9 Trial Hole on Site 4.

	Soil / subsoil Texture & Classification	Soil Structure	Density	Colour	Preferential Flowpaths
0.1M	A Horizon	Crumb	Medium	Dark brown	Roots, some evidence of macropores
0.2M					
0.3M					
0.4M	sandy Silt (w/clay)	Structureless -single grain	Medium	Reddish brown	Some root ends
0.5M					
0.6M	Very gravelly clayey Sand interspersed striated cobbles	Structureless – single grain	Medium	Dark brown	Some macropores evident in pockets of gravel and around cobbles
0.7M					
0.8M					
0.9M					
1.0M					
1.1M					
1.2M					
1.3M					
1.4M					
1.5M					
1.6M					
1.7M					
1.8M					
1.9M					
2.0M					
2.1M					
2.2M					
2.3M	Base of Hole				

Table 5.17 Characterisation of soil profile on Site 4.

divided into three distinctive layers. Grain size distribution curves for samples taken at depths of 1.0m, 1.5m and 2.0m below ground level highlighted the uniformity of the subsoil matrix below the percolation trenches (Table 5.18). While no sedimentation tests were carried out to determine the relative percentages of silt and clay the results of the BS5930 analysis would suggest that silt formed the greater percentage of the particles less than 0.063mm. Only a standard T-test was carried out on Site 4 and the results are shown in Table 5.19.

Depth of sample below groundlevel	% Gravel	% Sand	% Silt/Clay
1.0m	40.8	26.4	32.8
1.5m	44.7	23.2	32.1
2.0m	35.1	26.5	38.4

Table 5.18 Particle size analysis of soil samples from Site 4.

	Test Hole No. 1			Test Hole No. 2		
Fill No.	Start Time (at 300mm)	Finish Time (at 200mm)	Δt (minutes)	Start Time (at 300mm)	Finish Time (at 200mm)	Δt (minutes)
1	11:11	13:57	166	11:07	12:23	76
2	13:58	16:50	172	12:24	13:58	92
3	16:50	19:45	175	13:58	15:40	102
Average Δt (minutes)			171	Average Δt (minutes)		90.3
Average $\Delta t/4 = t_1$(minutes)			42.8	Average $\Delta t/4 = t_2$(minutes)		22.6
T-value = $(t_1 + t_2)/2$ = 32.7 (minutes/25mm)						

Table 5.19 Results of standard T-test from Site 4.

SITE OCCUPANCY

There were four people resident at Site 4, a mother and her three children. Two of the children were in school, generally leaving the house at 08:30 to return at 16:00, while the youngest had yet to start school. He was cared for at home by his mother.

6. SITE CONSTRUCTION

6.1 Introduction

To comply with the project specifications a septic tank was installed on two sites, with respective T-values in the range 1 to 25 and 25 to 50. A secondary treatment system, preceded by a septic tank, was installed on the other two sites. The effluent from all systems entered percolation trenches built to the EPA specification (EPA 2000) via similar distribution boxes. All of the septic tanks contained two chambers although the tank installed at Sites 1, 3 and 4 were of greater volume than that at the Curragh, 4000 litres in comparison with 2275 litres. As the capacity of the septic tank on Site 2 is less than the recommended EPA design capacity of 2720 litres there was a concern that an increase in suspended solids content due to a shorter retention time (as determined by tank capacity divided by inflow rate) would cause a reduction in the desired effluent quality. However, as the project progressed this fear was allayed as, even at the greatest flow rates recorded, a retention time of greater than the recommended 24 hours occurred (Section 9.2). Diversion works were necessary at Site 1, 2 and 3 to separate surface run-off and domestic wastewater, and prevent the former entering the septic tank.

6.1.1 Puraflo® Secondary Treatment System

The secondary treatment system installed on Sites 1 and 3 was a Puraflo® system produced by Bord na Móna. Puraflo® is a peat based biofiltration system for the treatment of septic tank effluent. Septic tank effluent enters a sump from where it is pumped to a fibrous peat media which is contained in moulded polyethylene modules (Figure 6.1). This pressurised dosing system is activated by a float switch on the pump. A distribution manifold is located within the peat media, approximately 150mm below the surface, to ensure even distribution of the septic tank effluent within the media and to avoid any odour that might occur due to surface ponding. Unlike many other systems available on the Irish market no mechanical aeration of the treatment media is required as it is naturally aerated through a series of holes at the top of each module thereby reducing the energy demand (Figure 6.2). The treatment of the septic tank effluent within the fixed film media is achieved by a

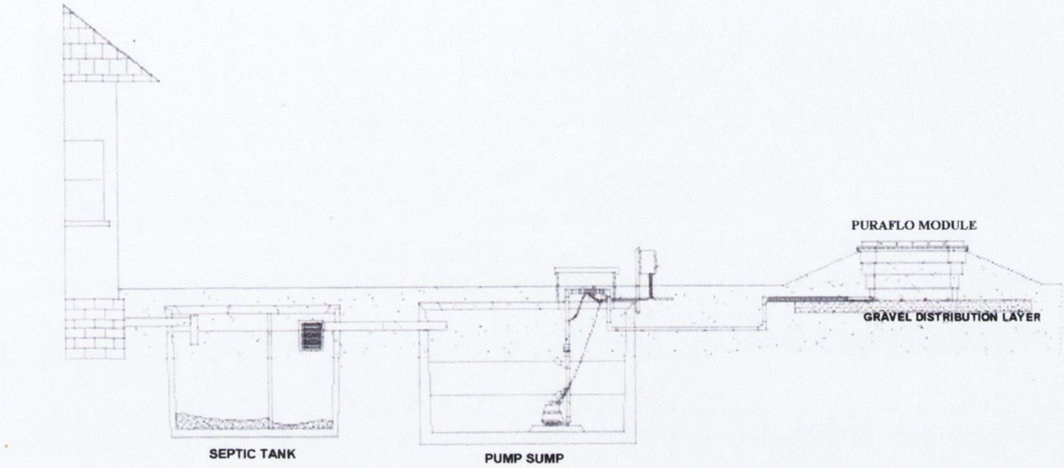


Figure 6.1 Layout of typical Puraflo treatment system.

Figure 6.2 Open Puraflo® module containing fibrous peat media. Note aeration holes along sides of lid and also the collection sump in left of photograph.

combination of physical (filtration and adsorption), chemical (adsorption and ion exchange) and biological (microbial assimilation) processes resulting from the interactions between the effluent and the peat media and associated biofilm (Henry, 1996). Most of the biological processes are carried out by aerobic and faculatively aerobic heterotropic bacteria which adhere to the surface of the peat media. The larger number of heterotrophic bacteria are found in the upper portions of the filter media with nitrifiers becoming more prevalent at depths of 30cm or greater where organic concentrations are relatively low. Therefore, the degradation and assimilation of the carbonaceous element of the waste is effected within the upper portions of the filter bed with nitrification occurring at greater depths provided that sufficient oxygen is available.

In the standard Puraflo® system the treated effluent percolates through the base of the module into gravel distribution trenches from where it enters the subsoil. However, modifications to the design of the system to allow the effluent to be gravity fed to the percolation trenches meant that the treated effluent was collected in a sump from where it flowed to the distribution box. It is from this sump that samples were obtained.

6.2 Construction of Percolation Trenches

6.2.1 Background

As outlined in Section 5.2.3 all sites required the construction of parallel percolation trenches each of 20m length, four on Sites 1 and 2 and two on Sites 3 and 4 (Figure 6.3). While it is recommended that the invert of the percolation trench should be 800mm below ground level along its full length (Figure 2.4), and this was achieved at Sites 2 to 4, this depth was unattainable over the whole length of trench on Site 1 due to its topography. The percolation area on Site 1 is located in a corner section of a field from which the land falls in all directions. The area required for the percolation trenches was marked out and spot levels taken along the ground surface (Table 6.1). Ground level above the start of Trench 1 was taken as datum. As can be seen from Table 6.1, in order to achieve an invert depth of 800mm at the end of each trench, a deeper excavation would be required at the start; for example, Trench 4 would require the invert at the start of the trench to be at 1.38m below ground level. However, this would not allow the installation of the instrumentation to the required depth.

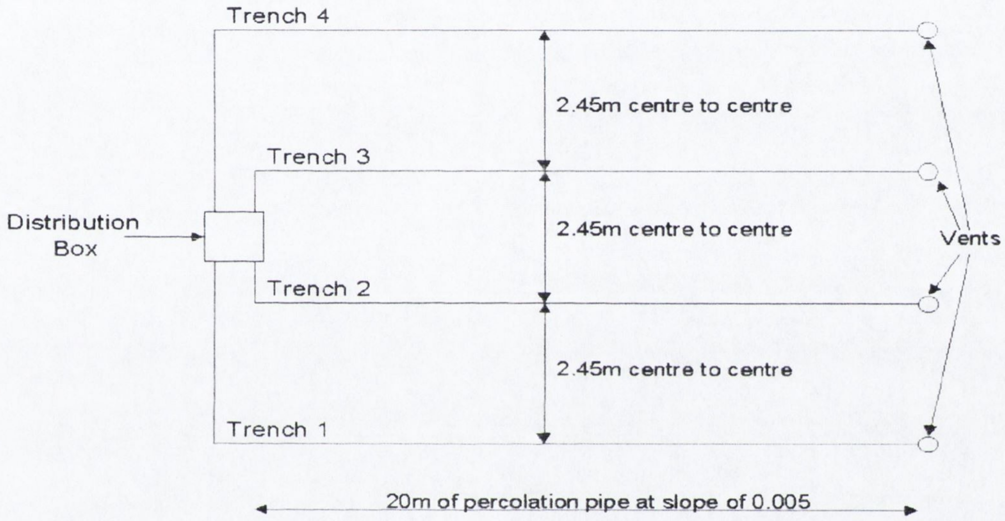


Figure 6.3 Plan of percolation area (EPA, 2000).

Distance along Trench (m)	**Trench Number**			
	1	**2**	**3**	**4**
0	0	-0.178	-0.206	-0.256
5	N/A	N/A	-0.436	-0.556
10	-0.266	-0.396	N/A	N/A
20	-0.381	-0.571	-0.786	-0.936

Table 6.1 Levels, in metres, taken at ground surface along the length of proposed percolation trenches (relative to ground level at start of Trench 1).

The depth at which the T-test was carried out and the depth to which the trial hole was excavated must also be considered. Even assuming that the results obtained from the on-site assessment are representative of the layers in which they were performed, and that these layers are isotropic and homogeneous, they might not be representative of the subsoil at the invert of the trenches should the trenches be excavated to a greater depth. As it is the depth and characteristics of subsoil below the base of the percolation trench, rather than that above it, that is of critical importance for effluent treatment it was decided to raise the trenches so that the recommended EPA trench invert depth of 800mm was achieved at the midpoint (10m) of each trench (Table 6.2).

Distance along Trench (m)	Trench Number			
	1	2	3	4
0	0.946	1.000	1.130	1.225
20	0.665	0.607	0.650	0.645

Table 6.2 Depth, in metres, of trench inverts below ground level at 0m and 20m.

6.2.2 Trench Construction

Prior to construction all material specified by the EPA had to be sourced and delivered to site (Table 6.3). It was necessary, however, to alter the design specifications slightly due to four problems encountered:

1) The percolation pipes specified were of 100mm bore whereas the standard sewer pipe bore in Ireland is 110mm and it was therefore necessary to use pipe of this bore.
2) As both percolation areas on Sites 1 and 2 were located on grazing land the site owners were anxious that land wastage would be kept to a minimum. Rather than installing the vents at the end of each percolation trench (Figure 6.3), it was decided to install them at the start, as this reduced land usage – easier to fence off area adjacent to site boundary.
3) As each trench was treated as a separate entity for research purposes they were not interconnected.
4) While the required trench width is 450mm, a 457mm bucket excavator was used.

Description	Approximate Quantity
Distribution box	1
20–30mm washed gravel*	21.12m^3 (35.52t)
110mm (4") percolation pipe	84m (supplied in 6m lengths)
110mm (4"") sewer pipe	12m (supplied in 6m lengths)
110mm (4") sewer swept bend	4
110mm (4") sewer T branch	4
110mm (4") sewer collar	4
110mm (4") sewer plain stopper	4
110mm (4") soil pipe cowl	4
Geotextile	36m^2

* density of 6.35 –50.8mm gravel = 1682kg/m^3 (Source: www.metric.fsworld.co.uk)

Table 6.3 Material required for the construction of a percolation area consisting of 80m of trenches.

There are a number of stages involved in the construction of a percolation trench (Figure 6.4). Prior to excavation, the location of the four percolation trenches was marked on the ground surface to aid the JCB driver. To begin with, one of the outside trenches was excavated first, Trench 4 at Site 1 and Trench 1 at Sites 2 to 4; in each case, the soil was left outside the percolation area and later used to backfill the last trench excavated. The soil from the excavation of subsequent trenches was used to backfill the previously excavated trench.

Some of the construction stages are more critical than others and therefore require more attention to detail. Levels were taken along the trench base, at the top of the first gravel layer and on the top of the percolation pipe to ensure that a slope of 0.005 was being achieved to promote even distribution of the effluent. Also of importance was the occurrence of smearing on the trench base and walls due to the compaction and/or glazing of the soil by the bucket – a problem that increases with increasing clay content. As Figure 6.4 (c) highlights this required scouring with a garden rake to expose a natural soil surface and so prevent reduced infiltration.

(a)

(b)

(c)

Figure 6.4 Construction of percolation area:

(a) Trench excavation.

(b) Smearing on trench wall.

(c) Raking trench wall.

(d) Pouring initial layer of distribution gravel – note white guide tubes for instrumentation.

(e) Percolation pipe in place.

(f) Geotextile in place.

(g) Backfilling trench with soil from the next trench excavation.

(d)

(e)

(f)

(g)

The gravel was placed in the trench in two phases, an initial layer of 250mm thickness as a distribution layer below the percolation pipe and a subsequent 250mm thick layer to protect the pipe. Prior to the placement of the distribution gravel in the base of the trench, guide tubes were inserted to allow for the future installation of the monitoring instrumentation (Figure 6.4 (b); Section 7.3.2). Prior to backfilling, a geotexile, Terram® in this case, was placed over the second gravel layer to prevent fines being washed into the distribution gravel and causing clogging.

On completion of the percolation trenches the distribution box was installed and levelled and a trench excavated on both sides to allow its connection with the percolation pipes (Figure 6.5). The connecting pipes were surrounded by gravel for protection and the trench backfilled with soil. To avoid the creation of preferential flow paths that would facilitate effluent movement from the distribution gravel this trench was excavated to the depth of the invert of the percolation pipes only and the area around the end of each percolation trench was backfilled with soil. On completion of the percolation areas, grass seed was sown on all sites to achieve representative evapotranspiration rates.

Figure 6.5 Installation of distribution box and connecting pipes.

6.2.3 Distribution Box

The achievement of an equal loading rate on each trench depends upon the attainment of an even distribution within the distribution box, all other things being equal. After the commissioning of Sites 1 and 2 it was observed that neither distribution box (Figure 6.6) was producing an even split over the 4 trenches, and therefore some modification was required. As a temporary measure a length of 110mm diameter sewer pipe was cut in half along its length to produce a gutter and four 90° V-notch weirs, bevelled on the downstream side, were cut into it to correspond with the outlet channels. This modification was inserted into the inlet pipe and levelled. At low flows it was observed that the nappe failed to spring clear of the notch and clung to the underside of the pipe until it reached the lowest point from where it dropped into the channel below. To curtail this, and force the effluent into the individual outlet channels, the void area underneath the pipe was filled with cement, this also ensured that the pipe remained horizontal.

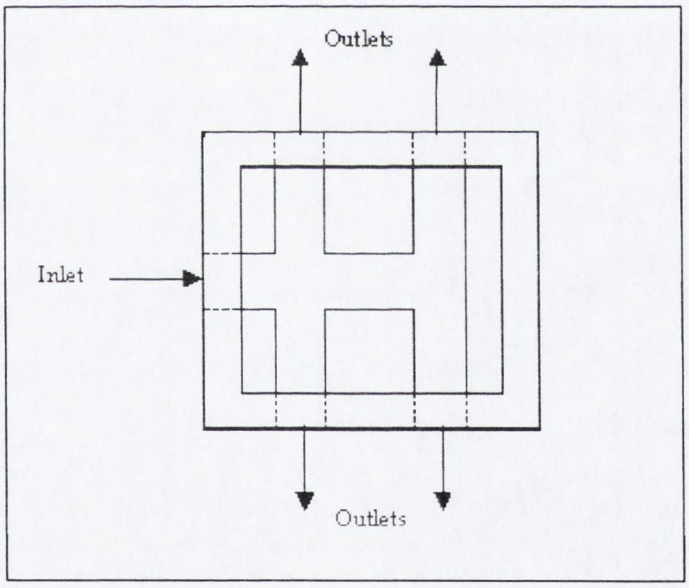


Figure 6.6 Plan schematic of distribution box installed on all sites.

As an adjunct to this project, an analysis was carried out as part of a final year project (Gill *et al.*, 2004) on effluent distribution within a similar distribution box using flow data obtained from Site 1 over an eight month period (July 2002 to February 2003). Frequency distribution analysis was carried out on the collated data to calculate the range of flows and the most frequent flow rate recorded. It was discovered that most flows recorded fell within the 0 - 20l/min range, with 2l/min being the most frequent. The behaviour of the distribution box under laboratory conditions, receiving flows between 0.5 and 20l/min, was examined: flows were increased by 1l/min increments up to 10l/min and by 2l/min increments thereafter up to 20l/min. Furthermore, it was decided to examine distribution box behaviour in response to shock loads of between 1l and 5l - where a shock load is the instantaneous release of a volume of water into the inlet pipe.

In summary it was found that, at all flowrates, the two outlet channels at the rear received the highest volumes of water, between 60 and 93%, with the right hand one clearly favoured at flowrates below 6l/min. It was not until a flowrate of 6l/min was achieved that the front left outlet received any flow. The same distribution pattern was experienced when the system was tested with shock loads. While it was noted that the slope of the front left hand outlet channel was less than that of the corresponding channel on the right, thus preventing water from exiting the channel at low flows, this was not envisaged to have affected the distribution between front and rear outlets. To counteract this distribution problem a form of storage and side weirs similar to that already installed at the two sites, but designed to sit into the distribution box rather than inserted into the inlet pipe, was tested (Figure 6.7). The location of the V-notch weirs with respect to the invert of the modified pipe, and their effect on its storage capacity, was critical in achieving an improved distribution. While a storage volume was required to dissipate the influent velocity and prevent distribution favouring the two weirs at the rear, it was important that the storage capacity:

(a) Should not be such as to promote deposition of suspended solids in the gutter, and
(b) should not result in build-up of liquid in the inlet pipe.

Various pipes with weirs cut at distances of 20mm, 30mm, 40mm and 50mm (and various combinations of same) from the invert were prepared and their performances were recorded under the same conditions under which the unmodified distribution box was tested (the distances were measured along the internal circumference). It was found that the pipes with weirs at 30mm and 40mm, respectively, performed best. While the former gave the required distribution at low flows, the latter was found to perform best over the whole range of flows and was installed on the test sites. It was also found that at high flows the two weirs at the rear were always favoured and therefore some form of baffle located between the front and rear weirs might be advisable to further dissipate the influent flow velocity.

Figure 6.7 Installed modification to standard distribution box – note the occurrence of deposition under field condition.

As the flowrates at Site 1 were measured upstream of the Puraflo® system the damping effect of this system on the flow profile must be considered. A shallower flow profile than that measured at the septic tank outlet would be expected at the distribution box thereby ensuring that, should the recorded data be representative of the annual flow profile, flowrates experienced here would be within the lower end of the range of those tested. This was corroborated by site inspections in which it was observed that, subsequent to saturation of the peat media, a continuous flow seemed to be entering the distribution box. The ability of the 40mm weir to provide improved distribution over a broad range of flowrates, allied with the damping effect experienced over additional piping (the laboratory analysis was carried out using only 6m length of sewer pipe), warranted its installation on Sites 2 to 4 even though no flow data were available for these test sites during the period of distribution box testing (Section 7.2.2). While the weirs provided improved distribution they required regular inspection and maintenance, which involved cleaning with a small brush, to ensure a continued even flow distribution.

7. INSTRUMENT INSTALLATION AND SAMPLING AND ANALYSIS METHODOLOGY

7.1 Introduction

The successful installation of instrumentation to facilitate the recording of a number of parameters over the research period was an essential component of the research (Figure 7.1). Automatic samplers and flow monitors were installed downstream of the septic tanks, and secondary treatment system to obtain a profile of the effluent entering the percolation trenches. Suction lysimeters (which will be referred to from now on as lysimeters) and zero-tension samplers were installed along the length of the percolation trenches to obtain soil moisture samples for analysis for some of the characteristic constituents of domestic wastewater effluent (COD, NH_4, NO_2, NO_3, PO_4-P, Cl and enteric bacteria). Rainfall volume, evapotranspiration and chloride concentration were also analysed to determine the effect of dilution on the system. Tensiometers were installed to monitor the soil moisture pressure below the percolation area.

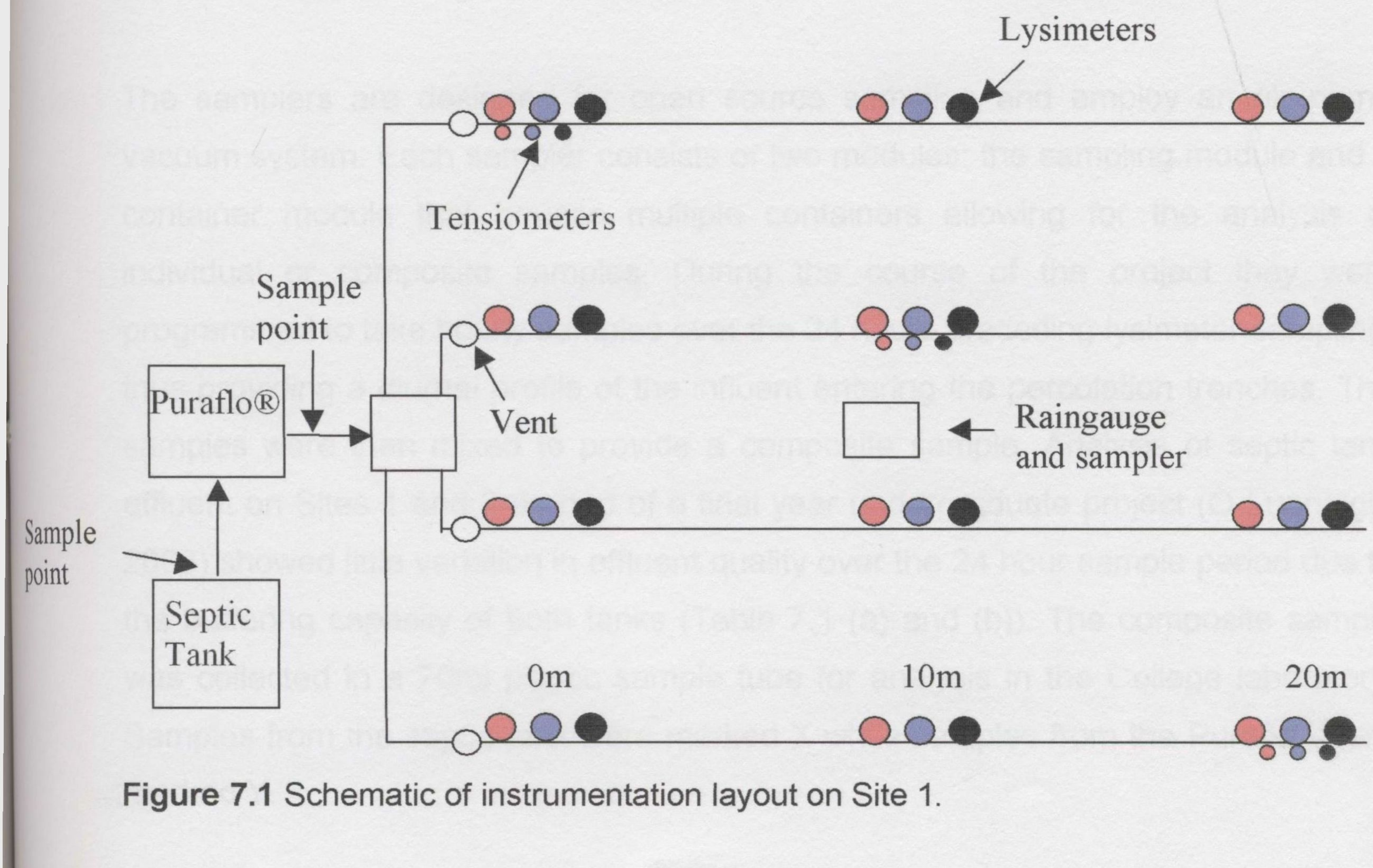


Figure 7.1 Schematic of instrumentation layout on Site 1.

7.2 Monitoring of Percolation Trench Influent

7.2.1 Septic Tank and Secondary Effluent Sampling

Bühler Montec xian 1000 automatic samplers (Figure 7.2) were installed on all sites, two on Sites 1 and 3 (downstream of the septic tank and Puraflo® respectively) and one on Sites 2 and 4 (downstream of the septic tank). The installation of samplers upstream and downstream of the Puraflo® system enabled the changes in effluent quality across the secondary treatment system to be assessed.

Figure 7.2 Bühler Montec xian 1000 automatic sampler.

The samplers are designed for open source sampling and employ an air pump vacuum system. Each sampler consists of two modules: the sampling module and a container module that houses multiple containers allowing for the analysis of individual or composite samples. During the course of the project they were programmed to take hourly samples over the 24 hours preceding lysimeter sampling, thus providing a diurnal profile of the influent entering the percolation trenches. The samples were then mixed to provide a composite sample. Analysis of septic tank effluent on Sites 1 and 2 as part of a final year undergraduate project (O Luanaigh, 2003) showed little variation in effluent quality over the 24 hour sample period due to the buffering capacity of both tanks (Table 7.1 (a) and (b)). The composite sample was collected in a 70ml plastic sample tube for analysis in the College laboratory. Samples from the septic tank were marked X while samples from the Puraflo® were marked Y.

Time	Concentration (mg/l)			
	COD	NH_4-N	PO_4-P	Cl
09:30 – 12:30	705	113.2	14.4	88
12:30 – 15:30	710	114.8	14.6	94
15:30 – 18:30	708	107.2	15.1	92
18:30 – 21:30	630	90	11.8	72
21:30 – 00:30	736	112	14.2	88
00:30 – 03:30	808	110	13.9	90
03:30- 06:30	750	109.2	14	85
06:30 - 09:30	762	108	15.3	85
Average	**729.1**	**108.1**	**14.2**	**86.8**

(a)

Time	Concentration (mg/l)			
	COD	NH_4-N	PO_4-P	Cl
10:00 – 13:00	323	69.2	12.8	84
13:00 – 16:00	226	66.4	11.9	70
16:00 – 19:00	338	66.8	11.6	60
19:00 – 22:00	296	69.6	14.2	58
22:00 – 01:00	310	66.8	13.8	62
01:00 – 04:00	284	66	13.3	69
04:00 – 07:00	328	66.4	11.7	82
07:00 – 10:00	332	672	11.6	83
Average	**318.3**	**67.3**	**12.6**	**71**

(b)

Table 7.1 Example of concentration variation within septic tank diurnal samples obtained from (a) Site 1 on 30/01/03 and (b) Site 2 on 23/12/02 (O Luanaigh, 2003).

7.2.2 Flow measurement

Initial measurement of the flowrate into the distribution box was attempted on both Sites 1 and 2 using a Bühler Montec xytec 7300 which is a wrap-around ultrasonic flowmeter. It consists of two ultrasonic transducers secured on opposite sides of a submerged pipe and connected to a microprocessor. This method was unsuccessful due to the sporadic nature of the flow entering the distribution box which resulted in very short duration pulses of low velocity flow in the pipe. It was decided to reduce the diameter of the submerged pipe from 110mm to 50mm in order to increase the cross-sectional flowrate, thus increasing the signal available to the transducers. However, this did not have the desired affect as the meter took discrete readings at fixed time intervals and a different method of flow measurement was therefore undertaken.

On Site 1 it was possible to install an ultrasonic level detector (Lascar EasyLog datalogger and Milltronics ultrasonic probe) in the pump-sump downstream of the septic tank (Figure 7.3). The floor plan shown is a cross-section taken at the half way distance between the top and the base of the sump. The level detector, which was set to record the effluent level every second for 5.5 days, was attached to a collar that was cemented onto the sump roof (Figure 7.4).

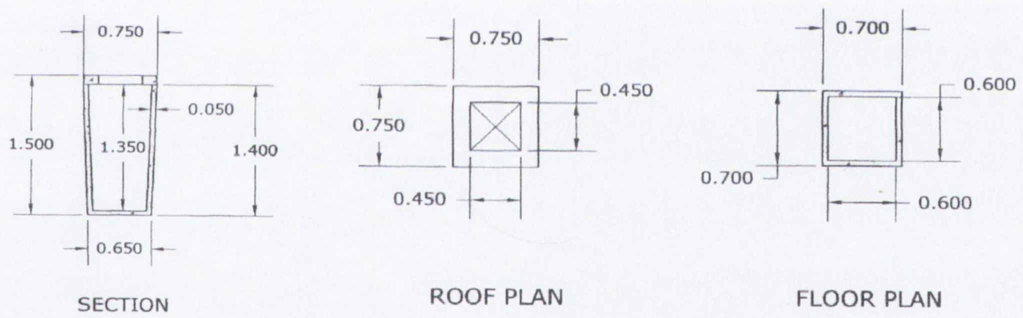


Figure 7.3 Plan of sump installed on Site 1 (Carlow Concrete).

Figure 7.4 Easylog ultrasonic level detector on Site 1.

The change in effluent level within the sump over any time step, and hence the volumetric flowrate of septic tank effluent, was calculated by obtaining the difference in volume calculated using the horizontal surface area of the effluent at any time T_1 and T_2:

At T_1 the horizontal surface area $\mathbf{E^2 = [0.55+2((D_0-D_1)*0.03)]^2}$

where, D_0 = distance from level sensor to base of sump
D_1 = distance from level sensor to effluent surface at T_1

at T_2 the horizontal surface area $\mathbf{F^2 = [E+2((D_1-D_2)*0.03)]^2}$

where, D_2 = distance from level sensor to effluent surface at T_2

Therefore,

$$\mathbf{dV/dT = [(F^2-E^2)\Delta D]/2}$$

where, dV/dT = increase in volume over any time step T_1- T_2.
$\Delta D = D_1-D_2$

The absence of such a sump on Site 2 required the investigation of alternative options. Contact was made with numerous flowmeter manufacturers but no suitable device was found, with most manufacturers stating that the nature of the flow (pulse-like, low volume and high suspended solids content) did not lend itself to measurement by orthodox methods. It was therefore decided to install another ultrasonic level monitor (Omega-SL-L320 datalogger and LVU-90 ultrasonic level sensor) to measure the septic tank effluent (STE) flowrate into the percolation area. The use of a level monitor required the installation of a pump sump to allow storage of a volume of STE (Figure 7.5). It was essential that this volume of effluent was large enough to allow accurate calculation of flowrate while at the same time small enough to minimise the effect on the natural flow profile of STE entering the percolation area. A further constraining factoring was that the sump also had to be of

Figure 7.5 Septic tank effluent flow measurement on Site 2.

sufficient size to house the pump. Unlike Site 1, the flowrate on Site 2 was calculated by measuring the volume of STE pumped each time the pump was triggered (4.75l) and using the level monitor to detect when each pumping event took place. To promote quiescent conditions within the sump for accurate level detection a 90° bend was attached to the influent pipe to act as a baffle. It was necessary to reduce the pipe diameter from 110mm to 50mm at the point of inflow due to the constricted space; this was done using a standard reducer with a funnel, with the stem removed, inserted to give a tapering effect thus avoiding deposition within the pipe.

Weekly flow regimes were recorded by the respective dataloggers and downloaded (Figure 7.4). While the favourable site conditions at Sites 1, 3 and 4 allowed the compilation of flow profiles for the entire sampling periods, the problems experienced on Site 2 prevented this. As a result flow records, which commenced in February 2003, are only available for a six-month period. However, it is considered that this reduced data set still gave a representative hydraulic loading regime for the site.

7.3 Percolation Trench Monitoring

7.3.1 Introduction

As alluded to in Section 6.2.2, nine 50mm pipes approximately 1m in length were inserted into each percolation trench during construction to facilitate the installation of Soilmoisture Equipment Corporation suction lysimeters below the trench (Figure 6.4 (d) – (g)). These guide pipes were placed in groups of three at 10m intervals (Figure 7.6). When installed, each trio consisted of different length lysimeters, 1.3m (red), 1.6m (blue) and 1.9m (black), installed to different depths below the trench invert. Smaller lysimeters (25.4cm) were also placed directly under the invert of the percolation trench using a combination of mechanical excavation and hand auger.

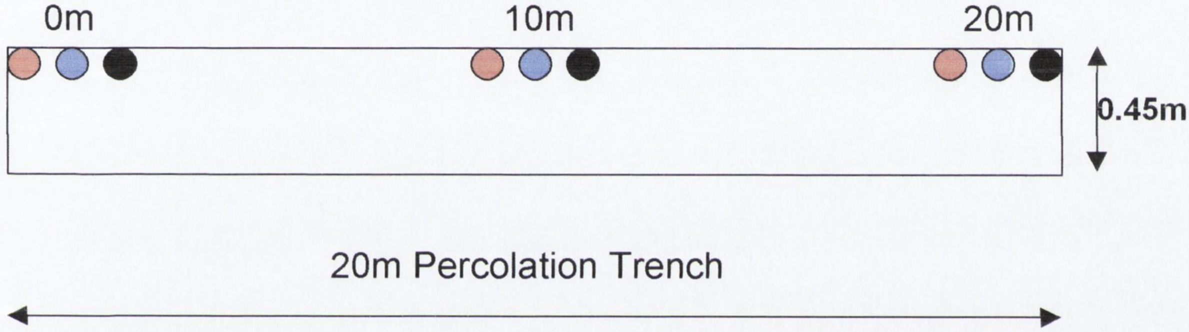


Figure 7.6 Plan view of single percolation trench with location of lysimeter guide tubes.

Three guide pipes, 1m in length, were installed at 0m, 10m and 20m along trenches 4, 3 and 1, respectively, at Site 1 and on trenches 1, 2 and 4, respectively, at Site 2 to facilitate the installation of nine Soil Measurement Systems tensiometers. On Sites 3 and 4 three tensiometers were installed at the 0m and 20m position on trenches 1 and 2 respectively. This was due to damage caused to six tensiometers on removal from Sites 1 and 2. Each trio consisted of tensiometers of 1.0m (red), 1.5m (blue) and 2.0m (black) in length that were installed to different depths below the invert of the percolation trench. Each site also contained a rain gauge and rain sampler.

7.3.2 Lysimeter Installation and Operation

Lysimeter Installation

Lysimeters (Figure 7.7) were used to collect moisture samples from the soil profile. They were installed to a desired depth and left in the soil for the duration of sampling, allowing periodic sampling to occur with minimal disturbance of the subsoil. The samplers consisted of a porous cup (48mm OD) and sample collection tube (48mm OD). The lid of the tube consists of a Santoprene stopper with neoprene tubing attached as an access tube for air evacuation and sample extraction. A vacuum is created within the lysimeter using a vacuum pump and moisture extracted from the soil matrix.

Figure 7.7 Percolation trench monitoring instrumentation and equipment required for its installation. From left of photograph: 2.0m tensiometer, 1.6m lysimeter, steel reinforcing bar, gouge auger, combination auger, screw auger, auger extension bar, auger handle and stilson wrenches.

To install the lysimeters on Sites 1 and 2 a screw/spiral auger of 40mm diameter (Figure 7.7) was inserted through the guide tube to bore an initial hole to the desired depth. It was intended to install all similarly coloured lysimeters to the same nominal depth below the percolation trench invert but this was not always possible and some

		Tip depths (m) below percolation trench invert			
		Site 1			
Distance	Colour	Trench 1	Trench 2	Trench 3	Trench 4
0m	Red	0.24	0.32	0.49	0.37
	Blue	0.68	0.58	0.91	0.61
	Black	0.74	0.87	1.10	0.79
10m	Red	0.39	0.40	0.62	0.49
	Blue	0.75	0.83	1.08	0.85
	Black	1.10	1.05	1.34	1.17
20m	Red	0.41	0.41	0.6	0.33
	Blue	0.83	0.84	1.14	0.88
	Black	1.14	1.20	1.48	1.19
		Site 2			
Distance	Colour	Trench 1	Trench 2	Trench 3	Trench 4
0m	Red	-	0.25	0.16	0.40
	Blue	0.55	0.72	0.75	0.70
	Black	0.74	0.95	0.99	0.96
10m	Red	0.33	0.35	0.25	0.39
	Blue	0.72	0.77	0.53	0.56
	Black	1.06	1.03	0.94	1.00
20m	Red	0.47	0.30	0.23	0.40
	Blue	0.59	0.70	0.62	0.81
	Black	-	0.78	0.82	0.81
		Site 3		Site 4	
Distance	Colour	Trench 1	Trench 2	Trench 1	Trench 2
0m	Red	0.25	0.17	0.23	0.35
	Blue	0.62	0.44	0.50	0.52
	Black	0.92	0.82	0.96	1.11
10m	Red	0.28	0.24	0.13	0.23
	Blue	0.57	0.68	0.66	0.54
	Black	1.28	0.85	0.99	1.01
20m	Red	0.17	0.16	0.30	0.30
	Blue	0.81	0.47	0.56	0.63
	Black	0.95	0.78	0.88	0.97

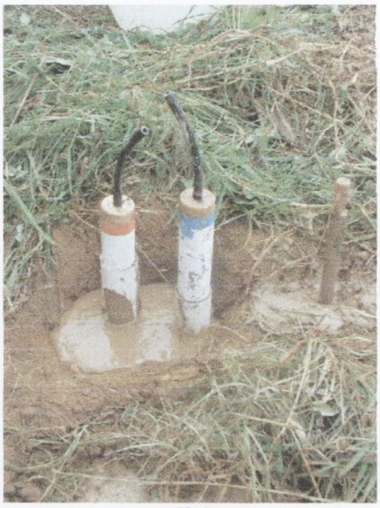
Table 7.2 Lysimeter tip depth below percolation trench invert on Sites 1, 2, 3 and 4.

lysimeters had therefore to be reclassified (see Sections 8.4.1, 9.3.1, 10.4.1 and 11.3.1). Table 7.2 shows the depth below the percolation trenches to which the lysimeters were installed. As the external diameter of the lysimeter tube was 48mm subsequent augering using a 45mm diameter combination auger (Figure 7.7) was required to increase the bore of the access hole. The initial augering step was required as a screw auger is more robust than a combination auger and subsoil conditions, especially at Site 2 where an unsorted till was encountered, would have resulted in damage to the combination auger.

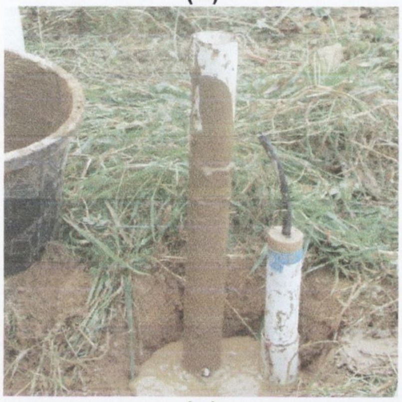
While the subsoil conditions at Site 1 would have allowed the use of the combination auger alone this would have slowed the installation process due to its reduced capacity for subsoil removal compared to the screw auger. After each hole was bored the relevant lysimeter (1.3m, 1.6m or 1.9m) was inserted through the guide tube and pushed down to the base of the hole to ensure it fitted. The presence of cobbles in the subsoil on Site 2 inhibited the insertion of some lysimeters and required the use of a steel reinforcing rod and lump hammer to remove the protruding obstructions. The use of an auger with a diameter slightly smaller than the external diameter of the lysimeter was intended to result in good contact between the porous cup and the subsoil matrix.

To maximise contact between the porous cup and the soil matrix the guide holes were partially backfilled with a soil slurry produced by mixing excavated soil, from which any gravel had been removed, with water. The lysimeter was then inserted through the guide tube and pushed into the slurry until the base of the hole was reached. Where the top of the guide tube was flush with, or below, ground level a conical trench was dug around it. The slurry was then poured into the conical trench and the void between the lysimeter and the guide tube also filled. The guide tube was then extracted. A bentonite slurry, consisting of bentonite powder, soil and water, was mixed and poured into the void between the lysimeter and the subsoil. This bentonite slurry was then tamped to ensure a good seal. Where the top guide tube protruded above the ground surface a circular bund was constructed which functioned in a similar manner to the conical trench. Figure 7.8 illustrates the installation process.

(a)



(b)



(c)

(d)

(e)

Figure 7.8 (a) – (e) Lysimeter installation

(e)

Due to the dense nature of the subsoil on Sites 3 and gravelly nature of the subsoil on Site 4 it was not possible to install the lysimeters using the hand augers so a mechanical auger called a Minute Man was used which had a diameter of 6.5cm (Figure 7.9). As a result the lysimeters were installed adjacent to, rather than in, the percolation trenches. The installation process only differed from that on Sites 1 and 2 by the virtue that -no guide tubes being required.

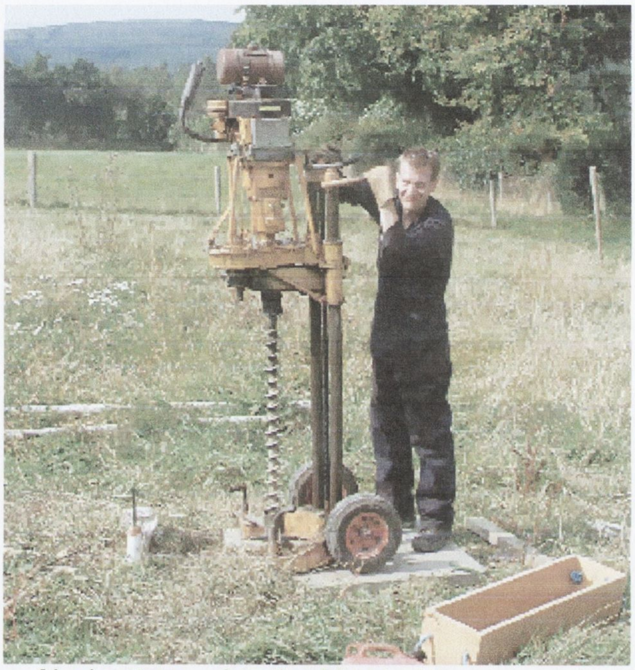


Figure 7.9 Installation of lysimeters using the Minute Man.

Smaller lysimeters (25.4cm) were installed on Sites 1 and 2 directly under trenches 3 and 4 on Sites 1 and 2 at distances of 2m, 7m and 15m along the trench length to monitor the development of the biomat along the trench base. To facilitate their installation a 1.0m deep hole was excavated between the trenches at each proposed sample point. The augers were then used to install the lysimeters at an acute angle to the horizontal. The installation of these lysimeters was not entirely successful, only providing soil moisture samples over a short period of the sampling period.

Lysimeter Sampling

On the morning preceding sampling all lysimeters were put under a suction of 50cbar using a vacuum-pressure hand pump. While a potential gradient is required to draw moisture from the soil matrix into the lysimeter, too great a suction would result in moisture, that would otherwise be unavailable to recharge due to the adhesive forces between the moisture and the soil matrix, being extracted. Soilmoisture Equipment Corporation advise that the practical limit for water flow in soils is about 65cbar, although in some soils this value can approach 85cbar, and it was hoped that the application of a suction of 50cbar would prevent the extraction of bound moisture. This suction was also recommended by Teagasc (Personal communication 2002). On application of suction the neoprene tubing of the lysimeter was clamped with a plastic ring.

Sampling generally commenced at 08:30 with the clamps on all lysimeters being released and analysis commenced by 13:00, generally concluding by 18:00. The extraction kit for sampling consisted of the vacuum-pressure pump and a 1000ml conical flask and rubber bung with an extraction tube attached (Figure 7.10). Two types of extraction tube were used:

(i) 2.4mm O.D. plastic tube that was inserted into the neoprene tube and pushed down to bottom of the sampler
(ii) 6.4mm O.D. plastic tube that was inserted into the sampler after the stopper was removed.

Figure 7.10 Lysimeter sample extraction kit.

To extract a sample the vacuum pump was connected to the side arm of the conical flask and the extraction tube inserted into the lysimeter. The full sampling methodology is outlined in Appendix B. Initially sampling was carried out using the smaller diameter extraction tube but this was unsatisfactory as it regularly became kinked, and over time these kinks punctured. These problems were overcome by using the larger diameter tubing. The total volume of sample collected in each lysimeter was recorded, samples collected in 70ml sterilised plastic sample tubes and numbered according to Figure 7.11.

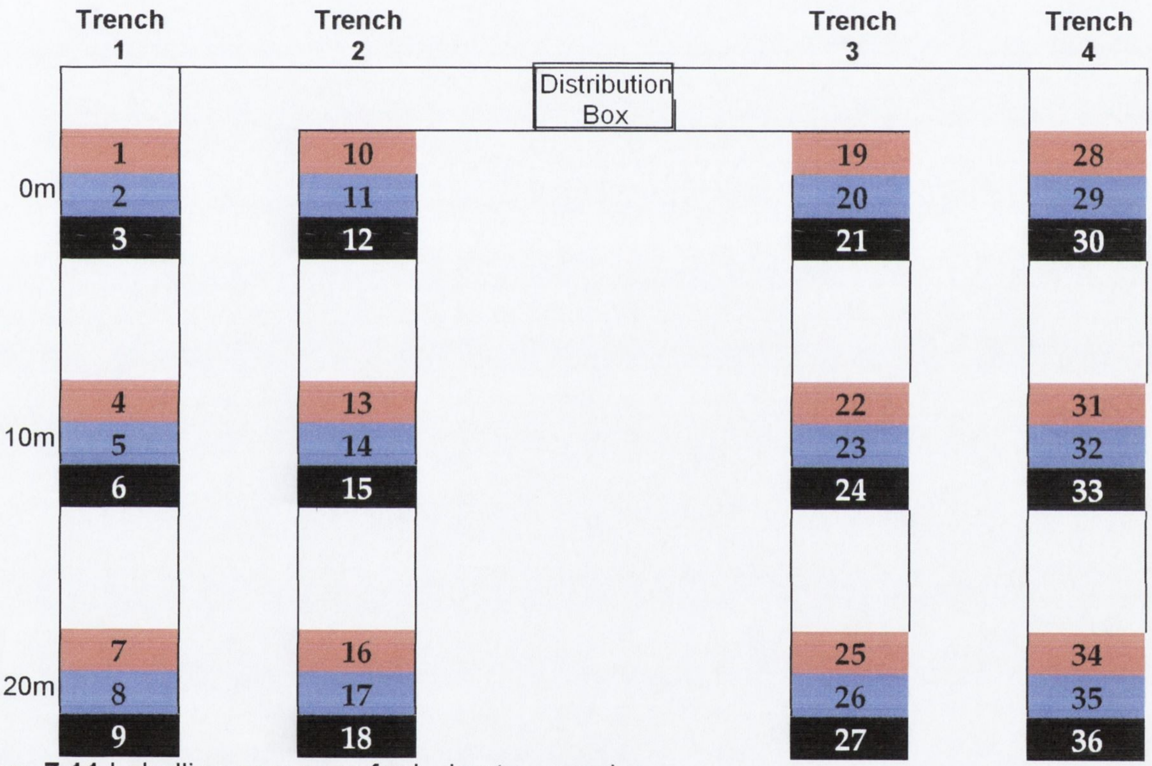


Figure 7.11 Labelling sequence for lysimeter samples.

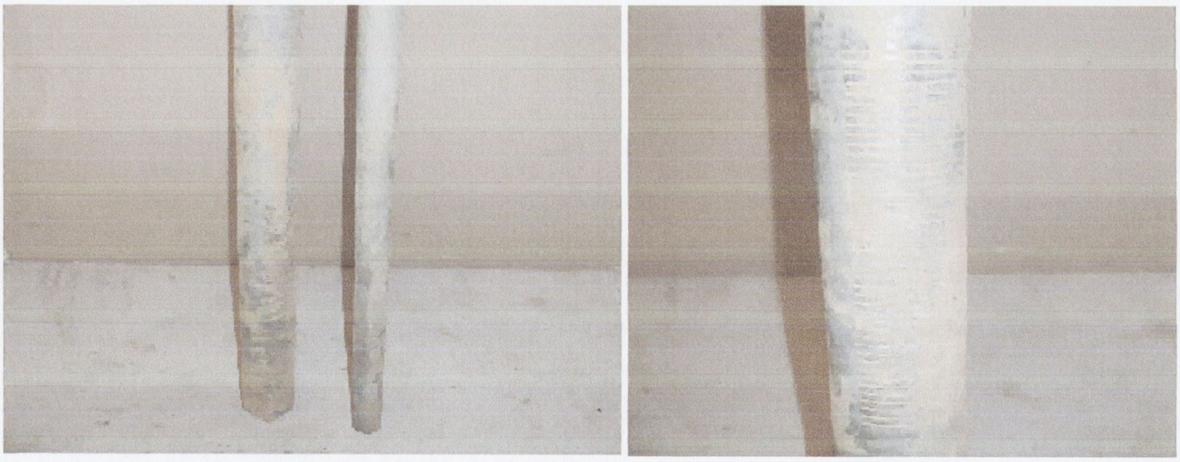
7.3.3 Zero Tension Sampler Installation

As lysimeter samples were collected under suction it was decided that samples should also be collected at ambient pressure to ensure that the lysimeter samples were representative of the percolating effluent. To this end zero tension samplers were designed and two prototypes produced (Table 7.3). Two of these samplers, which consisted of galvanised steel tubes with pointed tips (Figure 7.12 (a)), were inserted under trench 4 on Sites 1and 2, the smaller bore sampler at a distance of 2m and depth of 0.2m below the trench invert and the other at a distance of 7m and depth 1.2m below the trench invert. The protruding steel tubes were sealed with a

Sampler	OD	Grid Length	Grid distance from tip	Gap Width	Bar Width
Small Bore	3.35cm	25cm	27cm	2mm	5mm
Large Bore	6.0cm	25cm	31cm	2mm	4mm

Table 7.3 Specifications of zero tension samplers.

bentonite slurry and the top of the samplers covered to prevent infiltration. The samplers were installed at a 45° angle using the back-bucket of a mechanical excavator. A line was drawn on the sampler tubing from the slotted section to the top to ensure that the sampler did not twist during installation and that the slotted section remained facing the ground surface. However, during the course of the field studies no samples were obtained from the zero tension samplers and on extraction it was discovered that the gaps in the slotted section had been clogged by subsoil – most probably during the installation process (Figure 7.12 (b)). As a result no zero tension samplers were installed on Sites 3 and 4.



(a) **(b)**

Figure 7.12 (a) Large and small bore zero tension samplers and (b) clogged large bore sampler.

7.3.4 Tensiometer Installation and Monitoring

Tensiometer Installation

Tensiometers measure the pressure potential or matrix potential of the soil, i.e. the force with which water is held in the soil. They are used to monitor the moisture status of the soil, and where installed at different depths, the pressure gradient within the soil profile. A tensiometer is basically a water-filled tube closed at the bottom with a porous ceramic cup and at the top with an airtight seal. Nine Soil Measurement Systems tensiometers (Figure 7.7), consisting of a 21.5mm OD plastic tube with a 22.2mm OD porous ceramic cup attached, were installed on both sites, three at 0m, three at 10m and three at 20m (Figure 7.1). Each trio comprised tensiometers 1.0m (Red), 1.5m (Blue) and 2.0m (Black) in length. In order to obtain a profile of soil moisture tension across the percolation area each trio was installed on a different trench (Figure 7.1). The depths to which the tensiometers were installed are outlined in Tables 7.4 to 7.6. As with lysimeter installation, plastic guide tubes (32mm ID) were inserted into the percolation trenches during construction to facilitate tensiometer installation. Installation proceeded along the same steps as lysimeter installation except that a gouge auger of 30mm diameter was used (Figure 7.7). Due to the nature of the subsoil on Sites 3 and 4 it was necessary to install the tensiometers using the Minute Man. As was the case with the lysimeter this resulted in them being located adjacent to, rather than in, the percolation trenches.

Distance	Colour	Trench			
		1	2	3	4
0m	Red	N/A	N/A	N/A	0.07
	Blue	N/A	N/A	N/A	0.47
	Black	N/A	N/A	N/A	0.60
10m	Red	N/A	N/A	0.49	N/A
	Blue	N/A	N/A	0.84	N/A
	Black	N/A	N/A	1.36	N/A
20m	Red	0.42	N/A	N/A	N/A
	Blue	**Broken**	N/A	N/A	N/A
	Black	1.29	N/A	N/A	N/A

Table 7.4 Tensiometer tip depth below percolation trench invert on Site 1.

Distance	Colour	Trench			
		1	2	3	4
0m	Red	0.07	N/A	N/A	N/A
	Blue	0.64	N/A	N/A	N/A
	Black	1.05	N/A	N/A	N/A
10m	Red	N/A	0.06	N/A	N/A
	Blue	N/A	0.63	N/A	N/A
	Black	N/A	1.07	N/A	N/A
20m	Red	N/A	N/A	N/A	0.17
	Blue	N/A	N/A	N/A	0.65
	Black	N/A	N/A	N/A	1.15

Table 7.5 Tensiometer tip depth below percolation trench invert on Site 2.

Distance	Colour	Site 3		Site 4	
		Trench		Trench	
		1	2	1	2
0m	Red	0.10	N/A	0.08	N/A
	Blue	0.65	N/A	0.69	N/A
	Black	0.93	N/A	0.89	N/A
20m	Red	N/A	0.17	N/A	0.11
	Blue	N/A	0.72	N/A	0.70
	Black	N/A	0.97	N/A	0.94

Table 7.6 Tensiometer tip depth below percolation trench invert on Sites 3 and 4.

Tensiometer Monitoring

Once installed the tensiometers were filled to within 12.5mm of the top with de-aerated de-ionised water. A septum stopper, which allows measurement of the vacuum within the top of the tensiometer while maintaining an airtight seal, was then inserted in the top of each tensiometer. As UV light degrades the septum stoppers it was important that they were covered to protect them from exposure to sunlight. Tensiometers work by producing hydraulic continuity across the ceramic cup. Under unsaturated conditions the water in the tensiometer is drawn through the porous cup until the water potential in the tensiometer is the same as the soil water potential producing a vacuum under the septum stopper. In contrast, as the soil moisture content increases water enters the tensiometer thus reducing the vacuum. The vacuum is measured, in mbar, using a tensimeter, which consists of a pressure transducer and attached needle that is inserted through the septum stopper (Figure 7.13). As the tensimeter measures the vacuum, the height (in cm) of the column of water in the tensiometer must be subtracted from the transducer reading to give the actual soil moisture tension. Under continuous unsaturated conditions it is necessary to periodically refill tensiometers with de-aerated de-ionised water.

Figure 7.13 Tensimeter monitoring soil moisture potential below a percolation trench. The tensiometer cover can be seen to the left of the tensimeter.

7.3.5 Rain gauge and Rain Sampler Installation

Each site also had a Casella® tipping bucket rain gauge installed (Figure 7.14) which consisted of a datalogger with attached tipping bucket (Figure 7.14(b)). The top of the rain gauge cover was funnel shaped and directed the precipitation onto one side of the tipping bucket. Each 0.2mm of precipitation caused the tipping bucket to tip and each tipping event was recorded by the datalogger. In general when siting a rain gauge consideration must be given to the effects of shading from surrounding obstacles. A general rule of thumb is that no object should be closer than four times its height to the rain gauge so as not to impede data collection. However, with respect to this project these guidelines could be ignored as it was the precipitation falling on the percolation area rather than the areal precipitation that was of interest. The rain gauges were therefore located in the centre of the percolation areas. Each rain gauge was secured to a 45cm square concrete slab and levelled with the aid of the attached spirit level. The rain gauge funnel and inlet mesh were cleaned weekly and the data downloaded periodically. The rain sampler, which was located beside the rain gauge, consisted simply of a clean plastic container that was anchored to prevent its disturbance. Rainfall samples were collected in a plastic container and analysed for Cl.

(a) **(b)**

Figure 7.14 Casella® rain gauge (a) with cover in place and (b) with cover removed exposing tipping bucket and internal datalogger.

7.4 Analysis Methodology

7.4.1 Chemical Analysis

All septic tank, Puraflo® and soil moisture samples were analysed for ammonium (NH_4-N), nitrite (NO_2-N), nitrate (NO_3-N), chemical oxygen demand (COD), orthophosphate (PO_4-P) and chloride (Cl) using a Merck Spectoquant Nova 60® spectrophotometer and associated reagent kits (Appendix C) which are USEPA approved. It should be noted that Cl analysis of samples proved difficult due to the nature of the test. Results, while accurate (Table 7.7), were not precise and sometimes showed a variance of up to 20 mg/l. For this reason it was necessary to analyse a sample three times and take an average value.

If parameter concentrations were outside the detectable limit of the specified reagent kit it was possible to dilute the sample with a known volume of distilled water. However, it is important to note that where a sample was diluted an error in the analysis would be multiplied by the size of the dilution step in the reported concentration. For example if a 1:20 dilution was used this would mean that a small error in the analysis would result in a 20-fold increase in this error in the reported concentration.

During the sampling period four duplicate sets of samples, seven samples in each set, were sent to CAL Ltd, an accredited laboratory in Dún Laoghaire, as a quality control measure. The results obtained from the samples analysed by CAL (method contained in Appendix C) were compared against those obtained in the TCD laboratory and, as Figure 7.15 highlights, there was a good correlation between the two sets of data. However, it is important to also consider the slope of the line which should be equal to 1 but in this case is equal to 0.8. Closer examination of the data for the individual parameters, however, reveals an especially poor relationship between both sets of COD results (Table 7.7). This could be due to the delay in sample analysis by CAL.

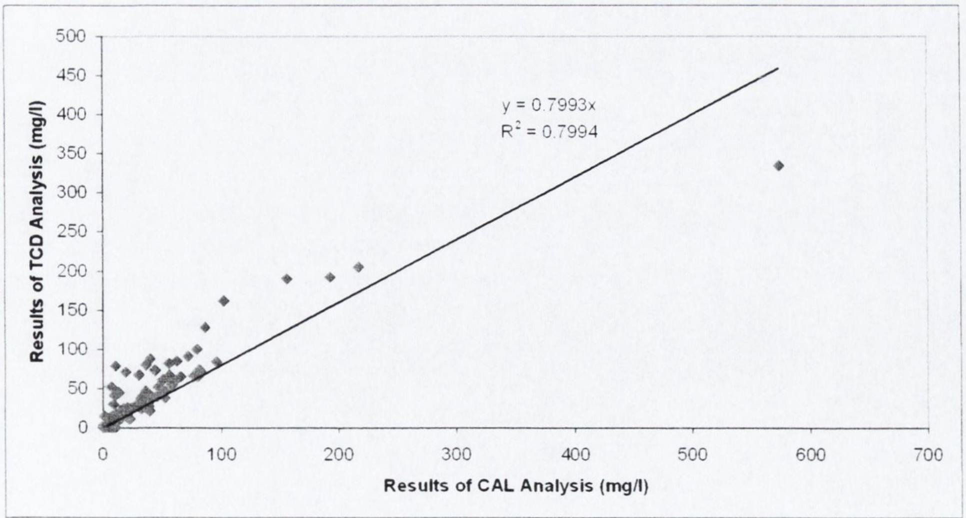


Figure 7.15 Graph of TCD results against CAL results for all parameters.

It can also be seen from Table 7.7 that TCD laboratory analysis for NO_3-N and Cl are very similar to results from CAL analysis for the same parameters. While there is a good correlation between both sets of results for NO_2-N, NH_4-N and PO_4-P analysis, with R^2 values of 0.98, 0.82 and 0.84 respectively, the results of sample analysis for the former parameter tend to be less than the CAL results while the latter two tend to be greater. When the results of the two sets of analyses minus the COD results are compiled and compared it shows that, overall, there is a good correlation (R^2 = 0.91) between the results of the analysis carried out in the TCD laboratories and those of the accredited laboratory (Figure 7.16).

Parameter	R^2	Slope
COD	0.62	0.77
NO_3-N	0.99	0.91
NO_2-N	0.98	0.80
NH_4-N	0.82	1.26
Ortho-P	0.84	1.36
Cl	0.92	0.89

Table 7.7 Results of comparison of CAL results against results obtained in the TCD laboratory.

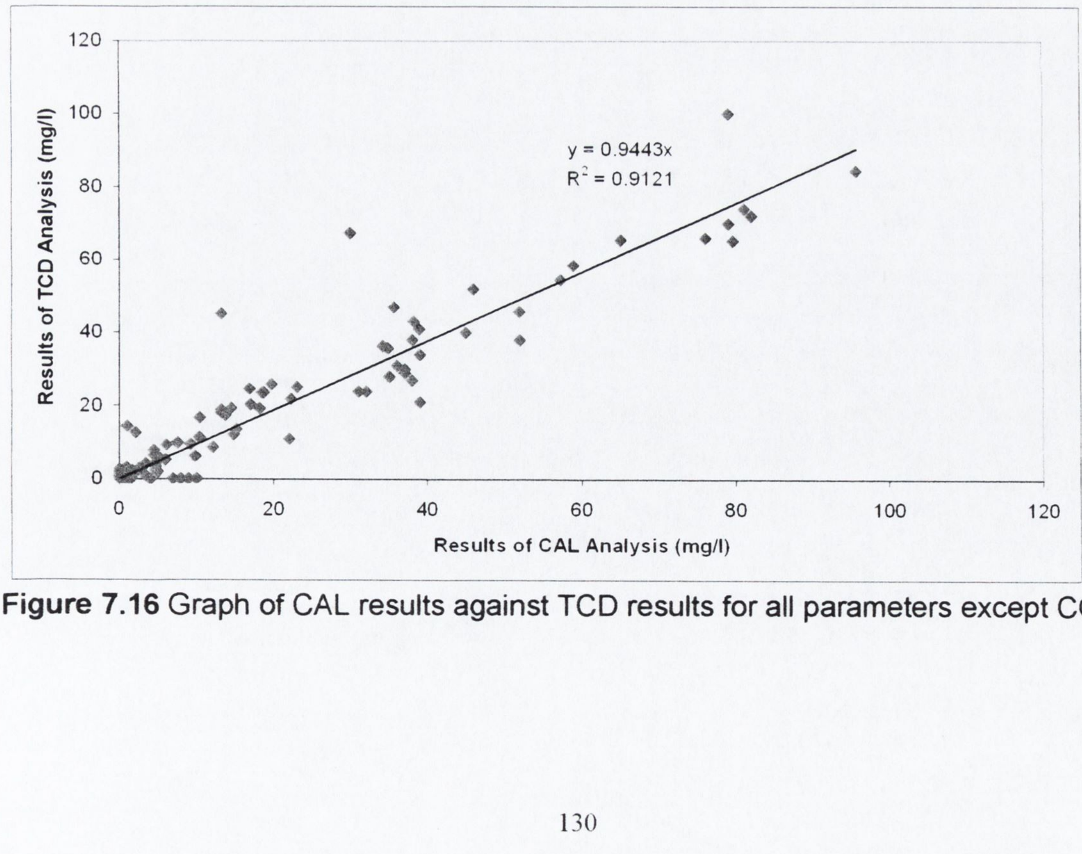


Figure 7.16 Graph of CAL results against TCD results for all parameters except COD.

7.4.2 Bacteriological Analysis

The faeces of a healthy person contains between 1 to 1000 million of each of the following groups of bacteria per gram: enterobacteria (of which *Escherichia coli* is a member), enterococci (of which faecal streptococci is a member), *Lactobacilli, Clostridia, Bacteroides, Bifidobacteria* and *Eubacteria* (Viessman and Hammer, 1998). During a disease event the number of pathogens in domestic effluent is a function of the number of carriers who excrete such organisms. As pathogenic bacteria are generally present in relatively small numbers compared to these indicator bacteria and as it is logistically impractical and prohibitively expensive to test water samples for all of the pathogenic bacteria potentially present samples are assayed for those bacteria associated with faecal contamination under the premise that their presence is suggestive of the presence of human pathogens. Ideal indicator bacteria should have the following traits:

- They originate only in the digestive tract of humans and warm-blooded animals.
- They must be present when faecal contamination is present.
- They must be present in equal or greater numbers than the target pathogenic bacteria.
- Their survival outside the intestine must be longer than the target organism for which it is a surrogate.
- The indicator bacteria must not reproduce outside of the host organism.
- They must be easily, rapidly, reliably and inexpensively identified and enumerated.
- They must not in themselves be pathogenic.

(adapted from EPA, 2001, Metcalf and Eddy, 2003 and Mihelcic, 1999)

As lysimeters were located beneath the percolation area sample analysis for indicator bacteria was carried out, not to ascertain the presence of bacteriological contamination, but to monitor the persistence of pathogenic bacteria in the subsoil.

COLIFORMS

The term "total coliforms" refers to a large group of Gram-negative, rod-shaped bacteria that share several characteristics (Bartram and Ballance, 1996). The intestinal tract of humans contains a large population of these coliform bacteria and each person discharges from 100 to 400 billion of them per day (Metcalf and Eddy 2003). While coliform bacteria are indicator organisms of faecal contamination and possible presence of pathogens there are some genera of the coliform group found in water and soil that grow and reproduce on organic matter outside the digestive system (Viessman and Hammer, 1998). Therefore the presence of total coliforms may or may not indicate faecal contamination and a more conclusive method of analysis is thus required.

The term "faecal coliform" is used in water microbiology to denote coliform organisms that grow at 44 or 44.5°C and ferment lactose to produce acid and gas (Bartram and Ballance, 1996). However, in practice some organisms that display these characteristics may not be of faecal origin giving rise to the more correct term "thermotolerant coliform" which is becoming more common. Nevertheless, the presence of thermotolerant coliforms nearly always indicates faecal contamination with usually more than 95% of thermotolerant coliforms isolated from water being *Escherichia coli* (Bartram and Ballance, 1996). *Escherichia coli* (*E. coli*) is a bacteria species exclusively of faecal origin (Mihelcic, 1999). Some *E. coli* are pathogenic causing diarrhoeal diseases in humans. The presence of *E. coli* in a water supply is proof faecal contamination has occurred and it is therefore a definite indication of the risk that enteric pathogens may be present (APHA, 1998). Where *E. coli* are present in large numbers the inference is that heavy, recent pollution by human or animal wastes has occurred; if the *E. coli* numbers are low it is inferred that the pollution from the same source(s) is either less recent or less severe. If coliforms not including *E. coli* are observed the indication is that either the pollution is recent and non-faecal in origin or of remote, faecal origin such that the intestinal coliforms have not survived (EPA, 2001). The reliability of coliform bacteria to indicate the presence of pathogens in water depends on the persistence of the pathogens relative to coliforms. While the die-off rate for pathogenic bacteria is greater than coliforms outside the intestinal tract of humans viruses, protozoal cysts and helminth eggs are more persistent (Viessman and Hammer, 1998).

FAECAL STREPTOCOCCI

As the normal habitat of faecal streptococci is the gastrointestinal tract of humans and animals their presence in water is also indicative of faecal contamination. While certain streptococci species predominate in some animal species and not in others it is not possible to differentiate the source of faecal contamination based on specification of faecal streptococci (APHA, 1998). As faecal streptococci tend to persist longer in the environment than coliforms and are more resistant to drying it is possible to isolate faecal streptococci from contaminated water that contains few or no coliforms, e.g. where the sampling point is distant in either time or space from the source of contamination (Bartram and Ballance, 1996). It is also useful in clarifying the position of water which shows no *E. coli* but large numbers of coliform bacteria present. The enterococcus group is a subgroup of the faecal streptococci which, despite having some pathogenic properties, is a valuable bacterial indicator for determining the extent of faecal contamination (EPA, 2001 and APHA, 1998). Where in the past a faecal coliform to faecal streptococci ratio greater than 4 was considered indicative of human faecal contamination Standard Methods (APHA, 1998) contends that this ratio cannot be recommended to differentiate between human and animal sources of pollution.

Six sets of bacteriological analyses were carried out at selected sampling points during the project. All samples were analysed by CAL for total coliforms and *E. Coli*, with analysis also carried out for enterococci, faecal streptococci and faecal coliforms on some occasions. The analysis methods used by CAL are contained in Appendix C.

8. ANALYSIS OF RESULTS OBTAINED FROM SITE 1

8.1 Introduction

A successful site assessment was completed for Site 1 on February 4th 2002. However, due to adverse weather conditions and delays resulting from the procurement of construction materials and sampling and analysis equipment, installation of the septic tank and secondary treatment system and the construction of the percolation area was not competed until 30th May, when it was commissioned. Diversion works were then required to separate the domestic wastewater effluent from the roof runoff, which was diverted to a soakaway. The installation of sampling equipment commenced on 21st June and all equipment was in place by 12th July. Sampling began at Site 1 on 9th of August 2002 and continued until 22nd of July 2003.

8.2 Analysis of Flow Data

As outlined in Section 7.2.2 a datalogger and ultrasonic sensor were installed in the sump downstream of the septic tank. Initially an Omega-SL-L320 datalogger and ultrasonic level sensor were used but these were then replaced by a Lascar Easylog datalogger with attached ultrasonic probe as the former was required for flow measurement on Site 2. Flow from the septic tank was measured from 20th July 2002 until 21st August 2003 (Appendix D). Apart from a few brief periods, where the data storage capacity of the datalogger had been reached and records had to be downloaded before flow measurement could resume, it represents a continuous record of flow to the percolation area over this period.

The percolation area was designed and constructed to EPA 2000 specifications, which recommend a combined percolation trench length of 80m based on a loading rate of 25l/m^{2}d and a typical daily hydraulic loading of 180l/person. However, this daily hydraulic loading figure was found to overestimate the measured flows on Site 1. As Table 8.1 highlights, the average flow measured over the research period was

288.3l/d. This was calculated by averaging the sum of the hourly flowrates` measured and multiplying by 24 to give the *total average*. A *daily average* was also calculated which is the average of all the complete daily records of flow. As the datalogger was normally activated during the daytime, and daily flows calculated from midnight, this resulted in incomplete daily records for some of the days on which the datalogger was activated and also for days on which the capacity of the datalogger was reached. These data, while used in the total average calculation, were excluded from the calculation of the daily per capita average which was calculated as 56.3l/pd. When the flow data was adjusted to take into account holiday periods, i.e. when the house was vacant, it can be seen that both the total and daily averages increase as does the minimum flow measured. This adjusted daily per capita average, which is essentially the average daily wastewater generation, equated to 59l/pd.

	Flows (l/d)			
	Max.	**Min.**	**Total Average**	**Daily Average**
Measured Flow	642.5	0.0	288.3	281.7
Adjusted Flow	642.5	93.0	300.9	294.9

Table 8.1 Flows measured on Site 1 (Adjusted Flow takes account of holiday periods).

While design of the percolation area must account for the maximum projected hydraulic load it was found that, even when the greatest flow measured (642.5l/d) was taken into consideration, the percolation area was hydraulically over-designed with respect to the EPA (2000) specifications. It must also be considered that in the current socio-economic climate it is not uncommon for both of a cohabiting couple to work outside the home. As the holdings on Site 1 contained an office, from which the husband ran his business, it is possible that these measured flows are greater than average.

While Table 8.1 gives an overview of flow behaviour on Site 1 it is important to also consider the frequency distribution of the recorded flowrates. As can be seen from Figure 8.1, the maximum flow of 642.5l/d was an isolated event and only 7% of flows recorded were greater than 440l/d. The majority of flows recorded, or 84%, were in the range 140l/d to 440l/d and when figures were adjusted to take the holiday periods into account this rose to 87% (Figure 8.2). The daily average flow recorded for this period was 281.9l/d and this increased to 284.1l/d when the holiday periods were accounted for.

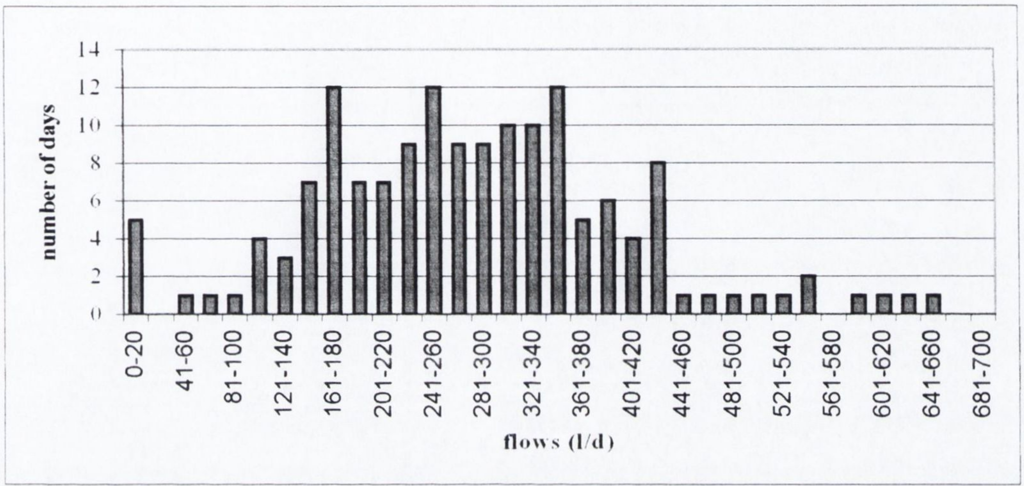


Figure 8.1 Frequency distribution of flows recorded on Site 1.

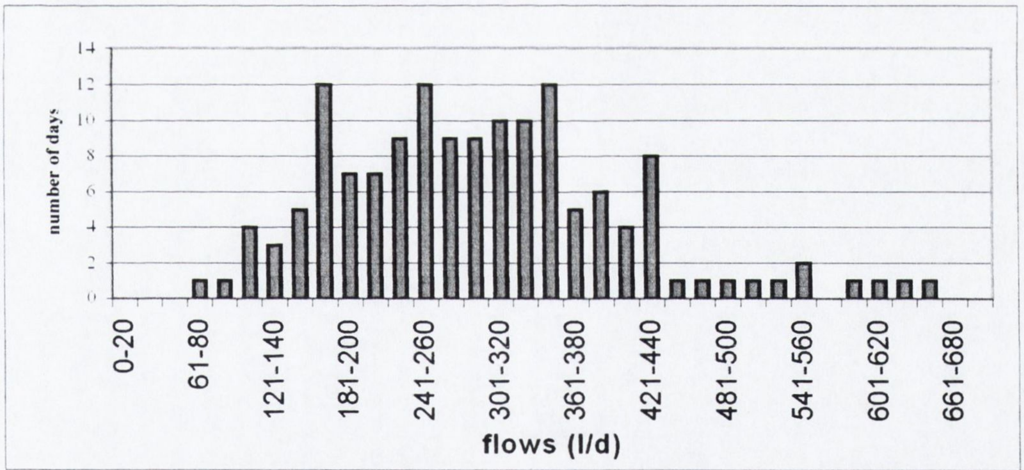


Figure 8.2 Frequency distribution of flows recorded on Site 1 adjusted to take account of holiday periods.

To attain an even distribution between the four percolation trenches the design of the distribution box had to take account of the influent flow regime. Analysis of the flow emanating from the septic tank on Site 1 showed that the average flow rate was less than 0.8l/min for 82% of the monitoring period (Figure 8.3). However, it must be considered that the installation of the Puraflo® system dampened the flow characteristics of the STE arriving at the distribution box.

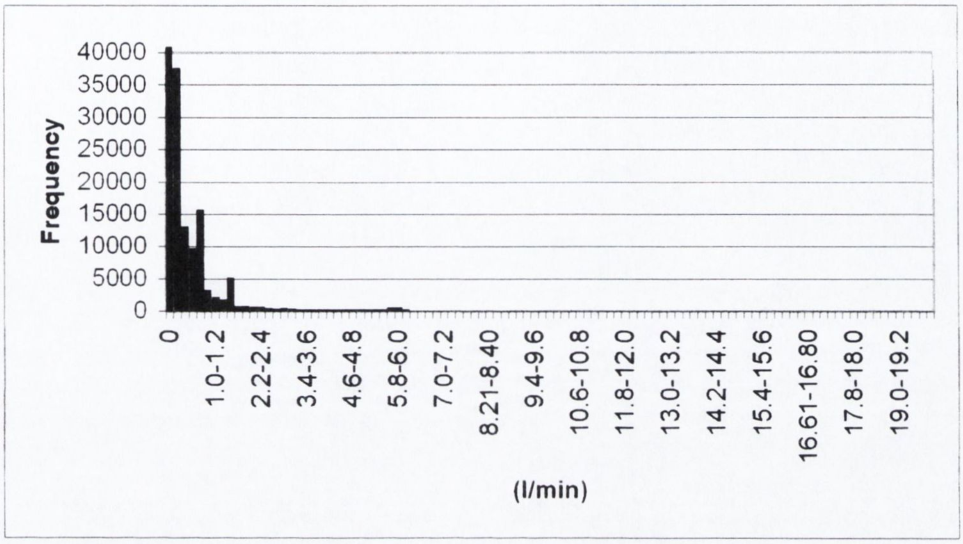


Figure 8.3 Frequency distribution (l/min) of flows recorded at Site 1.

To ensure that the separation work was successful and that the measured flow data accurately reflected the domestic wastewater effluent generation, and was not influenced by surface water intrusion, flowrates were graphed against the rainfall recorded on that day. It appears from Figure 8.4 that there was no correlation between rainfall and septic tank effluent flowrate and hence surface water separation works had been successful. This was corroborated when the surface water shores were examined for connectivity with the septic tank revealing no connection.

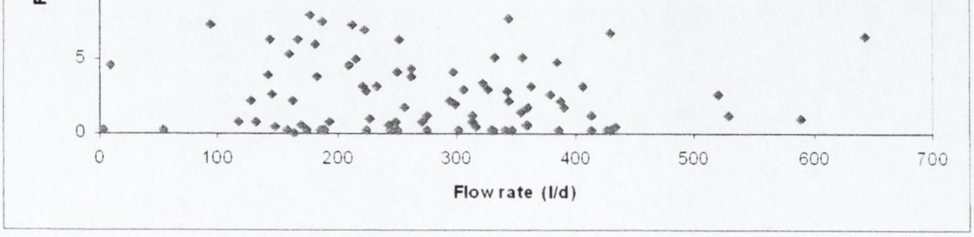


Figure 8.4 Graph of flowrate against rainfall for Site 1.

8.3 Results of the Analysis of Septic Tank and Secondary Effluent

8.3.1 Results of Chemical Analysis

Appendix E contains the results of the chemical analysis carried out on all samples taken from Site 1. Table 8.2, which summarises the results relating to the STE, shows high concentrations of organics and nutrients which, as outlined in Section 2.2.3, are characteristic of STE. The anaerobic environment of the septic tank facilitated the breakdown of organic nitrogen and phosphorus to their inorganic forms, ammonium and orthophosphate respectively, which were the prevalent nutrient forms in the STE. It was also responsible for the low concentration of NO_3 and NO_2. The NH_4, NO_2 and NO_3, concentration in the STE remained stable throughout the year. As inorganic nitrogen is more likely to exist in its reduced (NH_4-N) or oxidised (NO_3-N) form in the subsoil (Brady and Weil, 2002), NO_2-N concentrations were measured to enable the calculation of total inorganic nitrogen concentration in the percolating effluent rather than as an indicator of effluent attenuation.

	Concentration (mg/l)					
	COD	NH_4-N	NO_2-N	NO_3-N	PO_4-P	Cl
Maximum	1630.0	161.5	0.42	5.3	61.9	290.0
Minimum	484.0	98.8	0.00	0.0	19.3	74.1
Average	**791.6**	**131.0**	**0.28**	**1.4**	**32.3**	**117.8**

Table 8.2 Summary of chemical analysis of STE on Site 1.

There was little variation in the COD concentration until the 29/05/03 from where it increased to an annual high of 1630mg/l on the 25/06/03 before it dropped to 1329mg/l, which was recorded on the final day of testing (Figure 8.7). There was no corresponding change in the daily routine over this period to explain this increased organic load and it is therefore possible that this increase in COD concentration was due to increased biological activity within the septic tank. Anaerobic degradation of organic matter within the septic tank is a slow process and it is also possible, therefore, that it only began to have a significant impact on STE COD concentration near the end of the project as the septic tank started empty at the beginning. Apart from two highs of 61.90mg/l and 55.60mg/l on the 26/09/02 and 24/10/02 the orthophosphate concentration showed little variation throughout the year. The chloride (Cl) value of 290mg/l measured on the 15/08/02 was 2.3 times greater than the next highest Cl measurement and was therefore dismissed as egregious and omitted from further calculations (Figure 8.9). As the Cl values for all samples on this date are unusually high when compared to values recorded over the remainder of the sample period it is possible that these rogue values arose due to an error in analytical procedures such as the failure to calibrate the spectrometer. It is interesting to note that, overall, the concentrations of the chemical parameters measured in the STE at Site 1 were found to be higher than those described in Tables 2.2 and 2.3.

The aerated environment of the secondary treatment system promoted nitrification and the aerobic degradation of organic matter which, combined with the physical straining action of the bio-media, resulted in an effluent with a lower organic and higher nitrate concentration (Table 8.3). The aerobic and facutatively aerobic bacteria are found largely in the upper portion of the filter media with nitrifers becoming more prevalent with depth (>30cm) (Henry, 1995). Therefore, the degradation and assimilation of the carbonaceous element of the waste is effected within the upper portions of the filter media with nitrification occurring at greater depths provided that sufficient oxygen is available. It appears, from Figure 8.5, that there was no fall in nitrification over the winter months.

	Concentration (mg/l)					
	COD	NH_4-N	NO_2-N	NO_3-N	PO_4-P	Cl
Maximum	316.0	45.2	13.6	60.5	85.6	185.0
Minimum	98.0	6.9	0.8	15.3	16.8	51.3
Average	**188.1**	**19.7**	**7.4**	**36.9**	**33.6**	**92.6**

Table 8.3 Summary of the results of chemical analysis of SE on Site 1.

While Table 8.4 and Figure 8.5 show the effects of partial nitrification, highlighted by the drop in pH, within the secondary treatment system on effluent quality they do not highlight the reduction in overall nitrogen loading achieved by the secondary treatment over the research period as seen in Figure 8.6. It was found, on average, that there was a 51.6% reduction in inorganic N loading across the secondary

	% of total inorganic N			Average
	NH_4-N	NO_2-N	NO_3-N	pH
Septic Tank Effluent	98.9	0.2	0.9	7.9
Secondary System Effluent	31.2	11.1	57.8	6.4

Table 8.4 The average breakdown of inorganic N in STE and secondary effluent.

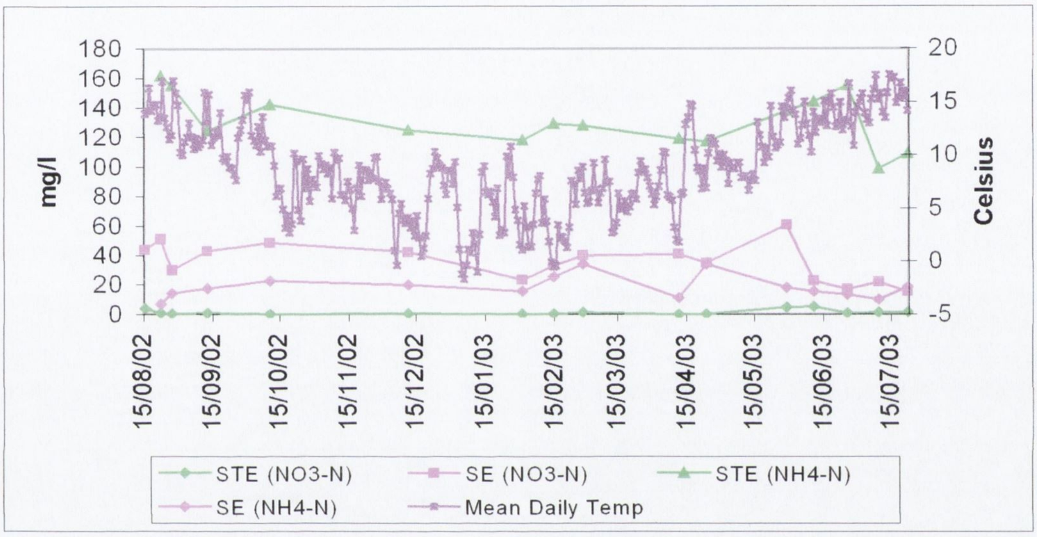


Figure 8.5 The effect of nitrification on secondary system effluent quality: (a) NH_4-N concentration and (b) NO_3-N concentration.

Figure 8.6 Reduction in Total inorganic N concentration across the secondary treatment system.

treatment system which must be due to denitrification within the peat module. This takes into account any subsequent mineralisation of organic nitrogen that might occur within the secondary system. It is probable that modifications to the design of the modules containing the peat medium created anoxic conditions in the base of the module. The treatment modules are usually placed on a gravel bed and the effluent allowed to percolate through perforations in the base into the gravel distribution layer from where it enters the subsoil. However, the modules had to be adapted for this project to enable the effluent to be evenly distributed between the percolation trenches. To this end the base of the treatment modules was not perforated and the effluent was gravity fed to a collection chamber, by pipes inserted 5cm above the base of modules' side-walls, from where it entered the distribution box. It is likely, therefore, that slightly flooded conditions along the base of the module promoted denitrification. An Institute for Industrial Research and Standards (IIRS) report produced in 1988 at the behest of Bord na Móna confirmed the occurrence of denitrification in flooded modules.

The installation of the secondary system on Site 1 reduced the organic load on the percolation area by an average of 76.2% (Figure 8.7). While the increase in COD concentration in the STE over the last two months of the analysis is reflected by an increase in the COD concentration in the secondary effluent (SE), the relative

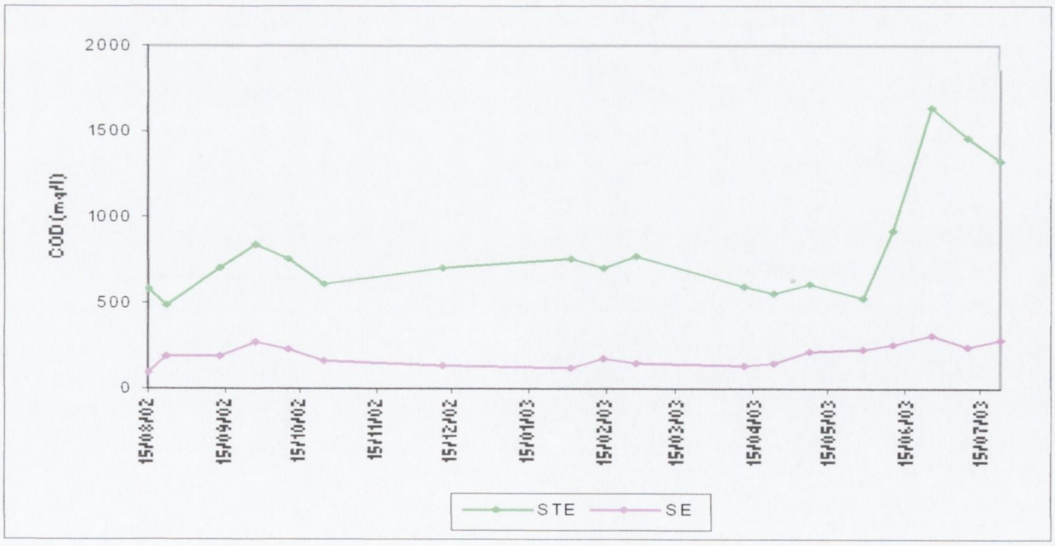


Figure 8.7 Comparison of COD concentration in septic tank and secondary system effluent.

magnitude of this increase is much smaller. This suggests an ability by the secondary treatment system to treat influent of varying organic quality while maintaining a relatively constant effluent concentration. This increase in STE COD concentration appears to coincide with a period of increased denitrification within the Puraflo® module (Figure 8.6) suggesting that the increased organic load allied with the saturated conditions has promoted further denitrification.

While phosphate removal within the Puraflo® is due to biological activity and is generally in the order of 10-15% (Paul Bolger, Bord na Móna – personal communication 2004) it was found that orthophosphate concentrations in the septic tank and secondary effluent from Site 1 were similar throughout the research period. However, there were incidents where the SE orthophosphate concentration was greater than that of the STE (Figure 8.8). This can be explained due to the fact that while biological phosphate removal of the order of 10 to 15% is achieved across the Puraflo®, 80-90% of organic phosphate in the STE is also mineralised (Paul Bolger, Bord na Móna). It is possible, therefore, that the orthophosphate concentration of the

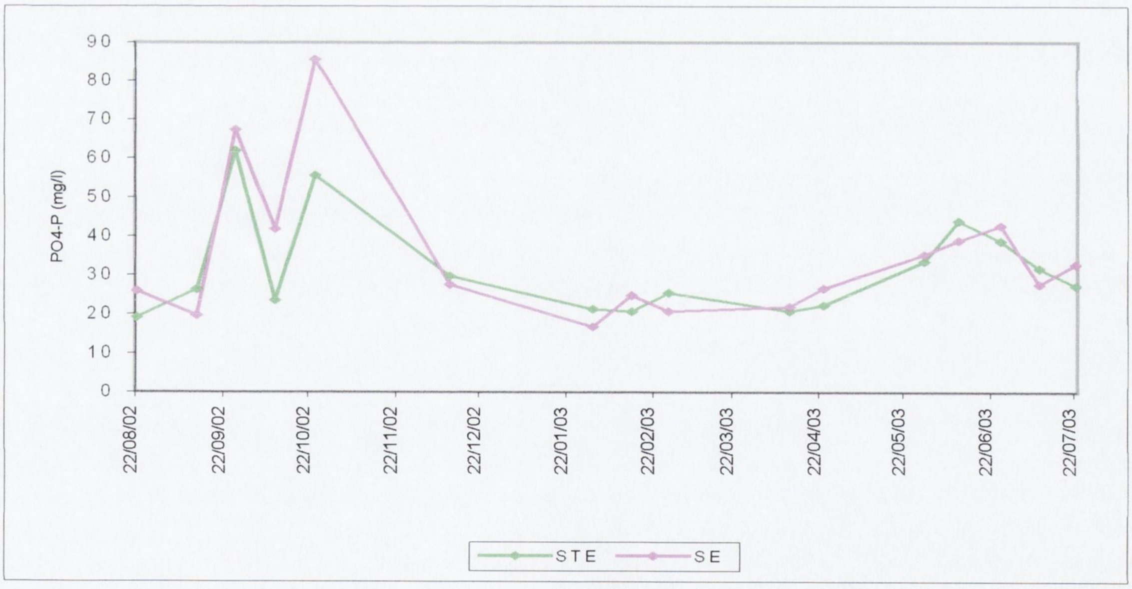


Figure 8.8 Comparison of orthophosphate concentration in STE and SE.

SE could be greater that that of the STE. It must also be considered that the analysis procedure for orthophosphate required the dilution of samples with distilled water as their concentration was outside the detectable limit of 5 mg/l. As a 1:5, 1:10 or 1:20

dilution was used this meant that a small error in the analysis of the diluted sample would result in a 5, 10 or 20 fold increase in any error in the reported concentration.

Taking into account the rogue value for Cl concentration on 15/08/02 it can be seen that, on average, there is an 18.2% difference in concentration between the STE and the SE (Figure 8.9). As the secondary treatment system is sealed, preventing intrusion by precipitation, allied with the fact that Cl does not take part significantly in any reactions within the peat medium, it is probable that the reduction in Cl concentration across the secondary treatment system is due to the removal of colloidal matter by the physical straining action of the peat fibres and associated biofilm. While STE samples analysed for Cl were not filtered during sample analysis for Sites 1 and 2 they were for Sites 3 and 4. It was found that, for 18 sets of STE samples analysed, there was on average a 22.6% reduction in the Cl concentration between unfiltered and filtered samples (Section 11.3.3). This would suggest that the difference between unfiltered samples of STE and SE was due to the removal of particulate matter within the Puraflo®. This would therefore account for the reduction in Cl concentration between the STE and SE recorded on Site 1.

Figure 8.9 Comparison of Cl concentration in STE and SE.

Based on the chemical analysis alone it is clear that, while the installation of a secondary treatment system downstream of the septic tank reduces the organic and nitrogen load of the domestic wastewater effluent, further treatment is required prior to its discharge to groundwater.

8.3.2 Results of Bacteriological Analysis

While the project's main aim was the assessment of the attenuation capacity of subsoil receiving domestic wastewater, and was therefore mostly concerned with the quality of the distribution box influent, some bacteriological analyses was carried out on STE. The results of all bacteriological analyses are contained in Appendix E. Samples were not analysed for the presence of viruses as this was beyond the scope of the project. The installation of a secondary treatment system on the site greatly reduced the bacterial loading on the percolation area as highlighted by the six results shown in Table 8.5.

Date	Bacteria	Effluent Concentration (cfu/100ml)		% Removal
		Septic Tank	Puraflo® System	
15/05/03	*E. coli*	397,260	4,320	98.91%
15/05/03	Faecal coliforms	>486,840	17,200	N/A
28/08/03	*E. coli*	1,416,600	5,040	99.99%
28/08/03	*Enterococci*	238,200	600	99.99%
28/08/03	Faecal coliforms	1,553,100	6,010	99.99%
28/8/03	Total coliform	>2,419,200	130,000	N/A

Table 8.5 Example of the reduction in bacteria concentration of domestic wastewater effluent resulting from the installation of a Puraflo® system on Site 1.

However, even when the removal efficiency of the secondary treatment system is taken into account, the high concentrations of bacteria measured in the SE deem it unsuitable for discharge to groundwater prior to further bacterial removal in the subsoil (Table 8.6).

Date	Effluent Concentration (cfu/100ml)				
	Total coliforms	E. coli	Faecal Coliforms	Enterococci	Faecal Streptococci
17/10/02	3,000,000	200,000			620
13/03/03	203,000	31,000			63,000
15/05/03	77,460	4,320			17,200
28/08/03	130,000	5,040	6,010	600	

Table 8.6 Concentrations of bacteria measured in SE on 4 separate dates.

8.4 Results of Analysis of Soil Moisture Samples

8.4.1 Method of Analysis

As can be seen from Table 7.2 it was not possible to install all identically coloured lysimeters to similar depths and therefore, prior to analysis of the chemical and bacteriological results, reclassification of some lysimeters was required. To achieve this, three nominal depths of 0.3m (red), 0.6m (blue) and 1.0m (black) below the invert of the percolation trench, referred to as depth planes and colour coded to conform with the lysimeter colour code (i.e. it had been desired to install red lysimeters to the red depth plane), were defined. The subsoil below the percolation trench was divided into three sections: 0 to 0.4m, 0.4 to 0.8m and 0.8 to 1.2m and the depth planes were chosen to represent the middle of these sections. However, as no lysimeter tip was installed to a depth less than 0.2m below the invert of the percolation trench, and as the 0.2 to 1.2m section of the soil profile below the percolation trench had been similarly classified (Table 5.6), a depth of 0.3m, rather than 0.2m, was decided to be more representative of the red depth plane. Lysimeters not installed to the correct depth, viz a viz their colour code, were then reclassified taking into account the location of their tip within the subsoil. For example, the red lysimeter located to a depth of 0.49m at the 0m sample position on trench 3 was reclassified as a blue lysimeter (Table 7.2).

During the research period it was only possible to obtain 4 samples from the 1.0m depth plane at the 0m sample position. However, as only a depth of 0.6m of unsaturated subsoil is required below the invert of the percolation trench on a site deemed suitable to receive SE (EPA, 2000) the absence of a complete set of samples from the lower plane was not of concern.

As Cl does not take a significant part in any geochemical reactions (Marshall *et al.*, 1999) the results of the soil moisture sample analyses for Cl were used to identify differences in loading rates within the percolation area and thus determine the most representative method of reporting the attenuation of the percolating effluent. As is the case with the presentation of all the results of chemical and bacteriological analysis of soil moisture samples, this method assumes homogeneous and isotropic subsoil properties and only takes account of matrix flow.

The results of the laboratory analysis for Cl at the three sample positions along each trench, i.e. 0m, 10m and 20m, were averaged over the depth plane on which they were recorded. This was then plotted over the research period to identify which of the following methods best represented the distribution of SE within the percolation area:

(i) Planar Average: This method involved the averaging, over the four trenches, of the concentrations of each parameter over the depth plane on which it was measured and comparing the difference between average loading rates calculated for the different planes that is, 0.3m, 0.6m and 1.0m.

(ii) Depth Average: The average concentration, over the four trenches, of each parameter within each plane was calculated at the three different sample distances along the length of the trenches and the corresponding differences in concentration between the planes compared.

When the average Cl concentrations measured for each of the sample positions were plotted it was found that the concentration at the 0m sample position, at all depths, was, on average, 8 to 9 times greater than the concentration measured at the other sample positions (Figure 8.10). When the average Cl concentration for each depth plane at the 0m sample position was then plotted it was discovered that they were very similar (Figure 8.11). This would suggest that the SE effluent was only reaching the 0m sample position and that the reduction in the organic load of the STE brought about by the installation of the secondary treatment system inhibited the formation of a biomat along the base of the percolation trench, preventing dispersion of the SE along the entire trench length and thus confining loading to less than the first 10m of the trenches. It would also suggest that as the Cl concentrations for the two depth planes at 0m are very similar, the influence of dilution on effluent attenuation between these planes was small (Section 8.4.2). This pattern was maintained throughout highlighting the fact that the SE, and hence the biomat, had not progressed 10m along any trench.

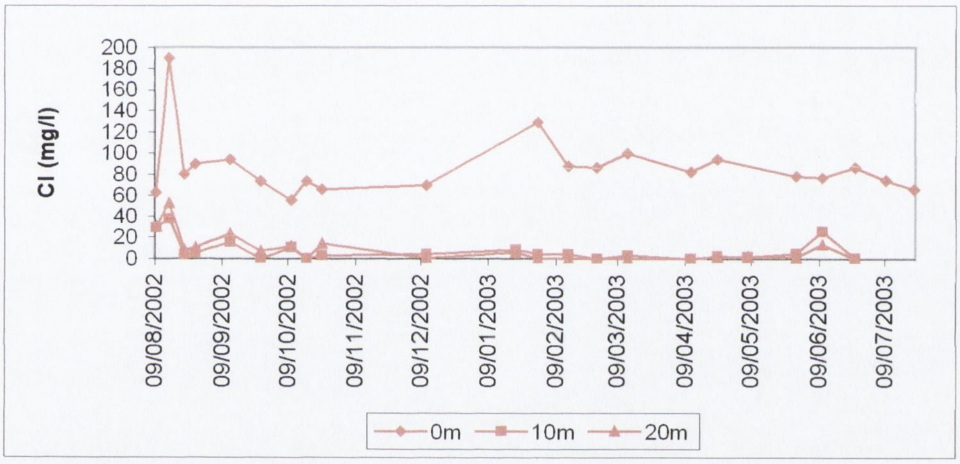


Figure 8.10 Cl concentration on the red depth plane at the 3 sample positions.

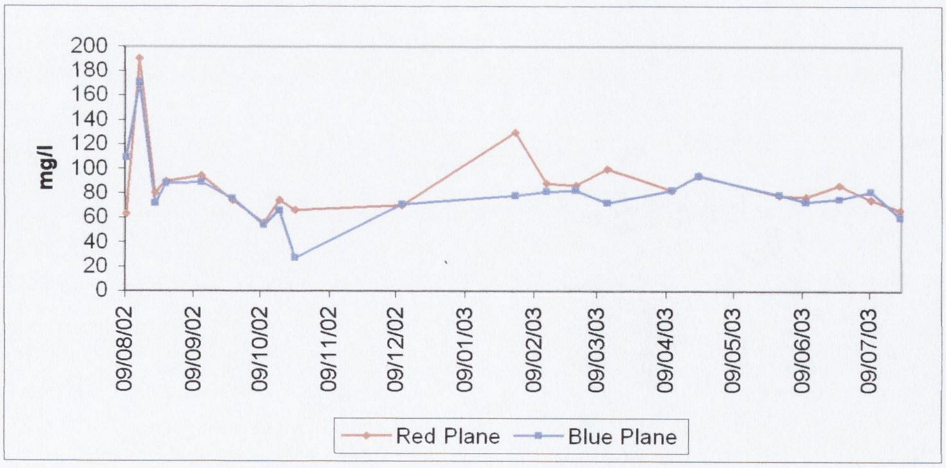


Figure 8.11 Cl concentration measured on the two depth planes at the 0m sample position.

A tracer study, which commenced on the 29th July 2003, was carried out over an eight-day period to validate the findings of the Cl analysis. Sampling was carried out on three consecutive days and then there was a break of three days before the lysimeters were put under suction again to obtain the final sample. An aqueous solution of potassium bromide (KBr) was poured into each outlet pipe in the distribution box. Bromide (Br), like Cl, is an ideal tracer due to its negative charge and has been widely used as a tracer to investigate water and contaminant transport in agricultural research scenarios (Smith & Davis, 1974; Flury & Papritz, 1993; Jabro *et al.*, 1994; Kessavalou *et al.*, 1996; Schuh *et al.*, 1997; Kelly & Pomes, 1998 and Richards 1999). It has an advantage over Cl of naturally low background concentrations in groundwater [<0.01 – 0.3g $\mathbf{Br^-}$ m^{-3}] (Flury & Papritz, 1993) whereas chloride is abundant in the natural environment. The results of the analysis of the soil moisture samples for Br showed that, over an 8-day period, Br was recorded in all the lysimeters at the 0m sample position, except the blue lysimeter on trench 2 (Appendix F). It is possible, however, that Br arrived at the blue lysimeter on trench 2 on the days when sampling was not carried out. As can be seen from Table 8.7 the time of travel of the tracer from the distribution box to the individual sample points on the same depth plane varied thus highlighting the inherent anisotropy and inhomogeneity of the subsoil. No tracer was sampled at any point at either the 10m or 20m sample positions. It is possible that the presence of Br in samples on day 8 was not the first incidence of Br at these sample points and that it could have arrived on day four when no sampling took place.

Lysimeter	Trench Number			
	1	2	3	4
Red	Day 2	Day 8	Day 3*	Day 8
Blue	Day 2	None	Day 8**	Day 2
Black	Day 2*	Broken	Day 3	Day 2*

*** denotes lysimeter reclassified as blue; ** denotes lysimeter reclassified as black.**

Table 8.7 Time of first arrival of Br at the 0m sample position on Site 1.

8.4.2 The Effect of Dilution on Effluent Attenuation

While physical, chemical and biological processes all play an important part in the attenuation of the percolating effluent, the effects of dilution by recharge must also be considered. The rainfall available for dilution at the depth planes over the project duration, or effective rainfall, was calculated using rainfall figures obtained on site and evapotranspiration figures calculated using data from a meteorological station at Pollardstown Fen, 14km from Site 1 (Appendix G). A model developed by Bartley (2004) based on the FAO Penman-Monteith method (FAO, 1998) of potential evapotranspiration (PET) calculation was used to calculate actual evapotranspiration (AET) (Appendix G). Where soil moisture deficit (SMD) was greater than 40mm the AET was considered to occur at a slower rate than PET and was therefore calculated using the Aslyng scale:

$$AET = PET\,(120 - SMD^*)/(120 - 40)\ \text{mm}$$

where,

SMD* is the accumulated soil moisture deficit at the beginning of each period and where SMD* > 40 it must be updated to SMD.

$SMD = SMD^* + (AET - RF)$

RF = total rainfall (mm) for the period

(Keane, 2001)

Daily effective rainfall was then calculated by subtracting the daily AET and accumulated SMD figures from the daily rainfall measurement (mm).

As the project commenced in August, when a SMD generally exists, it was necessary to determine this SMD at the start of the project to enable the calculation of effective rainfall over the project duration. As SMD is an accumulative number it was necessary to use historical meteorological data from the weather station at Pollardstown Fen to calculate the SMD at the start of the project. Ideally SMD calculations should start in January, when there is generally no SMD, however due to technical problems experienced at the weather station in early 2002 records only began in March. As meteorological data obtained from the Met Éireann weather station at Casement Aerodrome shows no SMD for the months of January and February it was therefore assumed that the SMD at the beginning of March was at a minimum and any error would be rectified in subsequent calculations where effective rainfall exceeded SMD. Using this method it was found that, for a recorded rainfall of 950.0mm for the period 01/08/02 to 22/07/03, the effective rainfall was 419.3mm.

Potential evapotranspiration was also calculated using the Hargreaves method (Equation 8.1) which is applicable where only limited meteorological data are available (FAO, 1998). In the calculation procedures for R_a, the latitude is expressed in radian (i.e., decimal degrees times π /180).

$$PET = 0.0023(T_{mean}+17.8)(T_{max}-T_{min})^{0.5}R_a \qquad \text{(Equation 8.1)}$$

where,

R_a = $187.013G_{sc}d_r(\omega_s\sin(\varphi)\sin(\delta)+\cos(\varphi)\cos(\delta)\sin(\omega_s))$ [mm]

G_{sc} = solar constant = 0.0820 [$MJm^{-2}min^{-1}$]

d_r = inverse relative distance Earth-Sun

ω_s = sunset hour angle [rad]

φ = latitude [rad]

δ = solar declination [rad]

(FAO, 1998)

Using this method to calculate the daily effective rainfall it was found that the effective rainfall for the period 01/08/02 to 22/07/03 was 407.9mm. It can be seen from Figures 8.12 and 8.13 that the effect of dilution, i.e. the difference between the

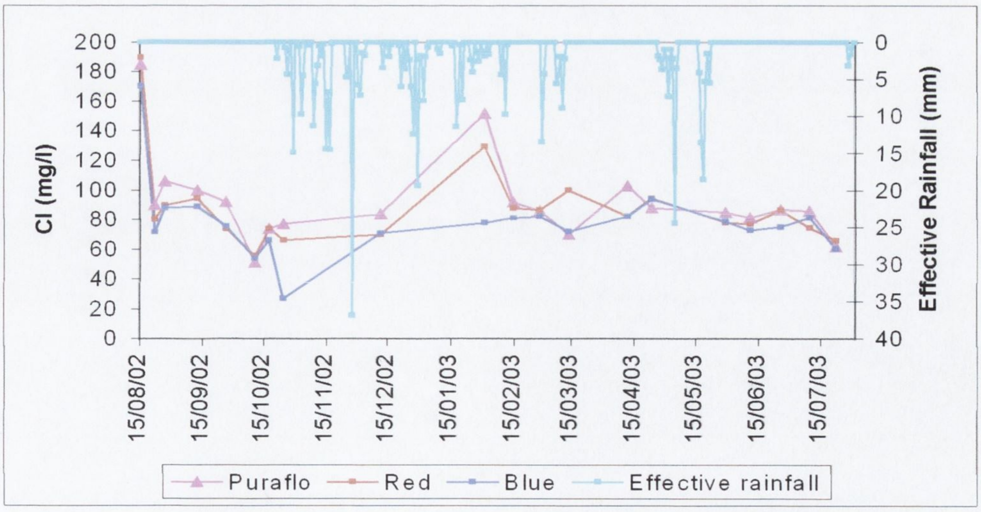


Figure 8.12 The effect of dilution by effective rainfall, calculated by the Penman-Monteith method, on Cl concentration.

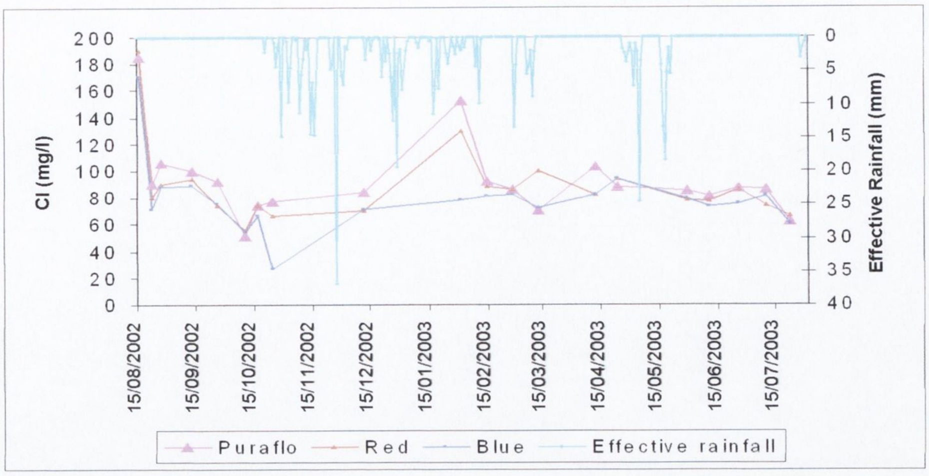


Figure 8.13 The effect of dilution by effective rainfall, calculated by the Hargreaves method, on Cl concentration.

Puraflo® and soil moisture Cl concentrations, was greatest during the period of sustained effective rainfall between November 2002 and March 2003. As can be seen from Figures 8.12, 8.13 and 8.14 simultaneous sampling resulted in occasions where Cl concentrations in the lysimeter samples were greater than those measured in the SE due to time lapse, i.e. the time taken for SE to travel from the Puraflo® to the lysimeters.

The reduction in Cl concentration between the Puraflo® and both the red and blue depth planes was examined as another method of quantifying this dilution effect and calculating the zone of contribution around each trench (Table 8.8). As can be seen from Figure 8.14 Cl concentrations on the blue depth plane on the 24/10/02 and 31/01/03 do not correspond as closely to Cl concentrations in the SE and red depth plane as Cl concentrations on other dates do. When the Cl concentrations for these dates were omitted from dilution calculations it was found that the effect of dilution was equivalent to, on average, the addition of 0.13 litres (or 10.63% reduction in concentration) and 0.15 litres (or 13.72% reduction in concentration) of effective rainfall per litre of effluent on the red and blue depth planes, respectively. With an average concentration of Cl in rainfall at Site 1 of 4.8mg/l (see Section 7.3.5 for method) and using the daily average flow of 281.7l/d (Table 8.1) this equates to an average daily effective rainfall contribution of 36.6 litres across the red plane and 42.3 litres across the blue plane. When the average effective rainfall contribution for

	% reduction in concentration		Equivalent contribution of effective rainfall (l)	
Date	Red Plane	Blue Plane	Red Plane	Blue Plane
15/08/02	0.00	7.89	0.00	0.09
22/08/02	10.74	20.67	0.13	0.28
27/08/02	15.41	16.98	0.19	0.22
12/09/02	5.67	11.20	0.06	0.13
26/09/02	19.57	17.75	0.26	0.23
17/10/02	0.00	10.81	0.00	0.13
24/10/02	14.29	N/A	0.18	N/A
11/12/02	16.67	15.48	0.21	0.20
31/01/03	14.69	N/A	0.18	N/A
14/02/03	4.35	11.59	0.05	0.14
27/02/03	0.00	4.26	0.00	0.05
11/04/03	19.90	20.06	0.26	0.27
29/05/03	8.05	7.40	0.09	0.09
10/06/03	5.52	10.57	0.06	0.13
25/06/03	0.57	13.41	0.01	0.17
09/07/03	13.37	5.23	0.16	0.06
22/07 /03	0.00	2.42	0.00	0.03
Average	**10.63**	**13.72**	**0.13**	**0.15**

Table 8.8 Calculation of the contribution of effective rainfall to dilution.

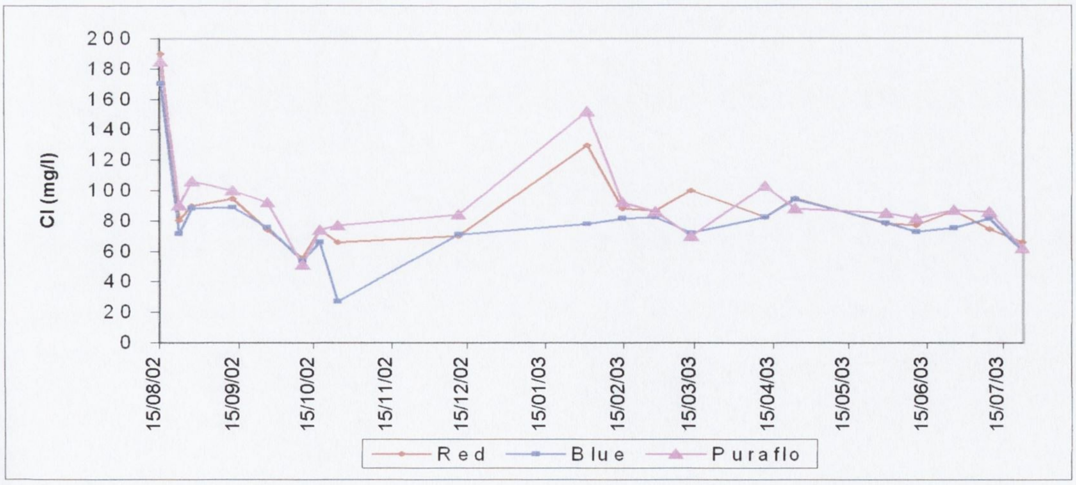


Figure 8.14 Comparison of Cl concentrations as an indicator of effluent dilution.

the red plane is calculated for the year it is found to be 13.4m^3. By dividing this by the effective rainfall calculated by the Penman- Monteith method, it was possible to estimate that the zone of contribution of effective rainfall was approximately 32.0m^2. As highlighted in Section 8.4.1 less than 10m of percolation trench was utilised. Where data are available for the smaller lysimeters installed below the invert of trenches three and four it can be seen that, while effluent was being measured at the 2m sample point, it was not present at the 7m sample point (Appendix E). Therefore, taking the trench width of 0.45m and an average trench length of 4m this equates to a zone of contribution of effective rainfall of approximately 0.5m on all sides of each trench. When the same calculation was carried out for the blue plane it was found that the zone of contribution of effective rainfall was slightly greater at 36.8m^2. This would be due to the dispersion of the effluent plume below each percolation trench.

Examination of the soil moisture tension values from the tensiometers installed at the 0m and 20m sample positions on Site 1 also suggest that it is physical, chemical and biological processes rather than dilution that are the more prominent attenuation processes operating in the subsoil. As can be seen by Figure 8.15 tensiometers at the 20m sample position, where no effluent was recorded, react to the variation in effective rainfall over the sampling period. If Figure 8.16 is examined, however, it can be seen that the tensiometer readings are more affected by the percolating effluent rather than the contribution of effective rainfall suggesting that the contribution of dilution to effluent attenuation is small.

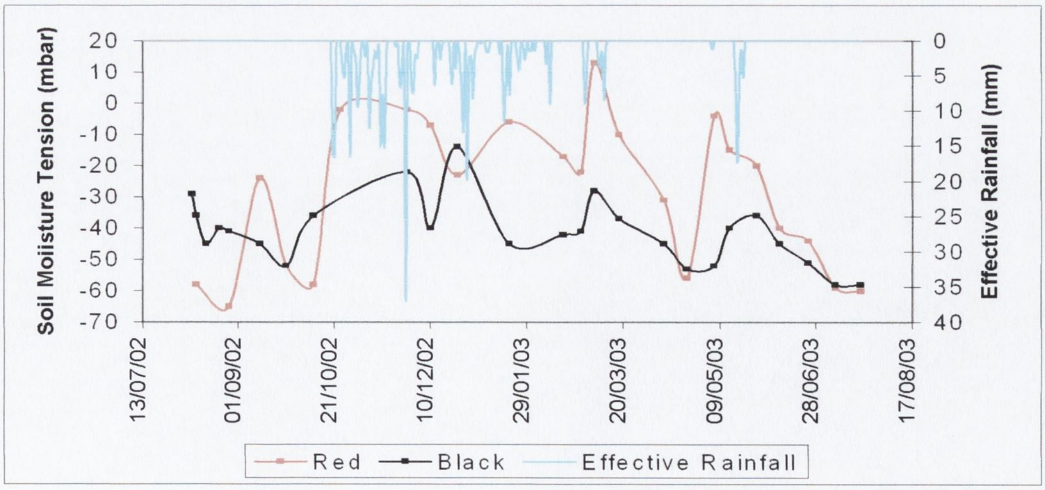


Figure 8.15 Soil moisture tension plotted against effective rainfall for the 20m sample position on Site 1.

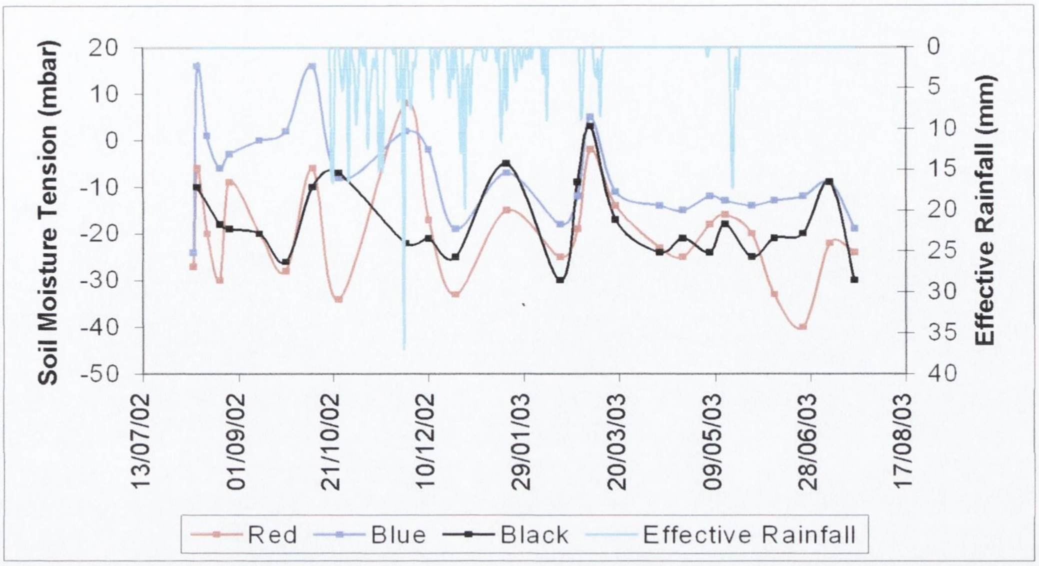


Figure 8.16 Soil moisture tension plotted against effective rainfall for the 0m sample position on Site 1.

8.4.3 Results of Chemical Analysis

While parameter concentrations were measured in the laboratory it was decided that results should also be reported as loading rates in an attempt to quantify the load of a particular parameter in the percolating effluent. To this end the concentrations (mg/l) were multiplied by the daily average flow (l/d), for SE and STE concentrations, or the sum of the daily average flow and the contribution of effective rainfall, for soil moisture samples to give a loading rate (g/d).

The reduction in COD concentration and load of the percolating effluent with subsoil depth (Table 8.9) is small when compared to the reduction that takes place across the secondary treatment system. While this reduction in COD within the subsoil results from aeration, physical straining and biological degradation, the reduced organic load of the SE failed to significantly stimulate bacterial activity along the subsoil-effluent interface to promote biomat formation along the entire base and side-walls of the percolation trenches. This has had the effect of concentrating the effluent on less than half the percolation area thereby reducing the effects of other attenuation processes such as dilution, dispersion and advection within the unsaturated subsoil. Similarly the potential bacteriological and chemical load on groundwater would be concentrated over a smaller area thus affecting attenuation processes within the saturated zone which could lead to elevated contaminant concentrations in the groundwater and therefore have health implications for adjacent water supply sources.

Sample Position	Concentration (mg/l)	Load (g/d)	Load Removal (g/d)
STE	791.6	223.0	
SE	188.1	55.5	167.5
Red Depth Plane	107.5	30.9	24.6
Blue Depth Plane	76.2	21.9	9.0

Table 8.9 Reduction in COD load attributed to the specific treatment steps.

The effluent from the secondary treatment underwent further slight nitrification within the subsoil as can be seen by Table 8.10 which shows the average concentrations and loading rates of NH_4-N, NO_2-N, and NO_3-N measured on the red and blue depth planes over the project duration. While near complete nitrification has occurred by the blue depth plane there is little change in the total inorganic N concentration with

Depth Plane	NH_4-N		NO_2-N		NO_3-N		Total N	
	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)
SE	19.7	5.5	7.4	2.1	36.9	10.4	64.0	18.0
Red	5.8	1.7	3.3	0.9	52.0	14.9	61.1	17.5
Blue	1.5	0.4	0.6	0.2	56.2	16.1	58.3	16.7

Table 8.10 Average concentration and loading rate of NH_4-N, NO_2-N, NO_3-N and Total inorganic N measured on the red and blue depth planes.

depth throughout the research period (Figures 8.17, 8.18 and 8.19). As outlined in Section 4.3.3 inorganic nitrogen, in the form of both NO_3-N and NH_4-N, can be removed from the percolating effluent by immobilisation and/or denitrification. However, the inhibition to biomat formation along the percolation trench base, resulting from the reduction in organic load attributed to secondary treatment, would

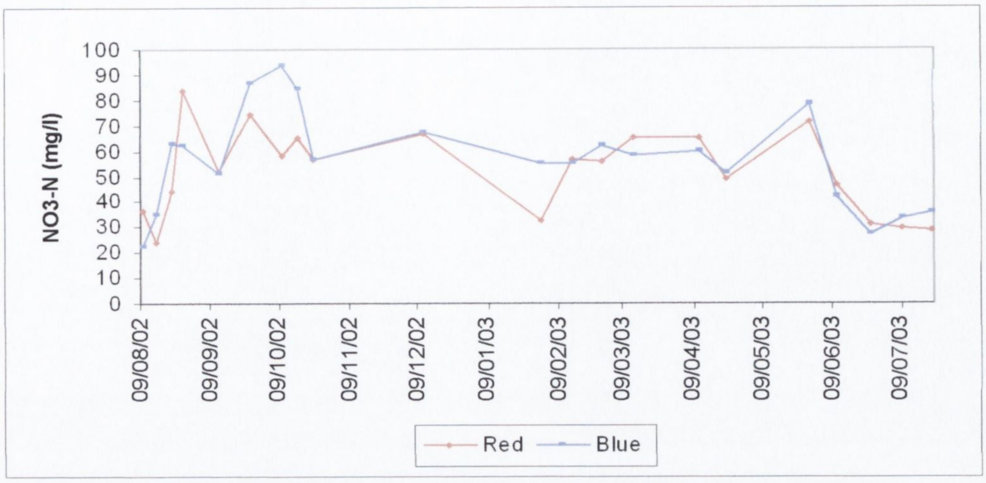


Figure 8.17 Average NO_3-N concentrations at the 0m sample position.

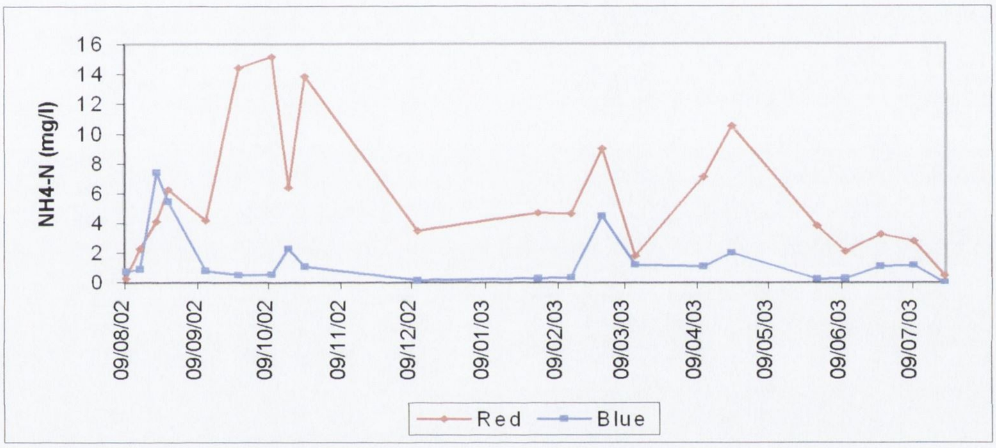


Figure 8.18 Average NH_4-N concentrations at the 0m sample position.

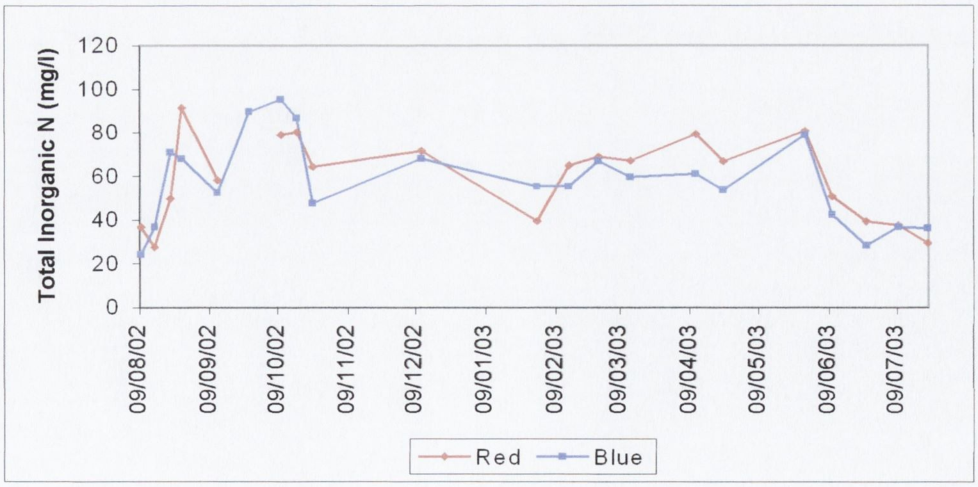


Figure 8.19 Average total inorganic N concentrations at the 0m sample position.

result in a reduction in microbial activity thereby reducing demand for inorganic nitrogen in the decomposition of organic matter. The absence of a biomat along the trench base would also have promoted unsaturated conditions. Bacteria responsible for denitrification are facultative anaerobic heterotrophs and as such require, not only saturated subsoil conditions, but also a supply of organic material from which to obtain their energy and carbon for metabolism. While unsaturated conditions appeared to be prevalent in the subsoil directly below the percolation trenches there were isolated incidents of saturation (Figure 8.16). Where these occurred it is possible that the organic load was insufficient to promote significant denitrification.

Removal of phosphorus from the percolating effluent is controlled by soil adsorption and mineral precipitation. As outlined in Section 4.2.2 the ability of a soil to fix phosphorus is dependent, not only on its clay content, but also on the presence of Al, Fe, or Mn in acidic soils, either as dissolved ions, as oxides or as hydrous oxides, and the presence of Ca in alkaline soils. When the results of the orthophosphate analysis of the soil moisture samples were analysed and averaged over the year it was found that most of the phosphorus fixation occurred above the red plane (Table 8.11). However, there were four occasions on which data were available for the black depth plane and Figure 8.20 and Table 8.12 show that on these occasions there was a noticeable increase in orthophosphate fixation between the blue and black planes. Particle size analysis of a sample taken at 1.0m depth, or 0.2m below the invert of the percolation pipe, shows it to have a higher clay content than samples at 1.5m and 2.0m depth suggesting a greater affinity for phosphate sorption (Appendix A). Furthermore, while no particle size data are available for the subsoil between the base of the percolation trench and the 1.0m sample depth it has been classified, using BS5930, as a sandy Clay (w/silt) while the subsoil below this point was classified as a sandy Silt (w/clay). X-ray diffraction analysis of a sample taken from the sandy Silt (w/clay) layer shows that while, it contains calcite, it is devoid of Al, Fe and Mn, oxides, hydrous oxides or dissolved ions (Appendix A) and therefore, in such a medium, fixation would be confined to the high pH range. This can be seen on the few occasions where data were available from the black plane (Table 8.12).

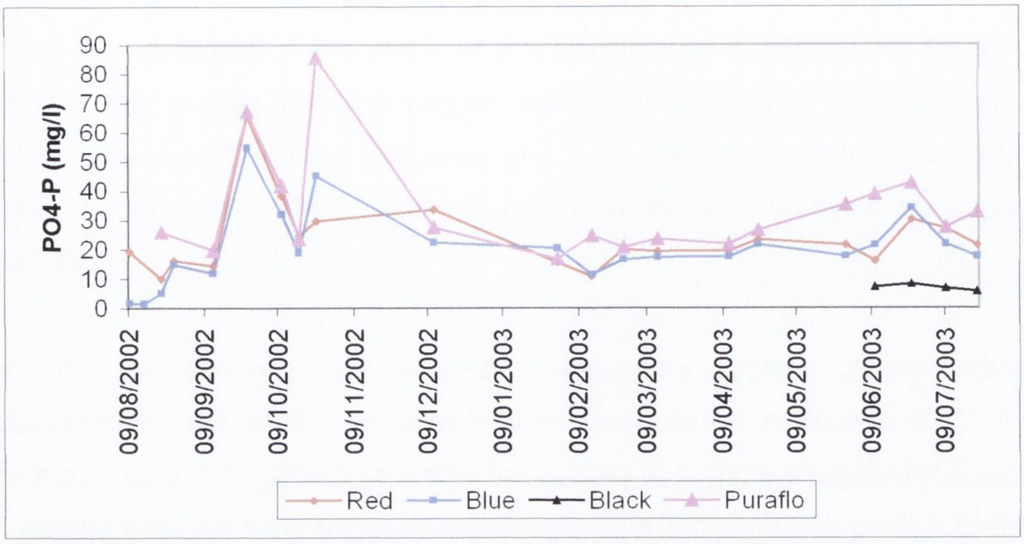


Figure 8.20 Average orthophosphate concentrations measured.

	Conc. (mg/l)	Loading (g/d)	pH
SE	33.6	9.5	6.42
Red Depth Plane	23.9	6.9	6.30
Blue Depth Plane	20.3	5.8	7.22

Table 8.11 Average orthophosphate concentration and loading rates measured over project duration.

	Conc. (mg/l)	Loading (g/d)	pH
SE	35.5	10.0	7.17
Red Depth Plane	23.8	6.8	6.47
Blue Depth Plane	23.8	6.8	7.24
Black Depth Plane	6.9	2.2	8.15

Table 8.12 Average orthophosphate concentration and loading rates measured on the four occasions where black depth plane samples were available.

8.4.4 Results of Bacteriological Analysis

During the course of the bacteriological analysis it was necessary to dilute some of the soil moisture samples as volumes were too small, or concentrations too high, for analysis. This affects the minimum detection level of the bacterial concentrations and results are therefore expressed in the form <x cfu/100ml to take account of the dilution. For example, if the result of the analysis on a sample for *E. coli* is <10 cfu/100ml this means that the original sample underwent a 1 in 10 dilution with distilled water and while the presence of *E. coli* was not detected the dilution step must be accounted for. The results of the bacteriological analysis are contained in Appendix E.

While the installation of the secondary treatment system greatly reduced the bacteriological load on the percolation area, as outlined in Section 8.3.2, it can be seen from Table 8.13, which presents the results of sample analysis for *E. coli* at the 0m sample position on 4 separate occasions over the research period, that there is some evidence of enteric bacterial contamination with depth. While samples were also analysed for total coliforms the results of this analysis were excluded from Table 8.13 as their presence in the soil moisture samples at low concentrations is not necessarily indicative of domestic wastewater contamination (Section 7.4.2). While there is no evidence of bacteriological contamination on the red and blue depth planes, the one sample taken from the black plane was contaminated with enteric bacteria. As the particle size analysis of the subsoil revealed it to have a high sand

Depth Plane	Number of Samples	Number of samples with concentration (cfu/100ml)			
		<10	10-100	101-1000	>1000
Red	5	5	0	0	0
Blue	5	5	0	0	0
Black	1	0	1	0	0

Table 8.13 Concentrations of *E. coli* on sample planes at the 0m position on 4 separate occasions.

content it is therefore possible that the associated grain size, and consequently pore size, facilitated the movement of bacteria through the subsoil. The reduced biomat development, the presence of which would improve filtration of the percolating effluent, would also have the effect of increasing the hydraulic load per unit area.

The tracer tests carried out on site show that by day 8 the presence of bromide had been recorded on the three depth planes of the 0m sample position. While the main mechanisms for the removal of enteric bacteria from the percolating effluent are inactivation/die-off, filtration and adsorption, analysis of these results allied with literary evidence of enteric bacterial survival times in subsoils suggests that filtration and adsorption, rather than die-off, were the dominant removal mechanisms at work.

8.5 Summary

Samples obtained from the small lysimeters showed that the percolation area was under utilised with less than 28m of the 80m of trench constructed receiving effluent. The average hydraulic load of 281.7l/d measured over the project duration would only require 25m of percolation trench rather than the 80m constructed (EPA, 2000) highlighting that the percolation area was over-designed hydraulically. However, as this site was only monitored for a year it is possible that the biomat is still forming along the base of the percolation trenches and that with time more even distribution along the trench bases would be achieved.

The installation of a secondary treatment system in the form of a Puraflo® unit greatly reduced the bacterial and organic load on the percolation area. This reduction in organic load also had the effect of inhibiting biomat formation and thus the distribution of effluent over a greater area. While the aerobic environment of the Puraflo® facilitated the nitrification of NH_4-N a reduction in overall nitrogen loading was also experienced across the system due to perceived saturation in the base of the modules which would mean that the total nitrogen loading to the groundwater could, in general, be higher.

Within the subsoil it can be seen that most of the attenuation of the percolating effluent occurred above the red depth plane (Tables 8.14 and 8.15). Allowing for the effects of dilution by effective rainfall, it appears that approximately 98% of this reduction was due to chemical, physical and biological activity within the subsoil. While the blue depth plane represents the minimum recommended thickness of unsaturated soil above the watertable, or the point of discharge of effluent to groundwater, the potential for further effluent contamination between this point of discharge and any potential target must also be considered.

Sample Point	Average Load (g/d)		
	COD	Total Inorganic N	PO_4-P
STE	223.0	37.4	9.1
SE	55.5	18.0	9.5
Red Depth Plane	30.9	17.5	6.9
Blue Depth Plane	21.9	16.7	5.8

Table 8.14 Average load measured on Site 1.

Sample Position	Number of Samples	Number of samples with concentration (cfu/100ml)			
		<10	10-100	101-1000	>1000
Puraflo®	4	0	0	0	4
Red Plane	5	5	0	0	0
Blue Plane	5	5	0	0	0

Table 8.15 Comparison of *E. coli* concentrations measured during the project.

9. ANALYSIS OF RESULTS OBTAINED FROM SITE 2

9.1 Introduction

A successful site assessment was completed for Site 2 on February 7th 2002. However, as was the case at Site 1, installation of the septic tank and the construction of the percolation area was delayed and was not completed until 29th May, when the site was commissioned, due to adverse weather conditions and delays resulting from the procurement of construction materials and sampling and analysis equipment. Diversion works, which were required to separate the domestic wastewater effluent from the roof runoff, were completed on the 19th July. The installation of sampling equipment commenced on 11th June and all equipment was in place by 18th June. Sampling began at Site 2 on the 8th of August 2002 and continued until the15th July 2003.

9.2 Analysis of Flow Data

The nature of STE flow and the absence of a sump downstream of the septic tank on Site 2 created difficulties for the measurement of flow from the septic tank (Section 7.2.2). When these difficulties were overcome flow was measured using an Omega-SL-L320 datalogger and ultrasonic LVU-90 level sensor and commenced on 4th February 2003 (Appendix D).

As can be seen from Table 9.1 the average flows on Site 2 were greater then those measured on Site 1. The dwelling house and associated wastewater treatment system on Site 2, having been constructed in the 1950's, consisted of a clay pipe network, for both surface and wastewater, and a single chamber septic tank with associated soakaway. While extensive separation works were carried out upstream of the septic tank during the installation of the EPA (2000) recommended septic tank and percolation area some intrusion into the domestic wastewater network was partially responsible for this greater flow. Periodic inspection of the distribution box during the early stages of the project, prior to the commencement of flow measurement, had aroused the suspicion of surface water intrusion. It was discovered that the diversion works had overlooked a drain in the stable yard that was connected to the septic tank. This drain received surface runoff which was a combination of rainfall and daily washings from the stables. However, while there appeared to be no correlation between rainfall and STE flow, as can be seen from Figure 9.1, chemical analysis appeared to confirm this contribution (Section 9.3.4).

	Maximum	Minimum	Average
Daily Flow (l/d)	1228	109.3	418.8
Total Flow (l/d)	N/A	N/A	400.0

Table 9.1 Septic tank effluent flows measured on Site 2.

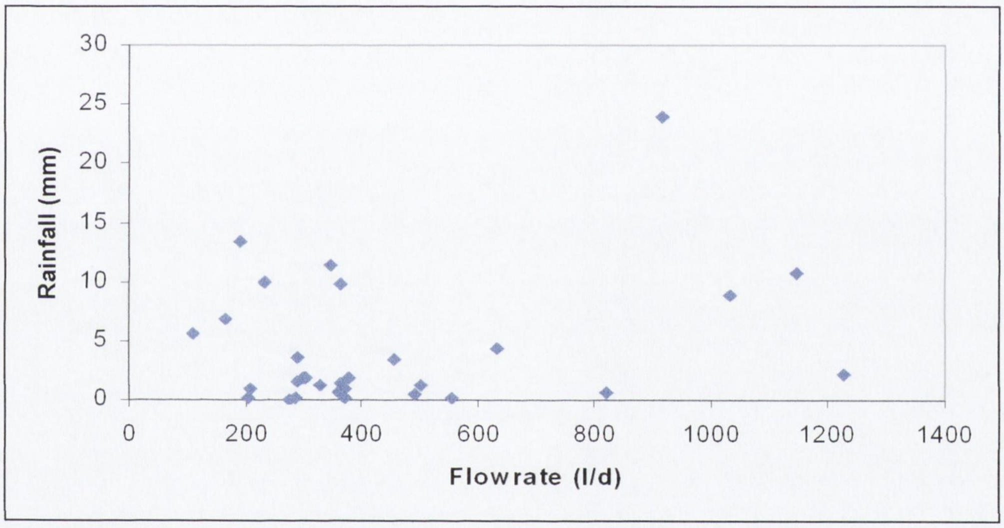


Figure 9.1 Graph of measured flow (blue) against rainfall on Site 2.

As the average daily flow record of 418.8l/d, or 104.7 l/pd, includes a contribution by surface runoff, the actual daily average of domestic wastewater generation would be less. As was the case at Site 1 this means that, as the sizing of the percolation area was based on the recommended loading rate of 20 $l/m^2/d$ and a typical daily hydraulic loading of 180l/person (EPA, 2000), the percolation area was over designed hydraulically.

The frequency distribution of the recorded flow rates shows that for the 49 days for which daily flow records are available 67% of the flows recorded fall in the range 161

– 380l/d (Figure 9.2). The average flow over this range was 278.2l/d. Unlike Site 1 where flow calculations had to take account of holidays there was no prolonged period over the project duration when the dwelling on Site 2 was vacant.

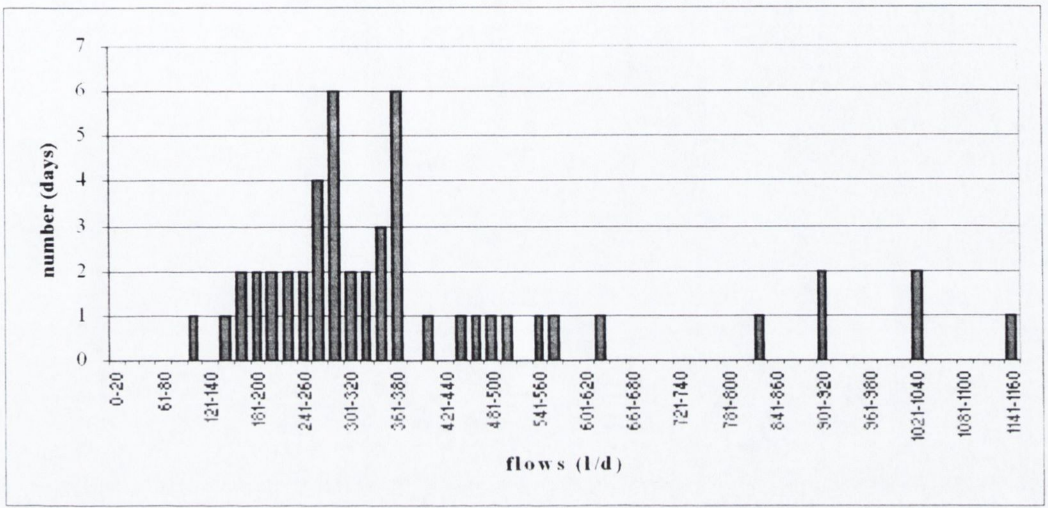


Figure 9.2 Frequency distribution of STE flows recorded on Site 2.

The achievement of even distribution between the four percolation trenches depended on the design of the distribution box which had to take account of the influent flow regime. Analysis of the flow emanating from the septic tank on Site 2 showed that the average STE flowrate was less than 3.8l/min for 90% of the monitoring period (Figure 9.3). While the modification to the distribution box has been shown,

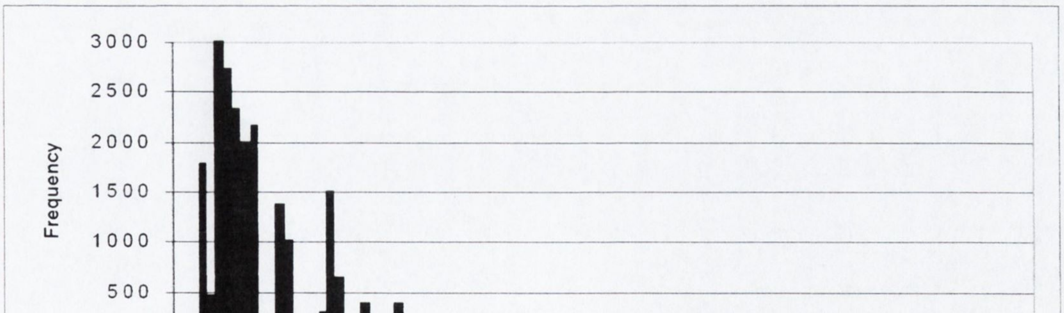


Figure 9.3 Frequency distribution (l/min) of flows recorded on Site 2.

under laboratory conditions, to perform well over the range of flows measured (Section 6.2.3) it must be considered that, as STE was pumped from the monitoring sump to the distribution box at a flowrate of 1.6l/sec, this is not a true representation of flowrate into the distribution box. It is possible, therefore, that even distribution of effluent was not achieved within the distribution box (Section 9.3.3).

9.3 Results of Analysis of Septic Tank and Soil Moisture Samples

9.3.1 Method of Analysis

As can be seen from Table 7.2 it was not possible to install all identically coloured lysimeters to similar depths and therefore, prior to analysis of the chemical and bacteriological results, reclassification of some lysimeters was required. To achieve this, three nominal depths of 0.2m (red), 0.55m (blue) and 0.9m (black) below the invert of the percolation trench, referred to as depth planes, were defined. The subsoil below the percolation trench was divided into three sections: 0 to 0.4m, 0.4 to 0.7m and 0.7 to 1.1m. The depth planes were chosen to represent the middle of these sections. The lysimeters were then reclassified, taking into account their location within the subsoil, and results reported citing the three depth planes. For example, the red lysimeter located at the 20m sample position on trench 4 was reclassified as a blue lysimeter.

The results of the analysis of soil moisture samples for Cl were used to determine which of the two methods outlined in Section 8.4.1, planar average or depth average, was the better method for representing the distribution of STE within the percolation area. When the average Cl concentrations at the three sample positions on the red plane were graphed (Figure 9.4) it suggested that the planar average method was the more representative as little difference in the concentrations was observed between the three sample positions, except on the 08/01/03 where sample 10 had a measured concentration of 212mg/l (this was 2.4 times greater than the next highest Cl measurement and was therefore dismissed as egregious). Similarly when the average Cl concentrations measured at all sample positions (Figure 9.5)

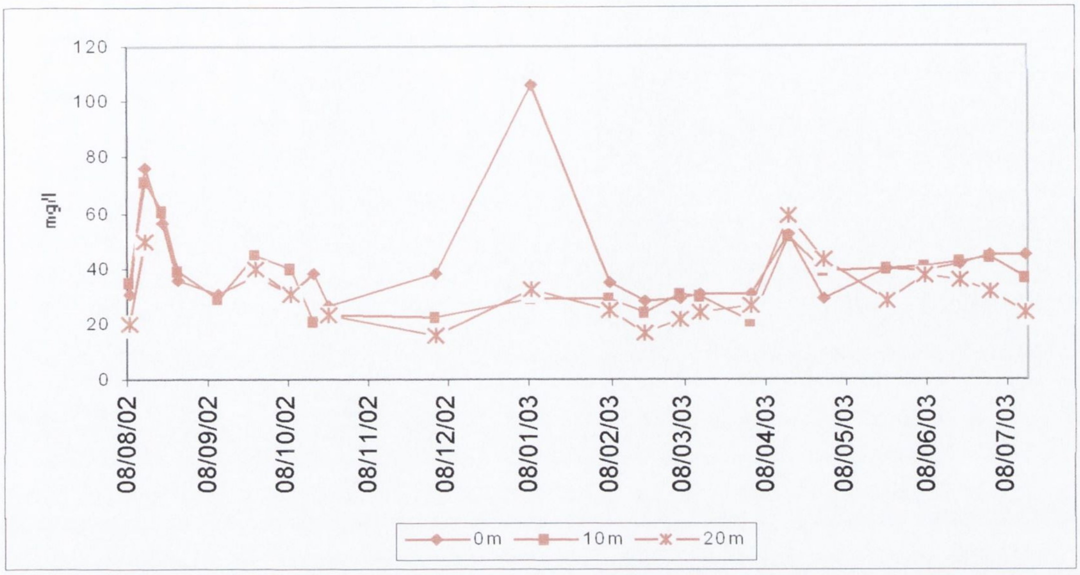


Figure 9.4 Average Cl concentrations measured on the red plane at the three sample positions.

and the average planar Cl concentrations (Figure 9.6) are graphed it can be seen that the concentrations measured for the different sample depths are similar. As the results of the T-test for Site 2 showed it to have a higher percolation rate, i.e. lower T-value, than Site 1 it appears that the formation of a biomat, discussed in more detail in Section 9.3.3, facilitated the distribution of effluent along the full length of the

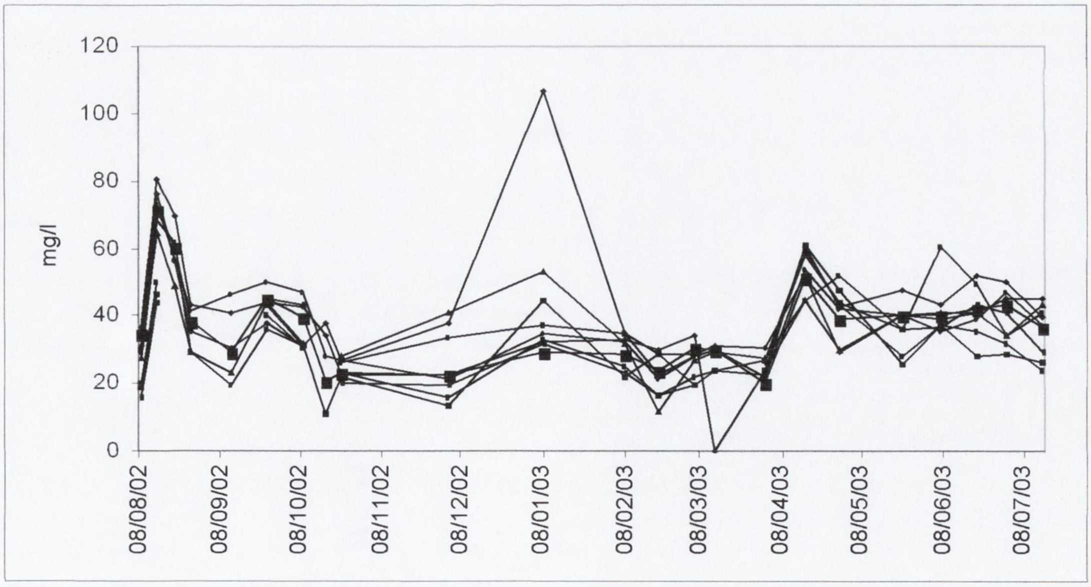


Figure 9.5 Average Cl concentrations measured for the three depth planes at the three sample positions.

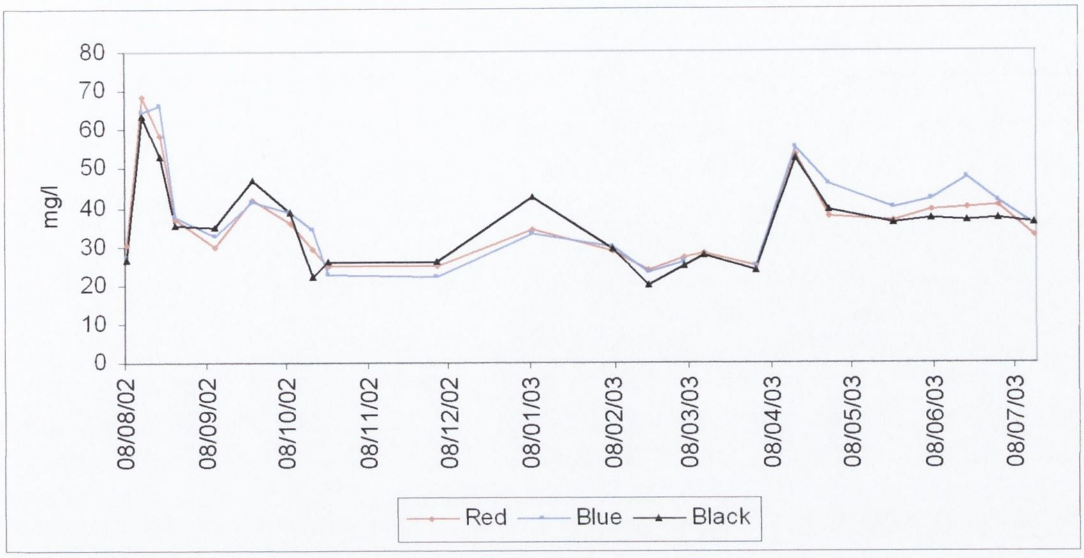


Figure 9.6 Planar average Cl concentrations.

percolation trenches. This is corroborated by the results of the bromide tracer study (Appendix F) which was carried out at the same time as the tracer study on Site 1 and revealed the presence of Br in samples taken from all sample positions over an 8-day period. While Br was not recovered from all sample positions it is possible that it arrived at these lysimeters on the days when sampling was not carried out. As can be seen from Table 9.2 the time of travel of the tracer from the distribution box to the individual sample points on the same depth plane varied. It must be

Sample Position	Lysimeter	Trench			
		1	2	3	4
0m	**Red**	Broken	Day 2	Day 2	Day 2
	Blue	Day 8	Day 8	Day 2**	Day 2
	Black	None	Day 2	Day 3	Day 2
10m	**Red**	Day 2	Day 2	Day 2	Day 2
	Blue	Day 2	Day 2**	Day 2	Day 2
	Black	Day 2	None	Day 2	Day 2
20m	**Red**	Day 3*	Day 3	Day 8	None
	Blue	None	Day 3	Day 8	None**
	Black	Broken	Day 3	Day 2	Day 2

*** denotes lysimeter reclassified as blue; ** denotes lysimeter reclassified as black.**

Table 9.2 Time of first arrival of Br at sample points on Site 2.

considered that, due to the sampling regime, it is possible that the presence of Br in samples on day 8 was not the first incidence of Br at these sample points. The presence of Br on day 2 at the black lysimeter at the 20m sample position on trench 4 is an isolated event and, as effluent was perceived not to have reached this sample position (Section 9.3.3), could be due to sample contamination.

During the project it became apparent that certain lysimeters were consistently providing relatively larger sample volumes than others. As can be seen from Table 9.3 average sample volumes obtained from sample points 16, 17, 18, 23, 24 and 33 were substantially larger than sample volumes obtained from the other sample

Sample Point	Average Volume	Sample Point	Average Volume	Sample Point	Average Volume	Sample Point	Average Volume
1	N/A	**10**	45	**19**	289	**28**	68
2	65	**11**	43	**20**	192	**29**	110
3	50	**12**	33	**21**	81	**30**	54
4	189	**13**	165	**22**	220	**31**	501
5	452	**14**	39	**23**	734	**32**	471
6	270	**15**	177	**24**	1022	**33**	891
7	279	**16**	841	**25**	391	**34**	111
8	474	**17**	953	**26**	486	**35**	279
9	N/A	**18**	1091	**27**	568	**36**	164

Table 9.3 Average soil moisture sample volumes obtained on Site 2.

possible that the installation of these lysimeters, despite implementation of best practice, was not successful and that samples taken at these points were not representative of matrix flow but of preferential flow down the side of the lysimeters due to poor contact between the lysimeter and the subsoil. Whereas analysis of the conservative 'tracer' Cl (due to the fact that Cl does not take a significant part in geochemical reactions) did not reveal any significant difference between the Cl concentrations recorded at these sample points and the other sample points (Table 9.4), analysis of NH_4-N for the different sample points did show changes. Table 9.5

shows that soil moisture samples obtained from these high-volume sample points had a much higher concentration of NH_4-N than samples obtained from the other

Sample Point	Average conc.	Sample Point	Average conc.	Sample Point	Average conc.	Sample Point	Average conc.
1	N/A	10	33	19	41	28	45
2	42	11	33	20	37	29	37
3	39	12	36	21	36	30	37
4	34	13	38	22	40	31	40
5	40	14	37	23	45	32	42
6	38	15	36	24	44	33	45
7	25	16	40	25	32	34	16
8	27	17	45	26	32	35	17
9	N/A	18	44	27	31	36	19

Table 9.4 Average Cl concentrations (mg/l) measured under percolation area on Site 2.

Sample Point	Average conc.	Sample Point	Average conc.	Sample Point	Average conc.	Sample Point	Average conc.
1	N/A	10	2.5	19	20.1	28	1.0
2	7.5	11	5.3	20	3.5	29	1.2
3	8.2	12	1.6	21	1.7	30	1.6
4	14.3	13	6.8	22	22.6	31	27.7
5	28.1	14	2.6	23	39.3	32	22.2
6	19.9	15	0.9	24	35.3	33	30.3
7	4.8	16	31.76	25	11.6	34	2.6
8	4.8	17	32.3	26	9.1	35	3.4
9	N/A	18	33.4	27	9.5	36	5.0

Table 9.5 Average NH_4-N concentrations (mg/l) measured under percolation area on Site 2.

lysimeters. When the reduction in Cl concentration between the STE, which has an average Cl concentration of 57mg/l, and these sample points is considered, i.e. the effect of dilution, the average STE NH_4-N concentration (52.9mg/l) and soil moisture

NH_4-N concentrations for these sample points are very similar (Table 9.6). Allied to this, it was found that when these lysimeters were removed from the subsoil their bases were dripping with STE and covered by a black biofilm (Figure 9.7) which suggested that they had been submerged in STE. No biofilm was present on the other lysimeters. For these reasons it was assumed that initial installation of these lysimeters had been unsuccessful and that preferential flowpaths had been created between the invert of the percolation trench and the lysimeter cups. Data from these sample points were therefore excluded from further chemical and biological analysis.

Sample Point	Concentration (mg/l)		
	Cl	NH_4-N	Adjusted NH_4-N
X	57	52.9	N/A
16	40	31.7	45.3
17	45	32.3	40.9
18	44	33.4	43.4
23	45	39.3	49.8
24	44	35.3	45.9
33	45	30.3	38.3

Table 9.6 Comparison of STE NH_4-N concentration soil moisture NH_4-N concentrations adjusted for dilution.

Figure 9.7 (a) and (b) Biofilm build-up on preferential flow lysimeters.

.As can be seen from Table 9.4 soil moisture samples 4, 5, 6, 19, 22, 31 and 32 also had high average NH_4-N concentrations although these lysimeters did not produce corresponding high sample volumes. On removal, lysimeters 5, 22, 31 and 32 were covered by a black biofilm suggesting that the installation of these lysimeters was also unsuccessful. When the NH_4-N concentration for these sample points was graphed against that of sample point 33 they were shown to be very similar (Figure 9.8). This data were therefore excluded from further analysis as not being representative of matrix flow conditions. Data from points 4, 6 and 19 were not excluded, however, as when the NH_4-N concentrations measured at these points were graphed against NH_4-N concentrations at point 33 it was observed that, after an initial period of similarity, concentrations at points 4, 6 and 19 dropped off (Figure 9.9). It should also be noted that these lysimeters, on removal, were not covered by a biofilm. It is possible, therefore, that while initial installation of these lysimeters might not have been successful any preferential flowpaths created during their installation were blocked over time. by fines, biomat formation, or colloidal matter within the effluent. This is reflected in Figure 9.10 which shows the reduction in sample volume obtained from these points with time.

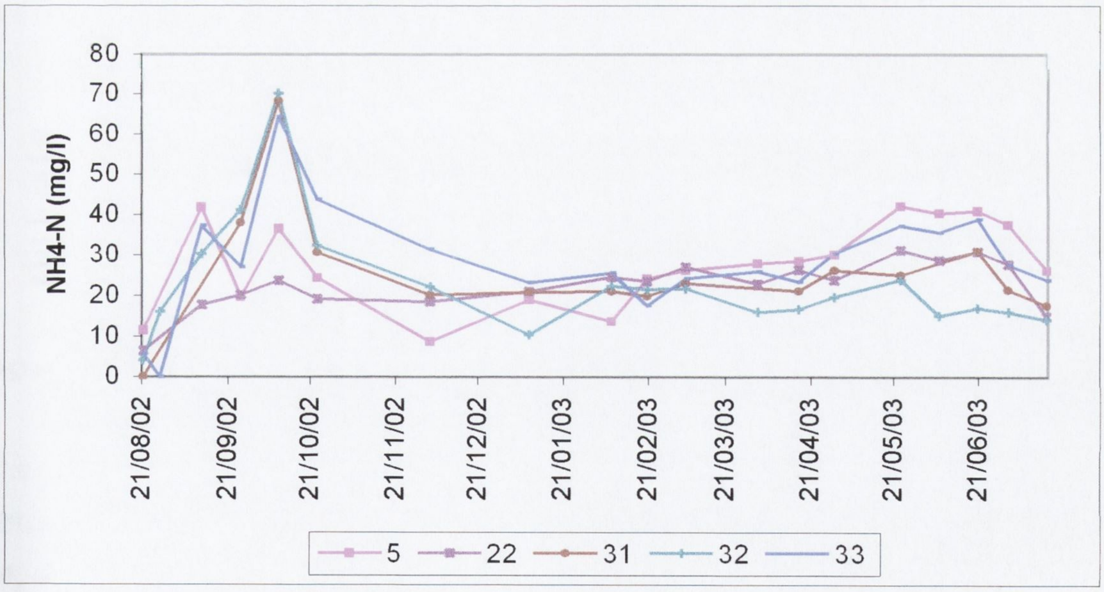


Figure 9.8 NH_4-N concentrations measured at sample points 5, 22,31,32 and 33.

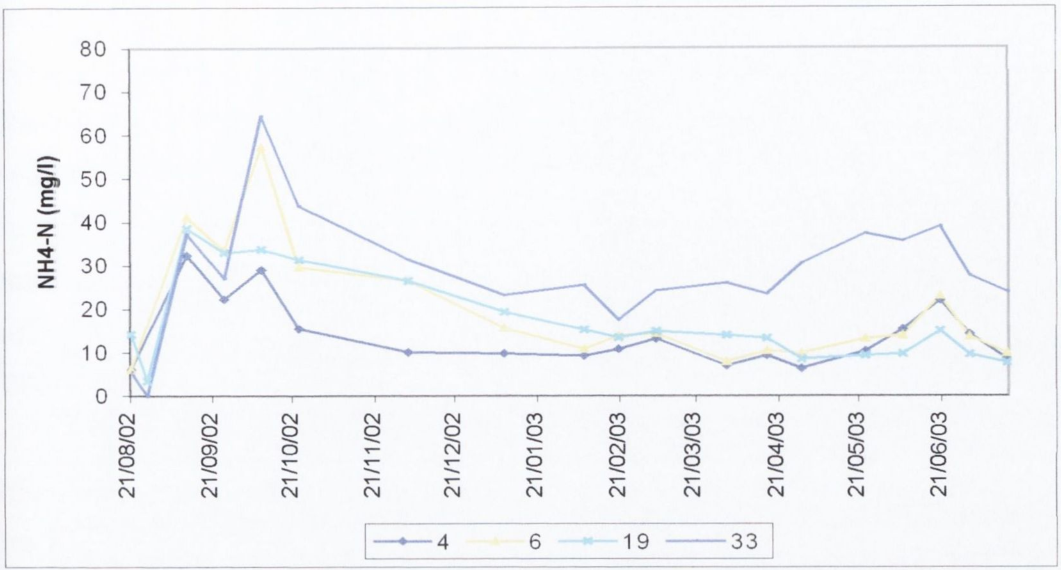


Figure 9.9 NH_4-N concentrations measured at sample points 4, 6, 19 and 33.

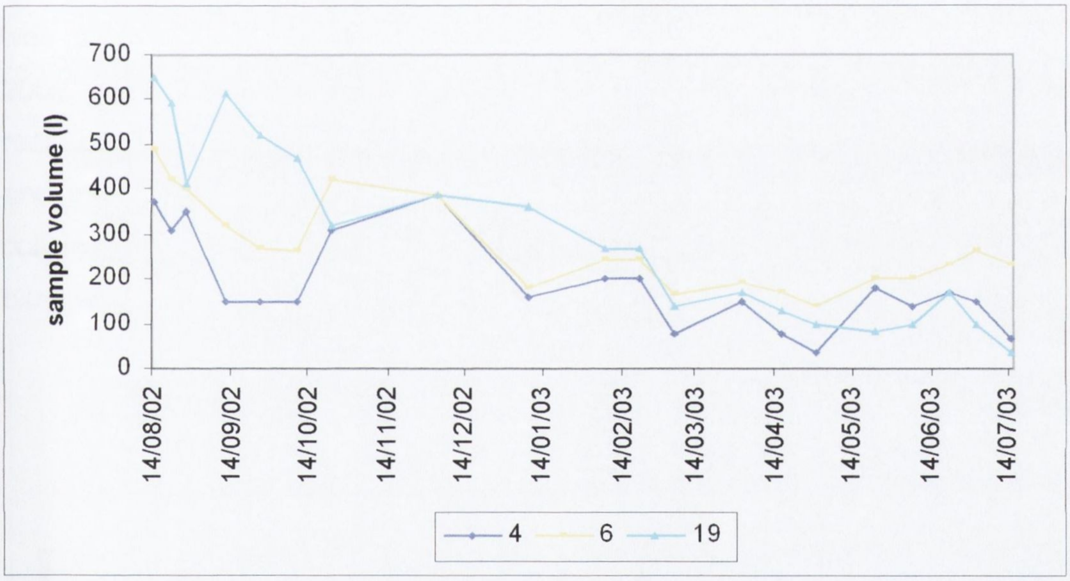


Figure 9.10 Sample volumes measured at points 4, 6 and 19.

Therefore, from a total of 36 lysimeters installed it was necessary to omit data obtained from 10. As two lysimeters were broken during installation this left a total of 24, seven, six and 11 on the red, blue and black depth planes respectively.

9.3.2 The Effect of Dilution on Effluent Attenuation

While the attenuation of the percolating effluent is mostly the result of physical, chemical and biological processes within the subsoil the effects of dilution must also be considered. As was the case for Site 1, the effective rainfall over the project duration was calculated using rainfall data obtained on site and meteorological data obtained from the meteorological station at Pollardstown Fen 3.5km away. As outlined in Section 8.4.2 PET, was calculated using the model developed by Bartley (2004) and AET was calculated using the Aslying scale.

The measured rainfall for Site 2 from the 01/08/02 to 15/07/03 was 852.8mm. Using the Penman-Monteith method it was calculated that the effective rainfall over this period was 385.5mm. The Hargreaves effective rainfall value calculated for Site 2 was 372.5mm. As can be seen by Figure 9.11 and Figure 9.12 dilution of the effluent was greatest during the period of sustained effective rainfall between November 2002 and March 2003. As Cl does not take a significant part in geochemical reactions it is probable that the reduction in Cl concentration between the septic tank and the red plane, during period of no effective rainfall, is due to the removal of colloidal matter by the combined straining action of the subsoil and associated biomat.

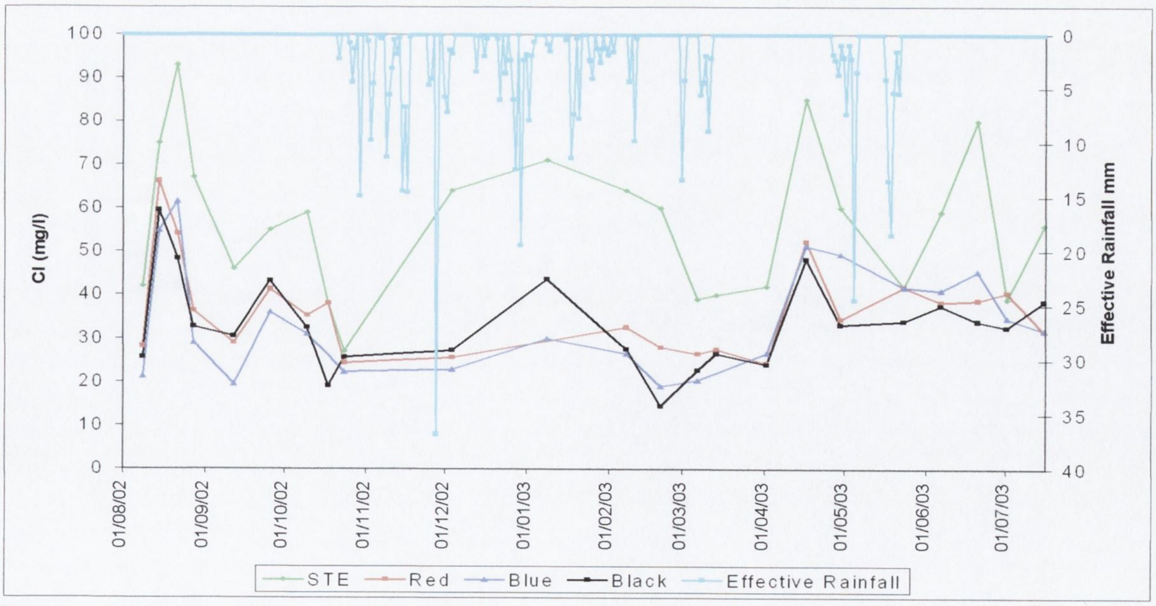


Figure 9.11 The effect of dilution by effective rainfall, calculated by the Penman-Monteith method, on Cl concentrations.

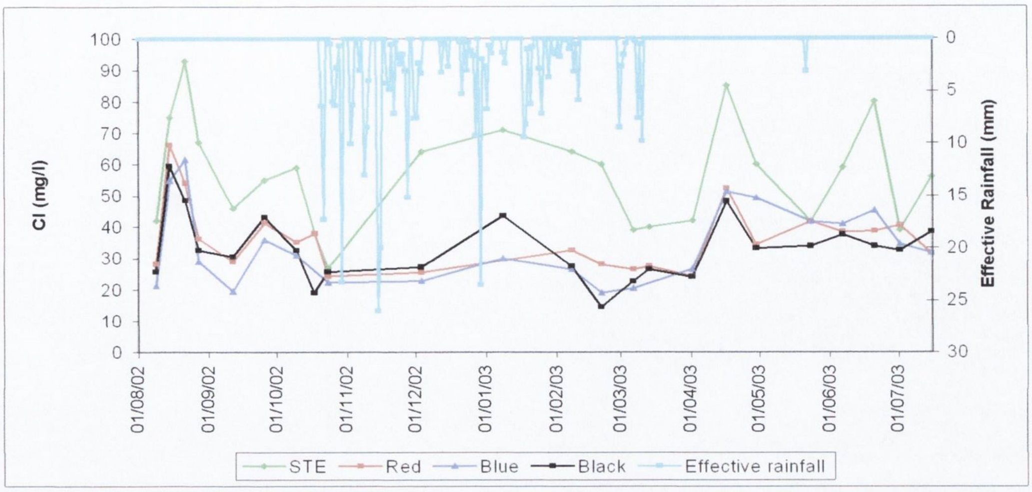


Figure 9.12 The effect of dilution by effective rainfall, calculated by the Hargreaves method, on Cl concentrations.

Using the same method as outlined in Section 8.4.2 Cl concentrations were used to quantify the contribution of effective rainfall to effluent dilution (Table 9.7). However, as can be seen from Table 9.4, average Cl concentrations at sample points 7, 8, 34, 35 and 36, which are all at the 20m sample position (Figure 7.11), are lower than average Cl concentrations measured at the other sample points. This is due to the time it takes for the biomat to develop along the entire length of the percolation trench (dealt with in more detail in Section 9.3.3). For this reason all concentrations measured at points 7 and 8 were excluded until 01/04/03 while all concentrations measured at points 34, 35 and 36 were excluded completely. While sample points 25, 26 and 27 are also located at the 20m sample position they provided no sample until 23/10/02. It was found that, on average, a 39.6%, 41.6% and 42.9% reduction in Cl concentration was recorded between the STE and the red, blue and black planes, respectively (Table 9.7). When the average Cl concentration for rainfall on Site 2 of 3.9mg/l is taken into account this equates to the addition of 0.79 litres, 0.91 litres and 1.02 litres of effective rainfall per litre of effluent on the red, blue and black depth planes, respectively.

However, this reduction in Cl concentration is not entirely due to dilution as it also results from the effects of physical straining on the percolating effluent. This effect was quantified by examining the reduction in Cl concentration between the septic

Date	% reduction in concentration			Equivalent contribution of effective rainfall (l)		
	Red Plane	Blue Plane	Black Plane	Red Plane	Blue Plane	Black Plane
08/08/02	24.76	29.37	33.67	0.38	0.48	0.59
21/08/02	41.94	33.87	47.98	0.78	0.55	1.00
27/08/02	45.90	56.72	51.53	0.95	1.51	1.20
11/09/02	36.96	57.61	33.70	0.68	1.70	0.58
25/09/02	25.00	34.55	21.45	0.37	0.59	0.30
09/10/02	38.42	47.46	42.56	0.70	1.03	0.84
17/10/02	N/A	N/A	50.00	N/A	N/A	1.26
03/12/02	57.03	55.47	52.26	1.55	1.44	1.25
08/01/03	45.92	48.83	42.25	0.94	1.07	0.81
07/02/03	49.06	45.83	57.01	1.09	0.95	1.24
20/02/03	53.33	60.42	75.83	1.33	1.83	4.29
06/03/02	25.64	47.69	27.47	0.40	1.13	0.44
13/03/03	31.25	N/A	33.75	0.53	N/A	0.60
01/04/03	42.38	36.67	42.86	0.88	0.68	0.90
16/04/03	38.53	39.71	43.33	0.68	0.71	0.83
29/04/03	42.67	17.78	44.72	0.84	0.23	0.92
23/05/03	N/A	18.80	N/A	N/A	0.26	N/A
06/06/03	34.97	30.55	36.41	0.60	0.49	0.64
20/06/03	46.25	43.33	57.61	0.95	0.84	1.19
15/07/03	32.14	43.21	31.38	0.53	0.87	0.51
Average	**39.6**	**41.6**	**42.9**	**0.79**	**0.91**	**1.02**

Table 9.7 Reduction of Cl concentration with depth.

tank and red depth plane between 08/08/02 and 17/10/02 when the contribution of effective rainfall was zero. It was found that, over this period, an average reduction in Cl concentration of 27.1% was recorded. When this was deducted from the average overall reduction in Cl concentration for each plane it was found that the effect of dilution was equivalent, on average, to the addition of 0.15 l/l (or 12.5% reduction in concentration) for the red depth plane, 0.18 l/l (or a 14.5% reduction in concentration) for the blue depth plane and 0.20 l/l (or 15.8% reduction in concentration) for the black depth plane.

Using the average daily flow of 418.8 litres (Table 9.1) this equates to an average daily effective rainfall contribution of 104.7 litres, 134.0 litres and 159.1 litres to the red, blue and black depth planes respectively. Using the same method outlined in Section 8.4.2 it was estimated that the zone of contribution for the red plane was only slightly greater than the trench width at 0.4m on all sides of each trench. As was the case for Site 1 the zone of contribution was greater for the blue and black planes and appeared to extend in the region of 0.6 to 0.8m on all sides of each percolation trench. For these calculations it was assumed that the effluent was evenly distributed over the entire base of trenches 1 to 3, as was the case for the periods of greatest effective rainfall, but that it was only distributed over 10m of trench 4.

As effluent on trench 4 had not reached the 20m sample position it can be seen from Figure 9.13 that the variation in soil moisture tension is a result of varying contribution of effective rainfall over the monitoring period. Figures 9.14 and 9.15, which present the effective rainfall and soil moisture tension records for the 0m and 10m sample points respectively, show that the tensiometer readings are affected more by the percolating effluent than the contribution of effective rainfall.

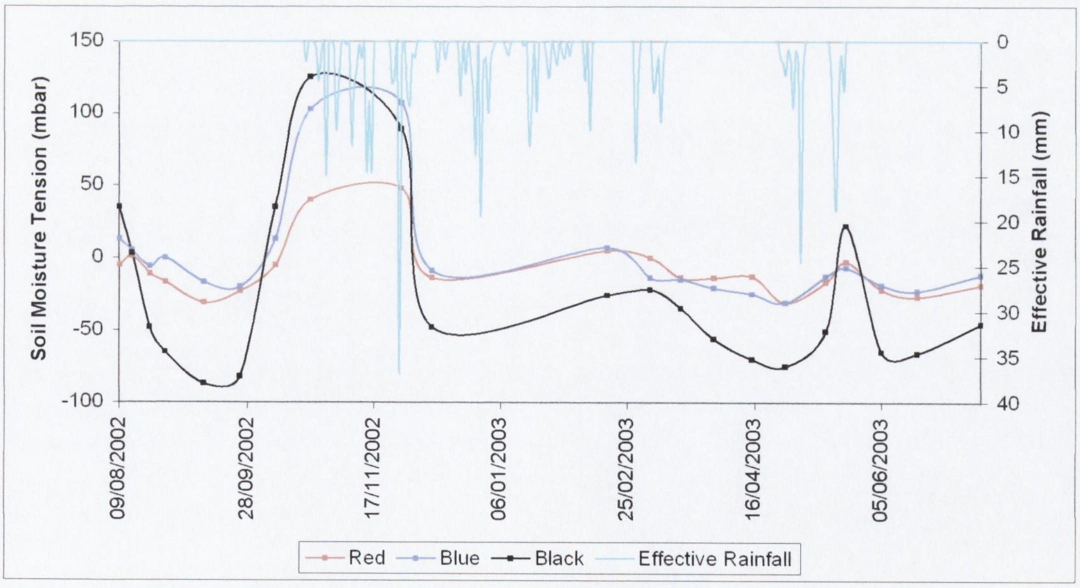


Figure 9.13 Soil moisture tension plotted against effective rainfall for the 20m sample position on Site 2.

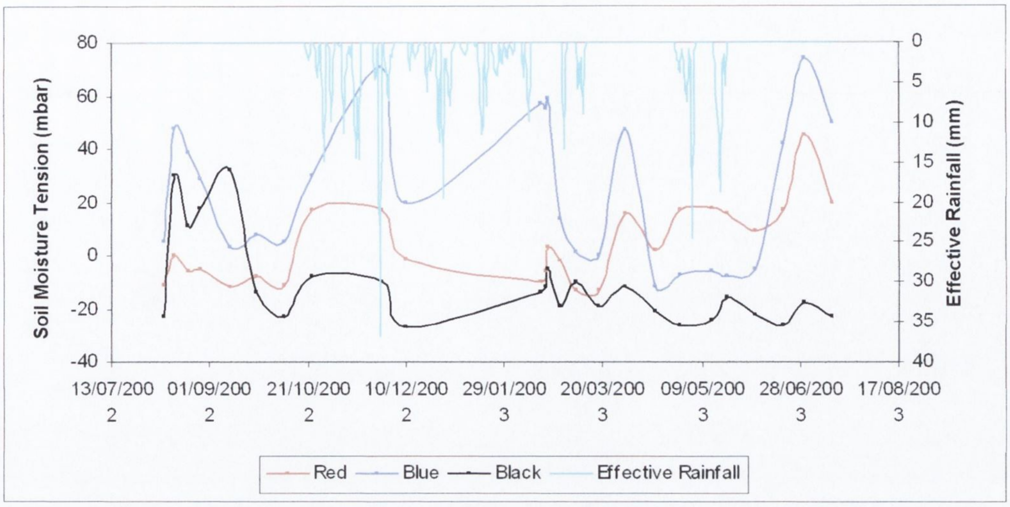


Figure 9.14 Soil moisture tension plotted against effective rainfall for the 0m sample position on Site 2.

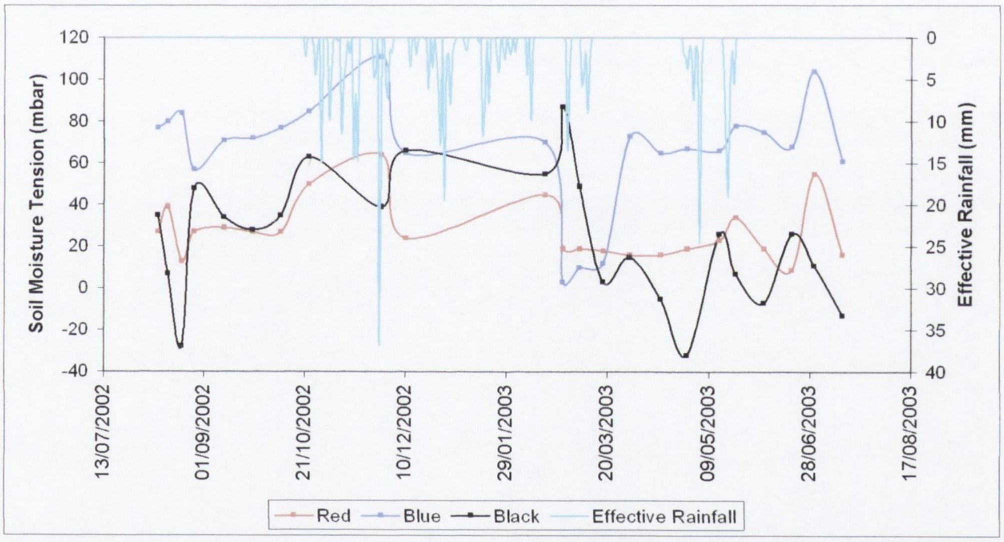


Figure 9.15 Soil moisture tension plotted against effective rainfall for the 10m sample position on Site 2.

This appears to corroborate the evidence of that dilution calculations which show that it is physical, chemical and biological processes rather than dilution that are the more prominent attenuation processes operating in the subsoil. It appears in Figure 9.15 that the red and blue tensiometers at the 10m sampling position were under saturated conditions throughout the monitoring period as a result of preferential flow of effluent from the percolation trench. This was corroborated by the presence of a black biofilm, similar to that seen on the lysimeters (Section 9.3.1), seen on those tensiometers upon removal.

9.3.3 Biomat Formation

As outlined in Section 9.3.1, Cl analysis of the soil moisture samples from Site 2 revealed that STE had been distributed along the entire length of the percolation trenches. This would suggest that the organic load of the STE, not having undergone a secondary treatment phase, facilitated the formation of a biomat along the effluent subsoil interface thereby promoting distribution along the base of the percolation trench. The formation and development of the biomat along the trench length, while promoting effluent distribution, was also dependent on the presence of effluent. The presence of elevated Cl levels at the 10m sample position throughout the project suggests that at the commencement of monitoring the biomat had already developed between the 0m and 10m sample positions. This was due to the time lag between site commissioning, 29th May, and the commencement of monitoring, 8th August.

As revealed by the results of the Cl analysis the presence of effluent was not recorded at the 20m sample position until 25/09/02 and this on Trench 3 only (Figure 9.16). While sample points 16, 17 and 18 were excluded from chemical analysis due to the presence of preferential flowpaths it is worth noting that, when Cl concentrations only are examined, the behaviour of the biomat in trench 2 mirrored that in trench 3. As the presence of effluent was not recorded at the 20m sample position on trench 1 until 01/04/03, and was never recorded at the 20m sample position on trench 4 during the project, it is clear that the rate of biomat progression was not constant along all trenches. It is possible that the progression of the biomat along trenches 1 and 4 was slower due to a smaller hydraulic load on these trenches compared to that on trenches 2 and 3. As the STE was pumped to the distribution box to facilitate flow measurement it is possible that the adjustment to the distribution box, i.e. the addition of the v-notch weirs, while improving distribution, did not equally distribute the effluent between the four outlets but favoured the back two.

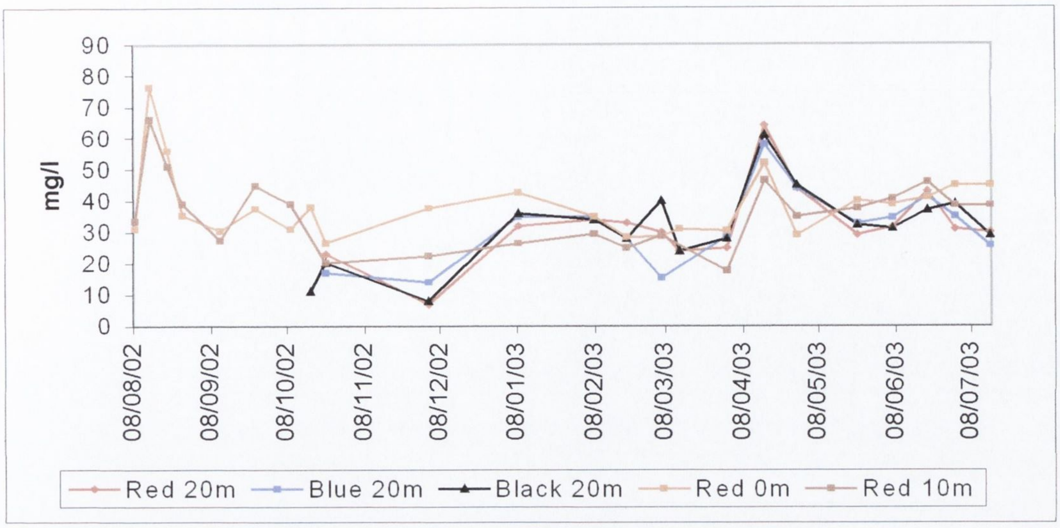


Figure 9.16 Using Cl concentration to highlight biomat progression.

9.3.4 Results of Chemical Analysis

The results of all chemical analysis carried out on STE and soil moisture samples obtained from Site 2 are contained in Appendix E. Table 9.8 summarises the results relating to the STE. As a result of the anaerobic environment of the septic tank the dominant form of inorganic nitrogen is NH_4-N. While STE concentrations are similar to those outlined in Section 2.2.3 it can be seen that, overall, the average concentrations in the STE on Site 2 are much lower than those measured on Site 1. This could be due to surface water infiltration as outlined in Section 9.2. The infiltration of surface water could also, therefore, be expected to contribute to the variation in STE concentration throughout the year. It must also be considered that as the house on Site 2 was constructed in the 1950s some of the water using domestic appliances, such as toilet cisterns, would be of greater volume.

	Concentration (mg/l)					
	COD	NH_4-N	NO_2-N	NO_3-N	PO_4-P	Cl
Maximum	638.0	72.8	0.23	3.4	54.8	93.0
Minimum	188.0	20.3	0.08	0.0	5.2	27.0
Average	**383.4**	**53.0**	**0.16**	**0.7**	**14.2**	**56.6**

Table 9.8 Summary of chemical analysis of STE on Site 2.

The characteristics of the STE varied over the sampling period. This is especially apparent for COD and NH_4-N, and hence total inorganic nitrogen, concentrations. It appears from Figures 9.17 and 9.18 that rainfall events have had an influence on the quality of the STE. While there are periods where the correlation between rainfall

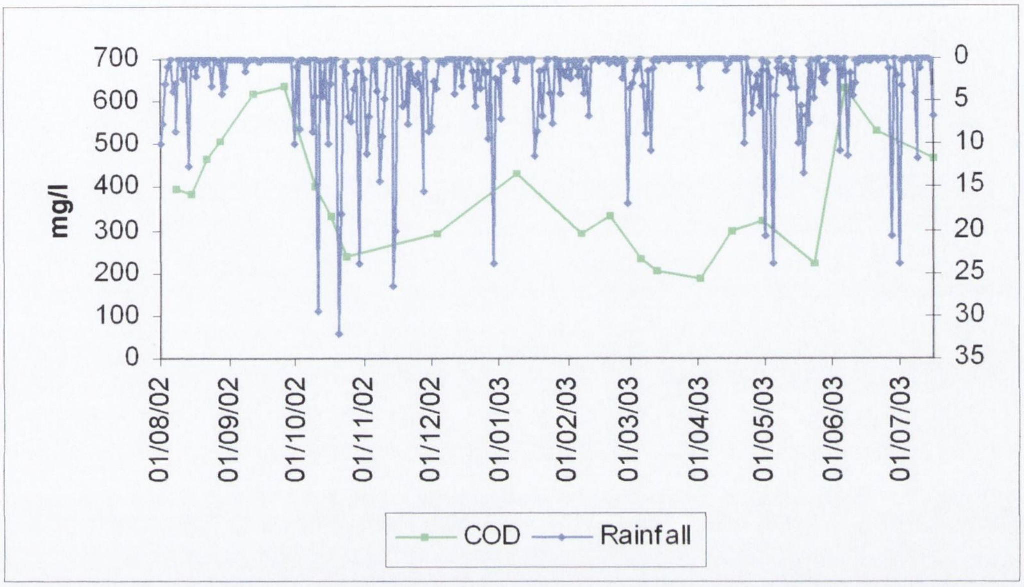


Figure 9.17 The effect of rainfall on STE COD concentration over the sampling period.

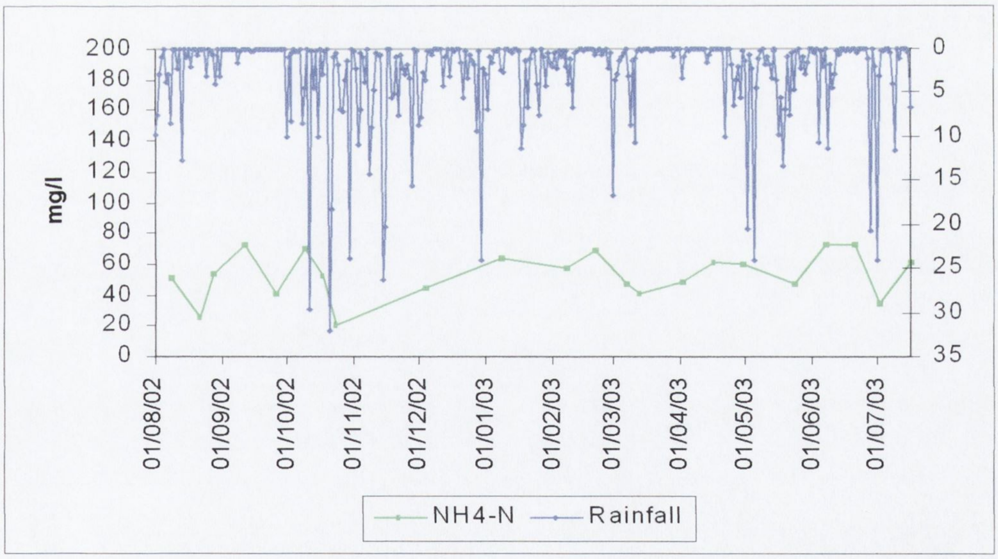


Figure 9.18 The effect of rainfall on STE NH_4-N concentration over the sampling period.

volume and STE concentration appears to be stronger than others, it must also be considered, as outlined in Section 9.2, that there was a contribution from surface water runoff, other than rainfall. The reduction in COD concentration between

09/10/02 and 23/05/03 corresponds to the period of greatest activity within the stables and therefore the period over which it would be expected to have the greatest contribution of surface runoff, other than rainfall.

It is clear from Figure 9.19 and Table 9.9 that the greatest reduction in effluent COD load and concentration occurs above the red depth plane. The COD concentration measured on the red, blue and black sample planes is very similar to the COD concentration measured at point 34, which did not receive STE (Section 9.3.3), suggesting that COD concentration had been reduced to background levels. It also

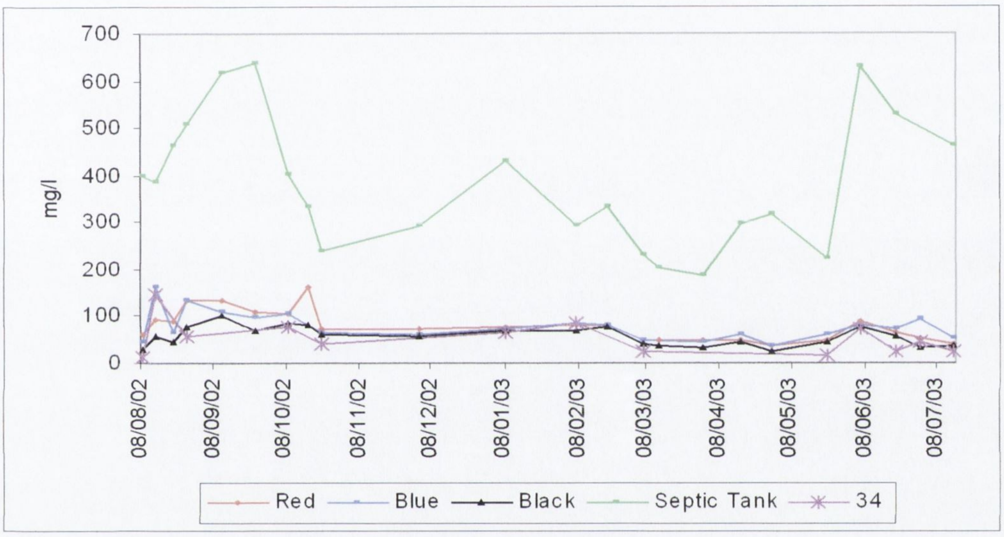


Figure 9.19 Reduction in COD concentration on Site 2.

	Concentration	Load	
	(mg/l)	(g/d)	Load Removal (g/d)
STE	383.4	160.6	
Red Depth Plane	78.0	40.8	119.8
Blue Depth Plane	77.7	42.9	0.0
Black Depth Plane	56.5	32.7	8.1

Table 9.9 Reduction in COD load attributed to the specific treatment steps.

appears that the attenuation performance of the subsoil is not affected by the varying organic load of the STE. When Figure 9.20 is examined it can be seen that the concentration at sample point 16 (20m sample position on trench 2), while greater than that for the red sample plane at 20m, is significantly lower than the STE COD concentration. As sample point 16 has been shown (Section 9.3.1) to be unrepresentative of matrix flow, and is receiving effluent from preferential pathways, it appears that the reduction in COD concentration above the red depth plane is mainly due to aerobic processes within the percolation trenches.

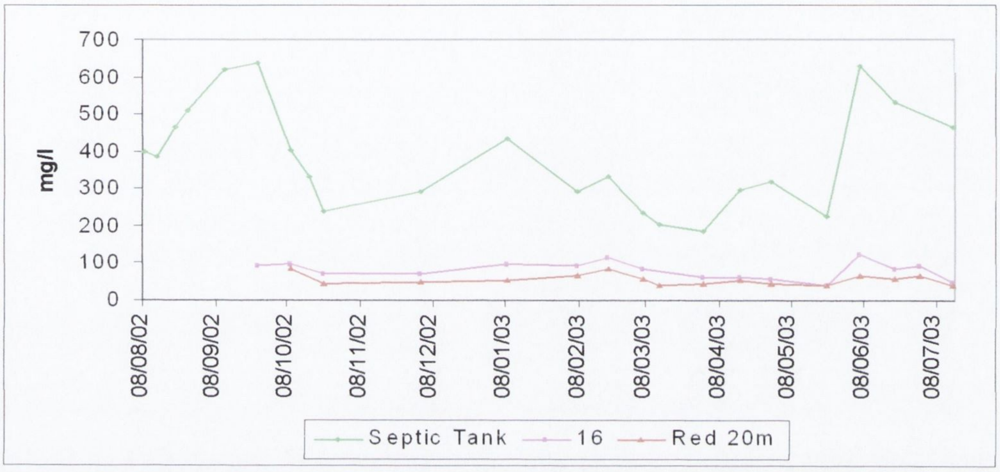


Figure 9.20 Comparison of COD concentrations measured in the STE with those measured at sample point 16 and on the red depth plane at the 20m *sample position.*

The O_2 demand required to achieve this reduction, 305.4mg/1d or 0.12kg/d on average, is present within the distribution gravel in the percolation trench, which is aerated along its full length by means of a ventilation pipe. There was 17.2m^3 of gravel in the percolation area, 4.3m^3 in each trench, which has an approximate porosity of 50% providing 8.6m^3, or 10.54kg, of air space (density of air = 1.226kg/ m^3). Since air is 21% O_2 this means that there was 2.21kg, of oxygen available in the distribution gravel. When the volume of air contained in the percolation pipe is included this increases to 2.26kg O_2 which is regarded as more than sufficient to maintain the concentration gradient across the biofilm and to sustain aerobic processes. Further reduction in COD concentration within the subsoil and incorporated biomat would result from a combination of further aeration, physical straining and biological degradation.

Similarly it was found that the greatest reduction in inorganic nitrogen loading was recorded above the red depth plane (Table 9.10). However, unlike COD, it appears that this reduction is a result of chemical and biological processes within the subsoil matrix between the invert of the percolation trench and the red depth plane, rather than processes within the distribution gravel. If Table 9.5 is revisited it can be seen that the NH_4-N concentration measured at the preferential flow sample points is significantly greater than that measured at the corresponding sample points in the other trenches which are monitoring matrix flow. While there is a decrease in NH_4-N concentration of the percolating effluent with depth, the corresponding increase in NO_3-N concentration is not sufficient to account for this reduction (Table 9.10 and Figures 9.21 and 9.22). It is suggested in the literature, Section 4.4.2, that the saturated conditions directly below the percolation trench promote sorption of NH_4-N by cation exchange which would have the effect of removing NH_4-N from the percolating effluent.

	NH4-N		NO2-N		NO3-N		Total N	
	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)
STE	53.0	22.2	0.2	0.1	0.7	0.3	53.9	22.6
Red	9.9	6.5	0.2	0.1	4.1	2.1	14.2	8.7
Blue	5.1	2.8	0.3	0.2	6.8	3.8	12.2	6.8
Black	5.8	3.4	0.2	0.1	6.7	3.9	12.7	7.4

Table 9.10 Average concentration and loading rate of NH4-N, NO2-N, NO3-N and Total inorganic N measured in the STE and at the three depth planes.

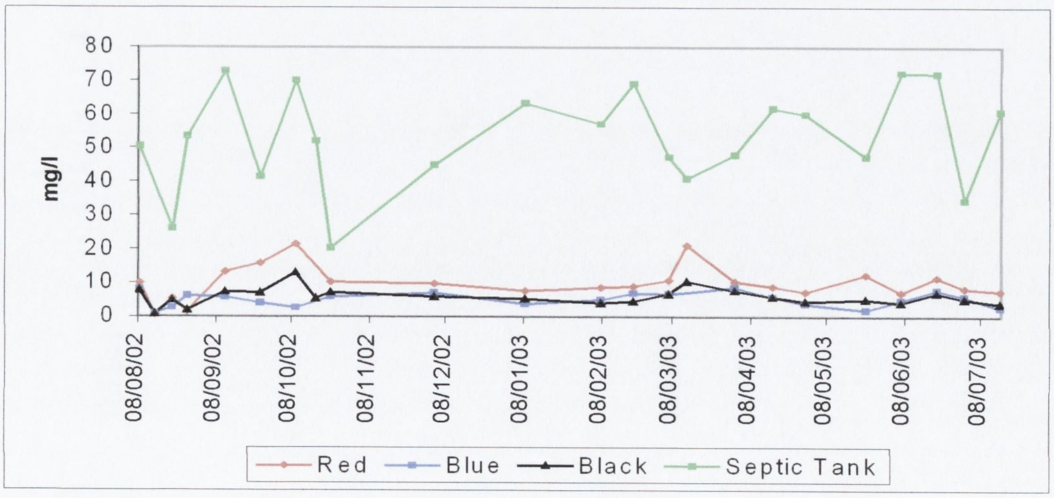


Figure 9.21 NH_4-N concentrations measured on Site 2.

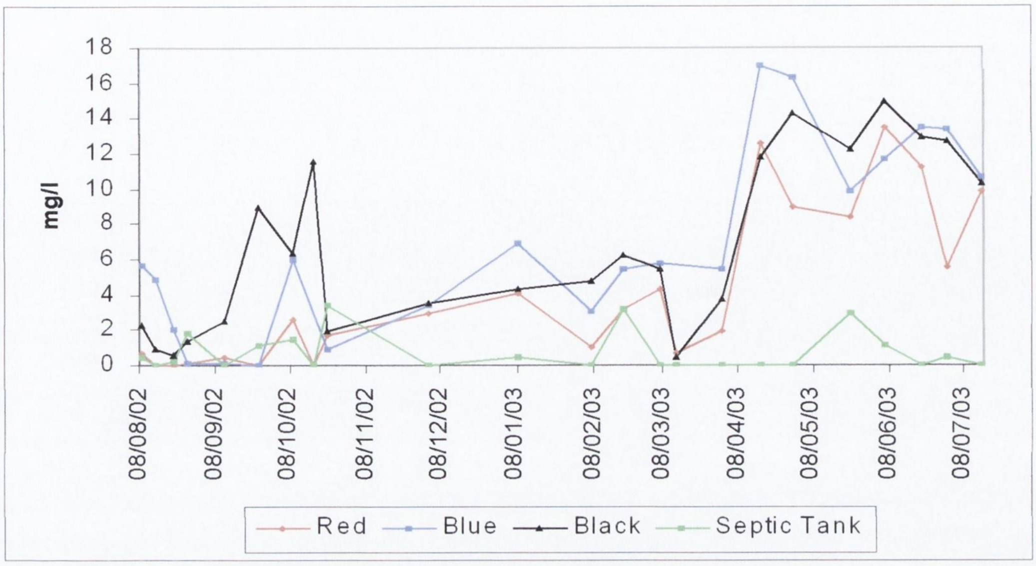


Figure 9.22 NO_3-N concentrations measured on Site 2.

It should be noted that the increase in NO_3-N concentrations in the subsoil from April 2003 corresponds with a period of minimal effective rainfall (Figures 9.11 and 9.12). This would suggest that that the main nitrogen removal mechanism in the subsoil is nitrification followed by denitrification. There is a sufficient organic load in the STE to support this process. The hydraulic load on the system throughout the year probably promoted localised saturated conditions that facilitated denitrification of nitrified NH_4-N. The reduction in the hydraulic load from April 2003, due to the absence of effective rainfall, would have reduced saturated conditions thereby reducing the effect of denitrification which would in turn lead to higher NO_3-N concentrations in the percolating effluent.

As the formation of a biomat along the base of the trenches would lead to increased biological activity it is also possible that some of this reduction in inorganic nitrogen is due to biological uptake. Microbes metabolise carbonaceous materials for synthesis of organic compounds and also to obtain energy. However, they must also obtain sufficient nitrogen to synthesise nitrogen containing cellular components such as amino acids, enzymes and DNA. On average, therefore, soil microbes must incorporate into their cells about one part of nitrogen for every eight parts of carbon (Brady and Weil, 2002).

The disparity between the PO_4-P concentration of the STE analysed on 09/10/02 and the samples analysed on other dates suggests that an error occurred during sample analysis and it was therefore omitted from further analysis. As outlined earlier the PO4-P concentration of the STE varied throughout the year. The high clay content of the subsoil below the percolation area on Site 2, revealed by the particle size analysis contained in Appendix A, suggests that the removal of phosphate from the percolating effluent is controlled by soil adsorption. The large specific surface of clay particles, their generally platy shapes allied with the iron, aluminium and hydrous oxides coating the subsoil clay minerals and magnesium-hydroxy clusters on the weathered surfaces of feromagnesium minerals provide excellent sorption sites (Section 4.2.2). As was the case with COD and inorganic nitrogen reduction it can be seen from Table 9.11 and Figure 9.23 that the greatest reduction in effluent PO_4-P concentration and load occurs above the red depth plane. It is important to note, however, that any given soil only has a limited capacity to fix phosphorus. While it appears that the fixation capacity of the subsoil above the red depth plane has not yet been exhausted it must be remembered that this project was carried out over only a 12-month period and therefore, in time, the subsoil below this depth plane may become dominant in PO_4-P fixation.

	Concentration (mg/l)	Load	
		(g/d)	Load Removal (g/d)
STE	14.2	5.9	
Red Depth Plane	2.2	1.2	4.7
Blue Depth Plane	1.2	0.7	0.5
Black Depth Plane	1.0	0.6	0.1

Table 9.11 Reduction in PO_4-P attributed to the specific treatment steps.

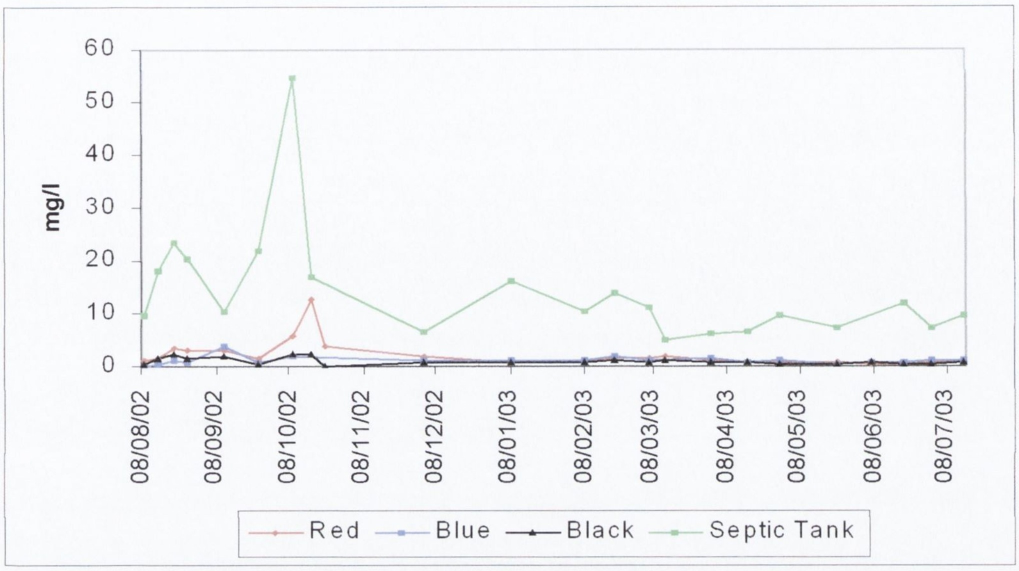


Figure 9.23 PO_4-P concentrations measured on Site 2.

9.3.5 Results of Bacteriological Analysis

Assuming that the results obtained from the bacteriological analysis are representative of the whole percolation area it can be seen that, allowing for the factor of safety introduced due to the sample dilutions, almost complete removal of enteric bacteria is achieved within the system (Table 9.12). Table 9.12 presents the results of sample analysis for *E. coli* on four separate occasions over the research period.

	Number of Samples	Number of samples with concentration (cfu/100ml)			
		<10	10-100	101-1000	>1000
STE	4				4
Red Plane	9	9			
Blue Plane	3	2	1		
Black Plane	12	10	2		

Table 9.12 Concentrations of *E. coli* measured on four separate occasions at Site 2.

The presence of *E.coli* on the black depth plane is confined to samples obtained from points 15 and 27. However, a reduction in bacteriological concentration with time is evident at 27 point with no *E.coli* evident on two sampling days, 15/05/03 and 28/08/03 (Appendix E). It is possible, therefore, that the initial high concentrations experienced may be due to the presence of macropores that facilitated the initial movement of bacteria to the lysimeter porous cup but became blocked with time. If the results of bacteriological analysis for these two latter sample dates only are considered, and allowing for the factor of safety introduced due to the sample dilutions, it was found that complete removal of *E. coli* was achieved within the percolation area on 15/05/03 but that 10 cfu/100ml was detected on the black plane, at point 15, on the 28/08/03.

The results of the bromide tracer test showed that after 8 days all of the sample points from which samples or bacteriological analysis had been taken had received effluent containing the bromide. While the main mechanisms for the removal of bacteria in the subsoil are inactivation/die-off, filtration and adsorption analysis of these results allied with literary evidence of enteric bacterial survival times in subsoils suggests that filtration and adsorption were the dominant removal mechanisms at work.

9.4 Summary

As was the case on Site 1, analysis of flow from the septic tank revealed that the hydraulic loading experienced on Site 2 was less than the design hydraulic load of 20l/m^{2}d. As the average daily flow of 418.8 litres would only require 47m of percolation trench, as opposed to the 80m constructed, it is clear that, with respect to the EPA guidelines, the percolation area was hydraulically over-designed. However, it was also found that at the hydraulic load recorded on Site 2 most of the surface area of the percolation trenches was utilised. It must also be noted that the flow records include a contribution by surface runoff which would have the effect of over-estimating the daily wastewater generation and diluting the STE.

The chemical and bacteriological analysis of soil moisture samples obtained from below the percolation area reveal that most of the effluent attenuation takes place above the red depth plane. Apart from a dilution effect due to effective rainfall of between 7% and 10%, the attenuation was due to physical, chemical and biological activity within the percolation area. The analysis also highlights the fact that the organic load of the STE was sufficient to facilitate the development of a biomat along the trench length, apart from on trench 4, thereby allowing the distribution of effluent along the entire trench length.

10 ANALYSIS OF RESULTS OBTAINED FROM SITE 3

10.1 Introduction

A successful site assessment was completed for Site 3 on 6th February 2003. While the septic tank treatment system and secondary treatment system were commissioned on 15th July 2003 sampling did not begin until 23rd September. This was due to the fact that the sampling instrumentation had to be removed from Sites 1 and 2, cleaned and then installed on Site 3. As the project was under financial constraints as a result of the installation and operational costs of the first year of the study it was desired that the financial outlay for sample analysis for Sites 3 and 4 were maximised. It also was felt, from experienced gained on Sites 1 and 2, that the initial month of percolation area operation equated to a settling in period and sampling during this period would not therefore be as beneficial as sampling for an additional month at the end of the project. The installation of sampling equipment commenced on 25th August and all equipment was in place by 27th August. Sampling continued until 18th March 2004. Chemical analysis of the samples obtained from Sites 3 and 4 was carried out with the assistance of a new technician.

10.2 Analysis of Flow Data

The Omega-SL320 datalogger and ultrasonic level sensor, which had previously been installed on Site 2, were installed in the sump downstream of the distribution box on Site 3. As was the case on Site 4 this sump was required to facilitate the installation of a pump for effluent distribution onto a stratified sand filter. The effluent was again evenly split between the sand filter and the percolation trenches by means of the v-notch weir modification to the distribution box. Flow was monitored from 4th September 2003 to 18th March 2004 (Appendix D). Apart from a few brief periods where there were technical difficulties with the datalogger it represents a continuous record of the flow from the Puraflo® over the sampling period. It should be noted that while on Sites 1, 2 and 4 the datalogger measured flow emanating from the septic tank, the datalogger on Site 3 measured flow emanating from the Puraflo® module.

As was found with flow measurements on the other research sites, the average daily flow to the percolation area and sand filter on Site 3 was less than the typical hydraulic load of 180l/person/day estimated by the EPA (EPA, 2000). While the average daily hydraulic load of 492.1 litres emanating from the Puraflo® (Table 10.1) equates to 123.0 litres/person it includes a contribution by surface runoff, as seen in Figure 10.1, and therefore over-estimates the average daily domestic wastewater generation on site. Prior to the commencement of sampling all of the surface water

	Maximum	Minimum	Average
Daily Flow (l/d)	1754.2	108.4	492.1
Adjusted Daily Flow (l/d)	892.4	160.2	452.1
Total Flow (l/d)*	N/A	N/A	510.6

* equal to the average hourly flow multiplied by 24.

Table 10.1 SE flows measured on Site 3.

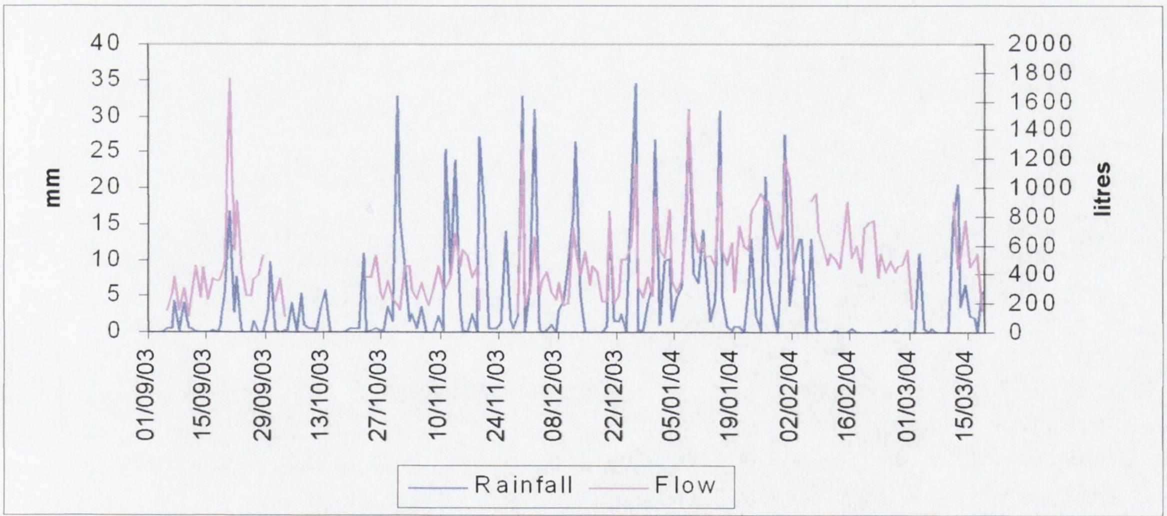


Figure 10.1 Graph of measured flow against rainfall on Site 3.

drains around the house, three at the back and three at the front, were examined to identity whether of not they were connected to the septic tank. It was found that two of the six shores, one at the front and one at the back were in fact connected. One of these two drains received effluent from the household dishwater and washing machine as well as the surface runoff from a gutter that ran the full length of the house, while the second shore received runoff from a small section of roof at the front of the house (approximately 20m^2). As the gutter at the back of the house had a down-pipe at both ends, one of which was connected to a soakaway, it was possible to divert the roof runoff. However, no arrangement was possible to divert the runoff from the front section of the roof. When the flow emanating from the Puraflo® on days of no rainfall was examined, i.e. the adjusted daily flow, it was found that the average flow was 452.1 litres or 113.0 litres/person (Table 10.1). While this provides a more accurate approximation of daily wastewater generation on Site 3, the average daily flow has been used in all load calculations.

It can be seen from Figure 10.2 that the most common SE flow range over the sampling period was 440 to 460l/d. It can also be seen that 80% of flows recorded were between 100 and 620 l/d. The average flow over this range was 375.7l/d.

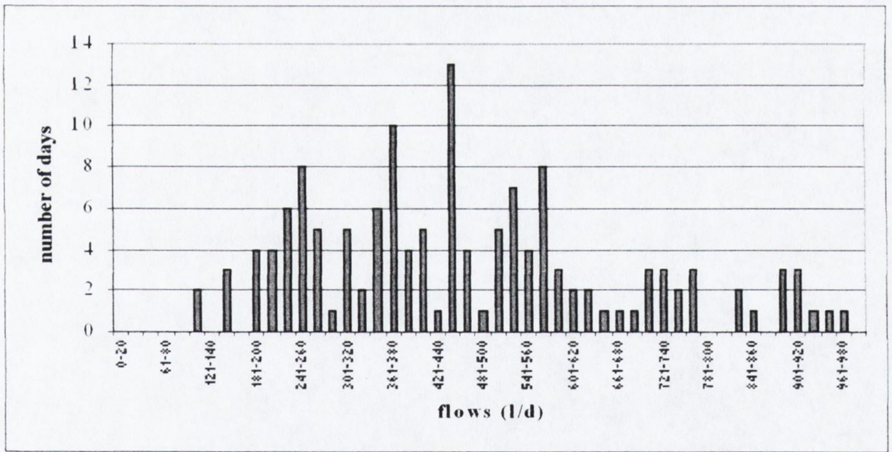


Figure 10.2 Frequency distribution of SE flows recorded on Site 3.

When the flows on days when no rainfall was recorded were examined it was found that 87% of them fell within this bracket and that the average flow for this range was 406.2l/d. When the daily flow profile emanating from the Puraflo® was examined it was found that 90% of the flows fell within the range 0.4 to 3.4l/min (Figure 10.3). As the flowrate on Site 3 was measured at the Puraflo® outlet rather than at the septic tank outlet the shape of the graph in Figure 10.3 differs from the graphs in Figures

8.3, 9.3 and 11.3, which are essentially j-shaped, due to the storage capacity afforded by the Puraflo® system.

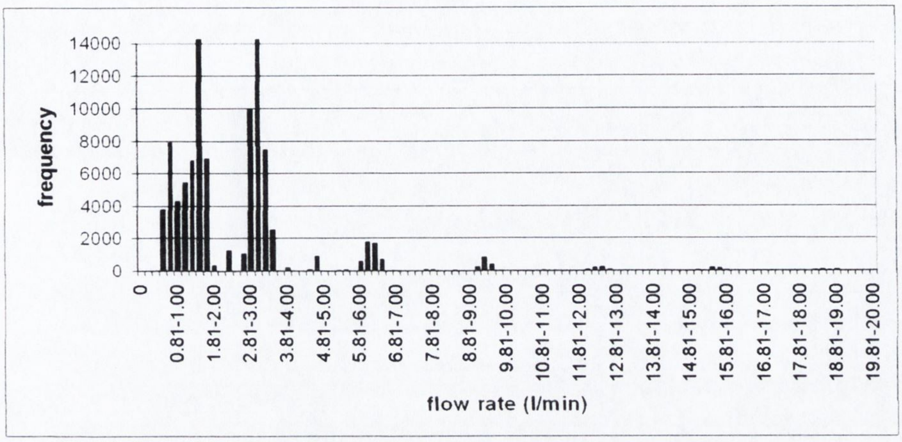


Figure 10.3 Frequency distribution (l/min) of flows recorded on Site 3.

10.3 Results of Analysis of Septic Tank and Secondary Effluent

10.3.1 Results of Chemical Analysis

The results of the chemical analysis of all samples obtained from Site 3 are contained in Appendix E. As can be seen from Table 10.2 results of the analysis of the STE from Site 3 show it to contain high concentrations of organics and nutrients in the form of orthoPO_4-P and NH_4-N. Conversely it can be seen from Table 10.3 that after undergoing treatment by the aerated peat and associated bio-media of the Puraflo® module the STE, or SE as it had become, had a reduced organic concentration and while it still had a high concentration of nutrients the NH_4-N had undergone nitrification. The effect of this nitrification was seen by the reduction in pH from an average of 7.3 for the STE to an average of 6.2 for the SE.

	Concentration (mg/l)				
	COD	NH_4-N	Total N	PO_4-P	Cl
Maximum	1393.0	120.0	122.3	18.4	159.0
Minimum	446.0	28.4	32.1	4.7	42.0
Average	**779.3**	**66.5**	**71.7**	**9.6**	**84.7**

Table 10.2 Summary of the results of chemical analysis of STE on Site 3.

	Concentration (mg/l)				
	COD	NO_3-N	Total N	PO_4-P	Cl
Maximum	370.0	63.4	66.3	12.4	87.0
Minimum	68.0	23.6	31.8	3.6	22.0
Average	**198.9**	**42.0**	**51.1**	**8.1**	**59.7**

Table 10.3 Summary of the results of chemical analysis of SE on Site 3.

The COD concentration of the STE varied throughout the year peaking at 1393mg/l on the 17/12/03. While this variation in the STE COD concentration, which is greatest between 04/11/03 and 20/01/04, is reflected in the SE COD concentration (Figure 10.4), unlike Figure 8.6 the relative size of this increase is greater for the SE. However, an average reduction in COD concentration of 71% for the two peak STE COD concentrations measured over this period suggests, as was the case on Site 1, that the Puraflo® system has the ability to provide a high level of treatment to influent of varying organic quality.

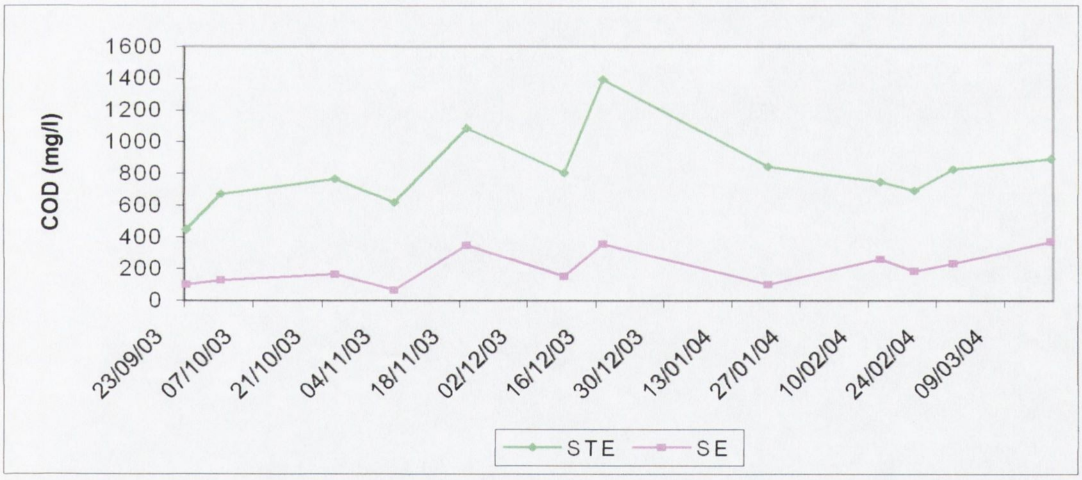


Figure 10.4 Comparison of COD concentration in STE and SE for Site 3.

While Tables 10.2 and 10.3 highlight the effects of nitrification within the secondary treatment system they also show a reduction in the overall inorganic nitrogen concentration between the STE and SE. This reduction, which averages 25% for the sampling period, is highlighted in Figure 10.5. As discussed in Section 8.3.1 it results from modifications to the standard Puraflo® unit that produced saturated conditions at the base of the module. While the effect of denitrification on Site 3 was less than

Figure 10.5 Reduction in total inorganic nitrogen concentration across the Puraflo®.

half that recorded on Site 1 it must be considered that, as the achievement of denitrification is not a goal in the design of the Puraflo® system, it would be reasonable to expect that the effects of denitrification will vary from module to module.

Whereas SE total inorganic N concentration for the year remained fairly constant there was considerable variation in STE total inorganic nitrogen concentration. As 93% of the STE total inorganic nitrogen, on average, was in the form NH_4-N it can be seen from Figure 10.6 that this variation was due to variations in STE NH_4-N concentration over the sampling period. In contrast it can be seen, also in Figure 10.6, that the SE NO_3-N concentration, and hence the SE total inorganic nitrogen concentration, showed a lot less variation over the sampling period. As can be seen from Figure 10.5 there were three occasions on which the total inorganic nitrogen concentration of the STE was reported to be less than that of the SE. As a result the reported effect of denitrification over the sampling period was reduced. It is possible that these are a result of an error in the analysis process. When the results from these dates were therefore omitted it was found that the effect of denitrification led, on average, to a 38% reduction in total inorganic nitrogen concentration.

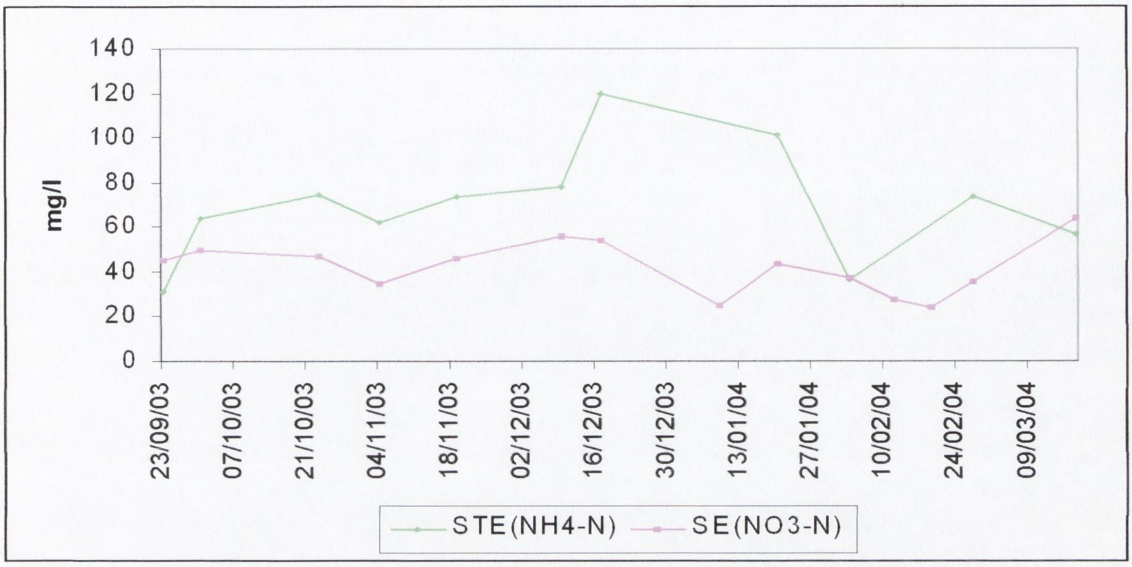


Figure 10.6 Comparison of SE NO_3-N concentration and STE NH_4-N concentration.

Tables 10.2 and 10.3 show that, as was the case on Site 1, there was very little difference in the average STE and SE concentrations of PO_4-P on Site 3. This was to be expected as the Puraflo®, apart from the 10-15% phosphate uptake by micro-organisms, does not have an affinity for PO_4-P removal. As was found in Figure 8.7 there were incidences when the reported SE PO_4-P concentration was greater than the STE PO_4-P concentration (Figure 10.7). This could be due to the mineralisation

Figure 10.7 Comparison of PO_4-P concentration in STE and SE on Site 4.

of organic phosphate within the Puraflo® or the result of an error within the sampling procedure that was multiplied in the reported concentration as a result of the necessity to dilute samples.

There was an average reduction of 30% in Cl concentration across the Puraflo® on Site 3 (Figure 10.8). Due to the reduction in Cl concentration measured across the Puraflo® system on Site 1 it was decided to analyse both filtered and unfiltered samples of STE for Cl on Sites 3 and 4. When this was done for samples obtained from Site 3 it was found that, on average, there was a 16.8% reduction in Cl concentration with filtration. Even with filtration the STE samples appeared more turbid than the SE samples and it is therefore possible that finer particulate matter was retained by the Puraflo® than by the 1.2μm sample paper. It is possible that these finer particles were removed by the filtering action of the peat and associated bio-media or that the saturated conditions at the base of the module provided quiescent conditions and sufficient residence time for the settling out of particulate matter.

Figure 10.8 Comparison of Cl concentration in STE and SE on Site 4.

When Cl concentration in the STE was compared against rainfall over the project duration it was found that even though STE flow included a contribution of surface runoff (Section 10.2) the volume of this contribution was insufficient to influence STE quality (Figure 10.9) and only helped to increase the hydraulic load closer to design values. This was due to the buffering capacity of the septic tank.

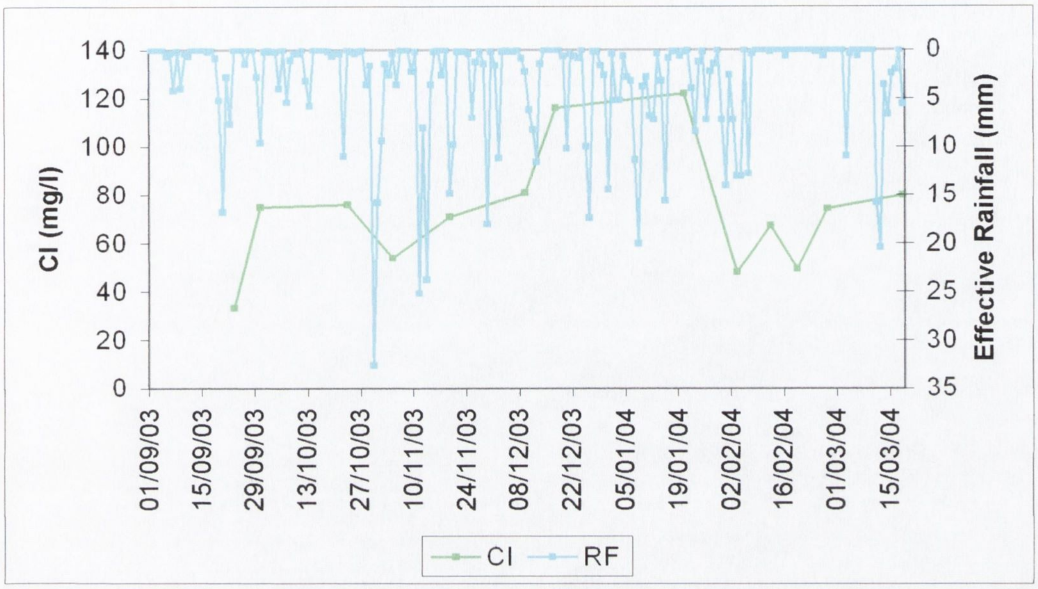


Figure 10.9 STE Cl concentration graphed against rainfall for the project duration.

10.3.2 Results of Bacteriological Analysis

Table 10.4 shows that even, with the high bacterial removal efficiency associated with the installation of a Puraflo® system, the presence of enteric bacteria in SE samples analysed on two occasions over the project duration highlighted the requirement of SE to undergo further treatment in the subsoil prior to discharge to groundwater.

Date	Bacteria	Concentration (cfu/100ml)		% Removal
		STE	SE	
02/12/03	Total coliforms	21,600,000	2,000	99.99
02/12/03	*E. coli*	616,000	58	99.99
02/12/03	*Enterococci*	2,696	2	99.93
02/12/03	Faecal coliforms	760,000	72	99.99
11/04/04	Total coliforms	4,110,000	5,480	99.87
11/04/04	*E. coli*	24,190	710	97.06

Table 10.4 Reduction in bacterial concentration across the Puraflo® on Site 3.

10.4 Results of Analysis of Soil Moisture Samples

10.4.1 Method of Analysis

As was the case for analysis of results for the Sites 1 and 2, the subsoil below the percolation area on Site 3 was divided into three sections: 0.to 0.4m (red), 0.4 to 0.8m (blue) and 0.8m to 1.2m (black). Three nominal depths of 0.2m, 0.6m and 1.0m were again defined to represent the location of the lysimeter tips within these sections. Unlike the other sites there was no reclassification of lysimeters required at Site 3 (Table 7.2).

Examination of the results of soil moisture samples for Cl revealed that, of the two methods outlined in Section 8.4.1, the depth average method was the more representative in highlighting the behaviour of the SE in and below the percolation trenches (Figures 10.10 and 10.11). This would suggest that, as appeared to be the case on Site 1, the reduction in STE organic load brought about by the installation of

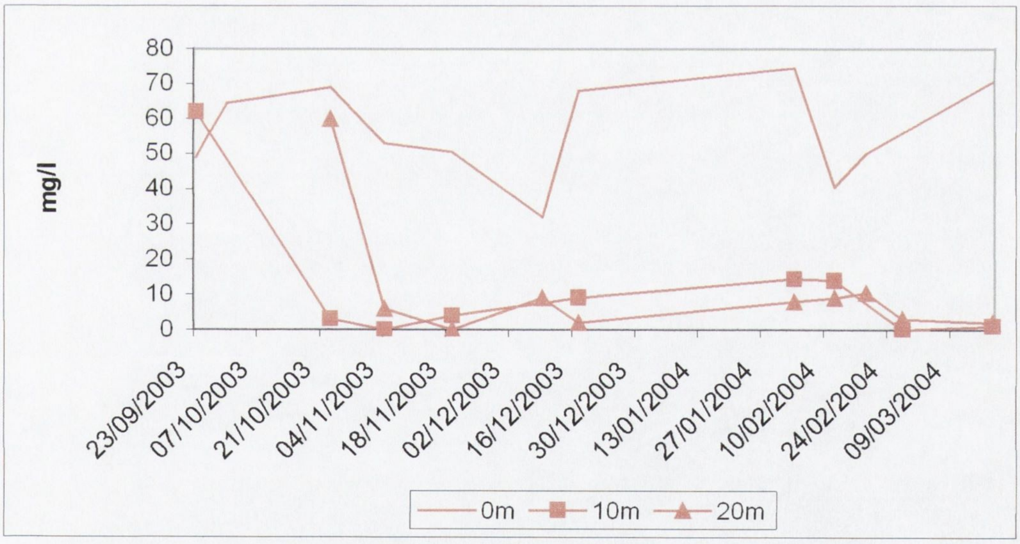


Figure 10.10 Cl concentration measured on the red depth plane at the three sample positions on Site 3.

a secondary treatment system inhibited the formation of a biomat along the base of the percolation trenches thus confining effluent loading to less than the first 10m of both trenches. As the Cl concentration for the three depth planes at the 0m sample position are very similar it would suggest that the influence of dilution on effluent attenuation between these planes was small (Figure 10.11).

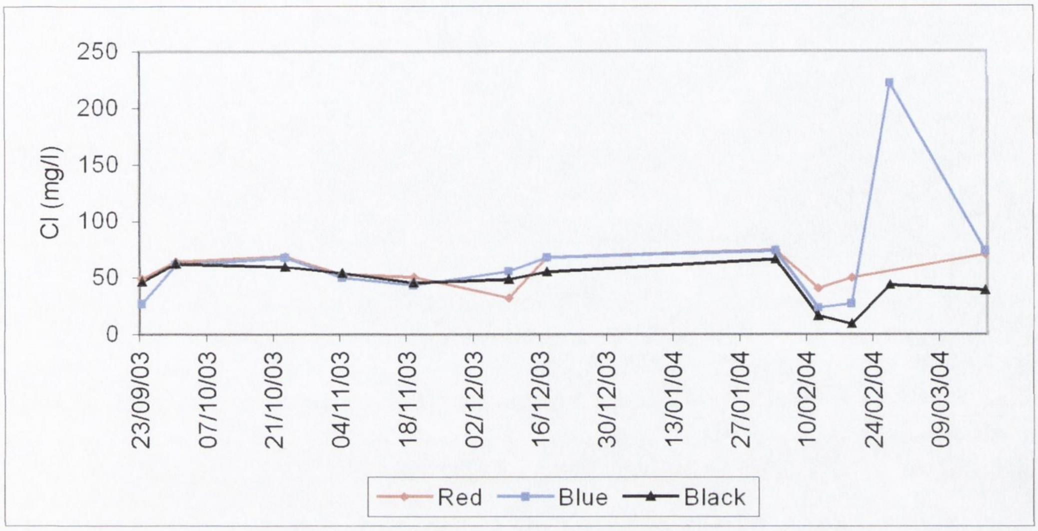


Figure 10.11 Cl concentration measured on the three depth planes at the 0m sample position on Site 3.

While the high Cl concentrations measured on the red depth plane at 0m over the sampling period (Figure 10.10) are reflected by high NO_3-N concentrations, as would be expected, the initial high Cl concentrations measured at the 10m and 20m sample positions are not. It appears, therefore, that these are due to sample contamination. It can be seen from Figure 10.11 that there is a drop off in Cl concentration, to background concentrations (9mg/l) in the case of the black depth plane, on the 12/2/04 and 19/2/04. This reduction is also reflected in the Cl concentration of the SE. There is also a reduction in other parameters on these two dates. It is not clear whether this reduction in overall effluent quality is genuine or is due to the analysis being carried out in the absence of the resident analysts on those dates. It can also be seen from Figure 10.11 that the Cl concentration on the blue plane on 27/2/04 is greater than twice that measured at any time on the blue depth plane and about three times that measured in the SE. It was therefore regarded as egregious and omitted.

10.4.2 The Effect of Dilution on Effluent Attenuation

The quantification of effluent dilution within the subsoil and estimation of the zone of contribution proved problematic for Sites 3. As can be seen from Figure 10.12 sample analysis for Cl on Site 3 was not as successful as for Sites 1 and 2. This is thought to result from a combination of it being a problematic test, as outlined in Section 7.4.2, and a change in personnel in the laboratory. It was decided, for the purpose of the quantification of effluent dilution by effective rainfall, to compare Cl concentrations between the Puraflo® and the black depth plane only and estimate the dilution on the red and blue depth planes from this.

Figure 10.12 Results of sample analysis for Cl on Site 3.

While a raingauge was installed on site no other local meteorological data were available and it was therefore necessary to use data obtained from the weather station on Pollardstown Fen approximately 52km north-west to calculate the potential evapotranspiration. While this distance was a concern with respect to the suitability of the data it was decided that as local rainfall data was available and as the Agroclimatic Atlas of Ireland (AGMET, 1996) showed evapotranspiration to be similar for the two areas the Pollardstown data would be useful in estimating the contribution of effective rainfall to effluent dilution. As only limited meteorological data was available from the station at Pollardstown Fen potential evapotranspiration was calculated using the Hargreaves method (Equation 8.1). The rainfall at Site 3 between 01/09/03 and 18/03/04 was measured as 640.4mm and the effective rainfall was calculated to be 414.9mm. Although the representation of effluent dilution was not as successful for Site 3 as it was for Sites 1 and 2, due to problems associated with the analysis procedure, it still appears from Figure 10.13 that the dilution effect was greatest over the period of sustained effective rainfall between 19/11/03 and 03/02/04. As there was little effective rainfall, and hence dilution, outside this period it was decided to let the difference in Cl concentration between Puraflo® and black depth plane samples obtained on these sample dates be equal to zero. These data were then used to approximate the effect of dilution on effluent attenuation and also

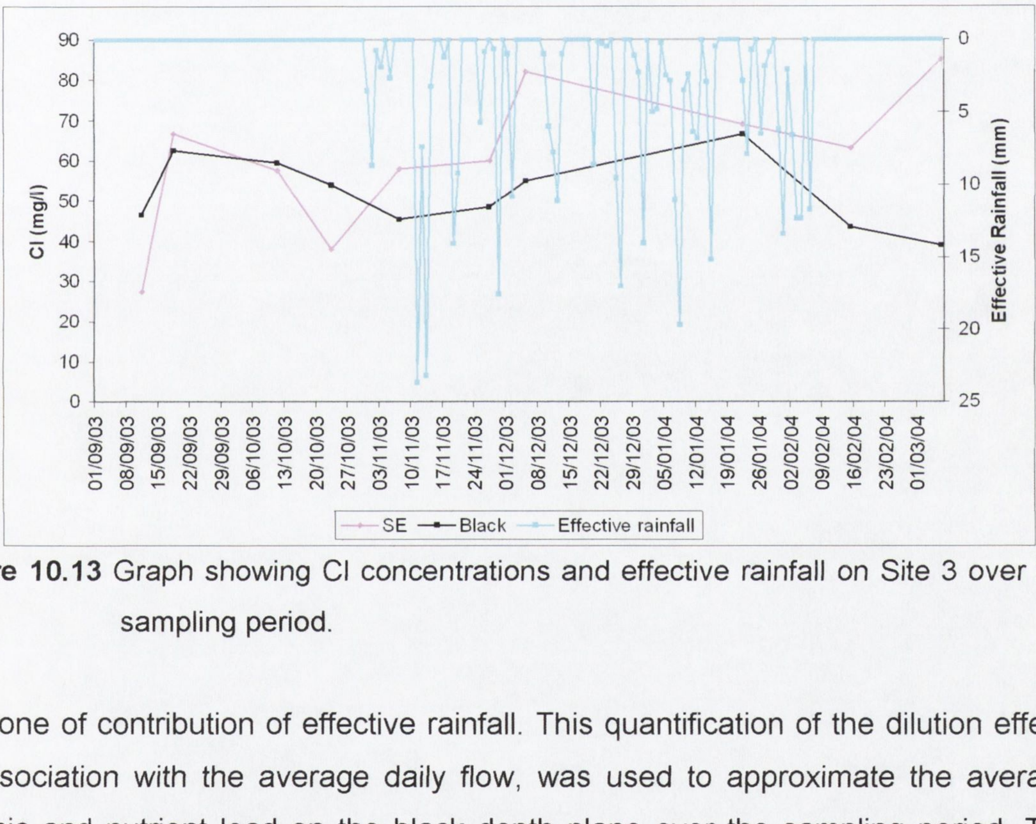


Figure 10.13 Graph showing Cl concentrations and effective rainfall on Site 3 over the sampling period.

the zone of contribution of effective rainfall. This quantification of the dilution effect, in association with the average daily flow, was used to approximate the average organic and nutrient load on the black depth plane over the sampling period. The average load of each parameter on the red and blue depth planes was guestimated, on consultation of the analysis from Sites 1 and 2, by assuming a 2% difference between sample planes.

When the reduction in Cl concentration between the Puraflo® and the black depth plane between 19/11/02 and 03/02/04 was calculated (Table 10.5) it was found that

Date	% reduction in concentration	Equivalent contribution of effective rainfall (l)
19/11/03	21.6	0.29
09/12/03	19.2	0.25
17/12/03	32.9	0.52
03/02/04	3.6	0.04
Average	**19.3**	**0.28**

Table 10.5 Calculation of the contribution of effect rainfall to effluent dilution on the black depth plane.

the effect of dilution was equivalent to, on average, the addition of 0.28 litres of effective rainfall, which had an average Cl concentration of 3 mg/l, per litre of effluent (or 19.3% reduction in concentration). However, when the effect of dilution over the entire sampling period was calculated, letting the effect of effective rainfall on dilution equal to zero on the other sampling dates, it was equal to 0.11 litres per litre of effluent, or a 7.7% reduction in concentration. Taking a 2% reduction in the affect of dilution between the black and blue depth planes and between the blue and red depth planes the effect of dilution on the red and blue depth planes was then estimated as resulting in a 3.7% and 5.7% reduction in concentration, respectively. By dividing the yearly contribution of effective rainfall to dilution by the yearly effective rainfall it was possible to estimate that the zone of contribution of effective rainfall. This was equated to an approximate zone of contribution of 10.9 m^2, 17.5 m^2 and 24.1m^2 for the red, blue and black depth planes respectively. While it was clear from Section 10.4.1 that less than 10m of percolation trench was utilised, there was no way to approximate the exact length of trench over which the effluent was distributed and so it was estimated, somewhat arbitrarily, as 5m. Therefore, taking the trench width of 0.45m this equates to a zone of contribution of approximately 0.3m, 0.5m and 0.7m on all sides of each trench for the red, blue and black depth planes, respectively.

It appears from Figures 10.14 and 10.15 that, as was the case on Sites 1 and 2, the soil moisture tension readings corroborate the chemical analysis in that they

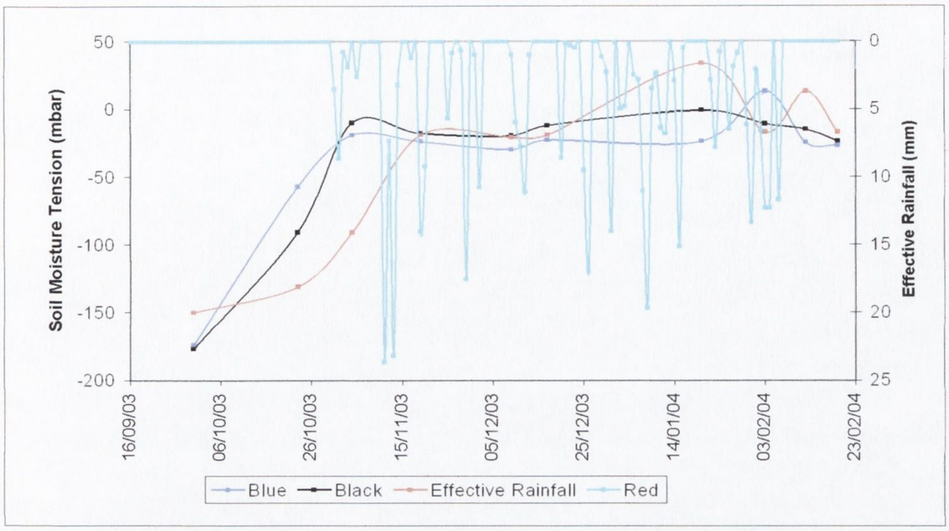


Figure 10.14 Soil moisture tension plotted against effective rainfall for the 20m sample position on Site 3.

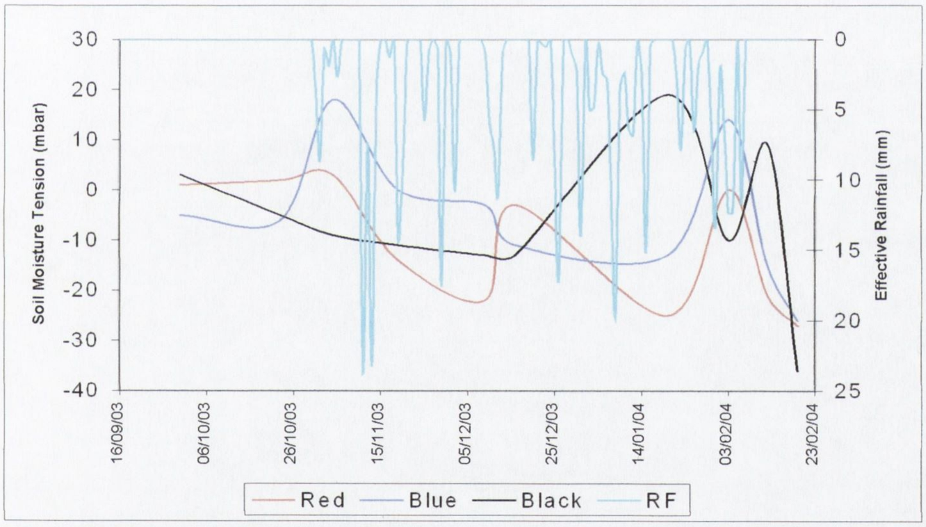


Figure 10.15 Soil moisture tension plotted against effective rainfall for the 20m sample position on Site 3.

show that it is physical, chemical and biological processes, rather than dilution, which are the more prominent effluent attenuation processes. It appears from Figure 10.14 that the tensiometers installed at the 20m sample position, where no effluent was recorded, react to the variation in effective rainfall over the sampling period while the tensiometers installed at the 0m sample position appear to be uninfluenced by effective rainfall. This suggests that it is the percolating effluent that influences the change in soil moisture tension and that the contribution of dilution to effluent attenuation is small, as calculated by the chemical analysis.

10.4.3 Results of Chemical Analysis

Whereas the presence of a minimum of 0.6m of unsaturated subsoil below the invert of the percolation trenches is one of the requirements to satisfy the EPA recommendations for site suitability, lysimeters were also installed to a greater depth. This allowed the benefits, if any, of a greater thickness of unsaturated subsoil below the percolation area to be examined.

The reduction in COD concentration of the domestic wastewater effluent with depth is small when compared to the reduction that takes place across the Puraflo® unit (Table 10.6). This reduction in organic load has had the effect of concentrating the effluent over less than half the percolation area by inhibiting the formation of a biomat along the subsoil-effluent interface. The similarity between the COD results obtained from Sites 1 and 3 (Table 8.8), especially between the SE and the red depth plane given the very different nature of the subsoils, again suggests that the majority of COD reduction within the percolation area occurs above the red depth plane or within the distribution gravel. In fact it can be seen from Figure 10.16 that, because of the similarity between the COD concentration on the red depth plane at all three sample positions, almost complete attenuation of the organic content of the domestic wastewater has been achieved within the system.

Sample Position	Concentration	Load	
	mg/l	g/d	Load Removal (g/d)
STE	779.3	191.8	
SE	198.9	48.9	142.9
Red depth Plane	109.3	28.0	20.9
Blue Depth Plane	89.5	22.0	6.0
Black Depth Plane	89.7	22.0	0.0

Table 10.6 Removal of COD load attributed to the specific treatment steps.

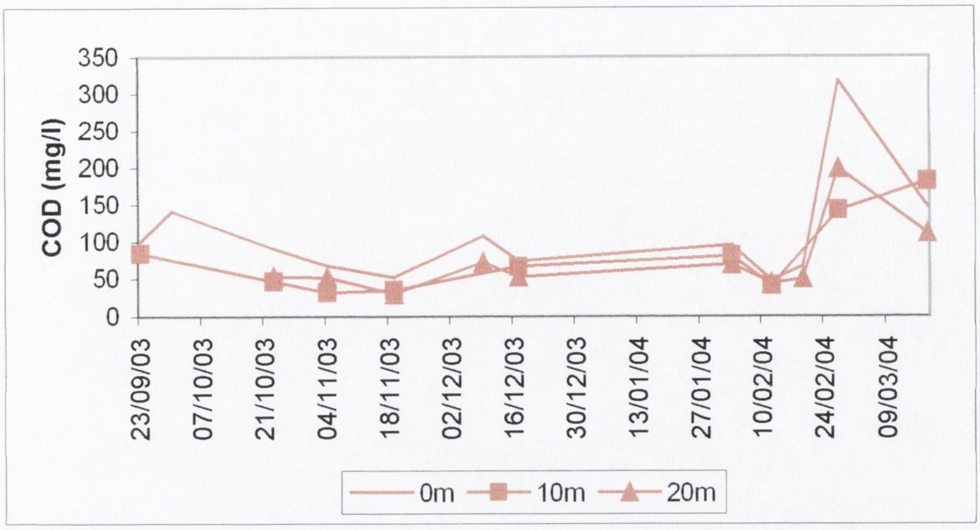


Figure 10.16 COD concentrations measured on the red depth plane at the three sample positions.

There was a small reduction in the total inorganic nitrogen concentration and load with subsoil depth (Table 10.7 and Figure 10.17). The reduction in NH_4-N load with depth was not reflected by a corresponding increase in NO_3-N load. In fact it can be seen that there was a decrease in NO_3-N concentration with depth (Figures 10.18 and 10.19), reflected by an increase in pH from 6.0 on the red plane to 6.2 on the blue plane to 6.6 on the black plane suggesting denitrification. However it is more likely that his reduction in both NO_3-N and NH_4-N is due to dilution. This is because the reduction in organic load of the STE, to near background concentrations, would inhibit denitrification. It has also been shown that the reduction in effluent concentration as a result of dilution was approximated as 3.7%, 5.7% and 7.7% for

Depth Plane	NH_4-N		NO_2-N		NO_3-N		Total N	
	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)
SE	6.5	1.6	0.2	0.05	42.0	10.3	48.7	12.0
Red	3.8	1.0	0.4	0.10	50.6	12.9	54.8	13.1
Blue	3.0	0.8	0.3	0.08	48.5	12.7	51.8	13.6
Black	2.1	0.6	0.1	0.03	45.6	12.1	47.8	12.7

Table 10.7 Average concentration and loading rate of NH_4-N, NO_2-N, NO_3-N and Total inorganic N measured on the red and blue depth planes.

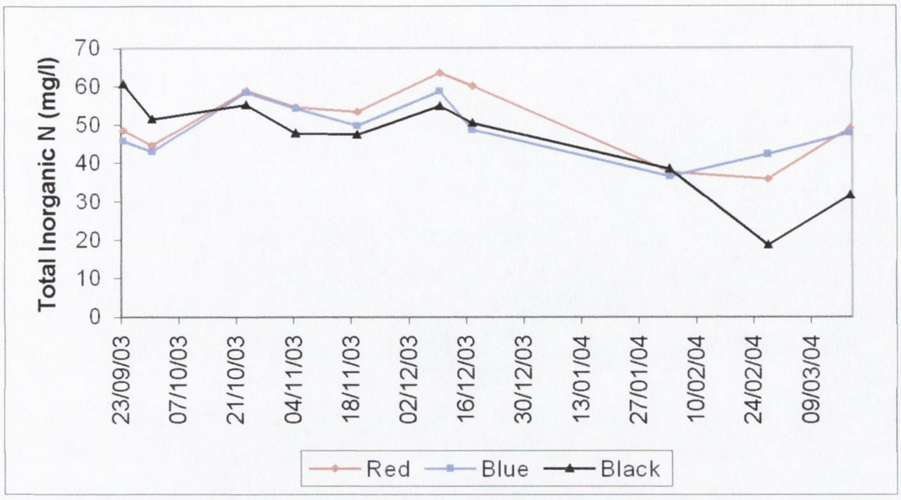


Figure 10.17 Average total inorganic N concentrations at the 0m sample position.

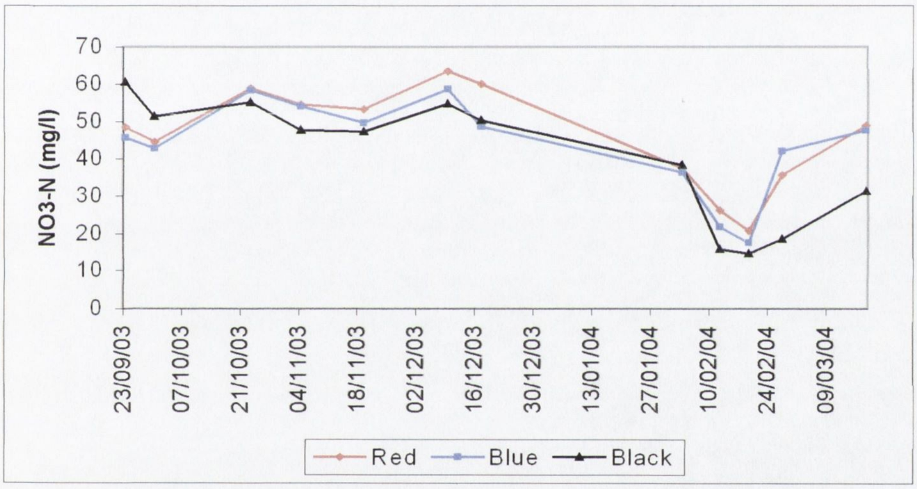


Figure 10.18 Average NO_3-N concentrations measured on Site 3.

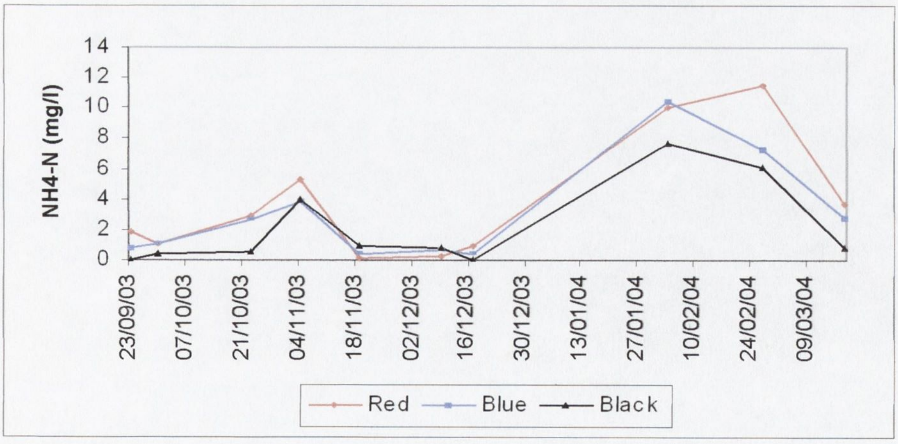


Figure 10.19 Average NH_4-N concentrations measured on Site 3.

the red, blue and black depth planes, respectively. Figures 10.17, 10.18 and 10.19 also show that whereas total inorganic nitrogen for the red and blue depth planes remained stable throughout the sampling period there appeared to be a fall off in nitrification over the penultimate month of sampling. It can be seen from Figure 10.14 that this coincides with a period of increased saturation.

Examination of the subsoil exposed by the trial hole and the results of particle size analysis of three subsoil samples taken at 1.0m, 1.5m and 2.0m below ground level resulted in a uniform classification of the subsoil below the invert of the percolation trenches (Appendix A). However, as can be seen from Figure 10.20 and Table 10.8 the greatest reduction in PO_4-P load occurred between the blue and black depth planes. The low clay and high sand content of the subsoil would suggest a reduced

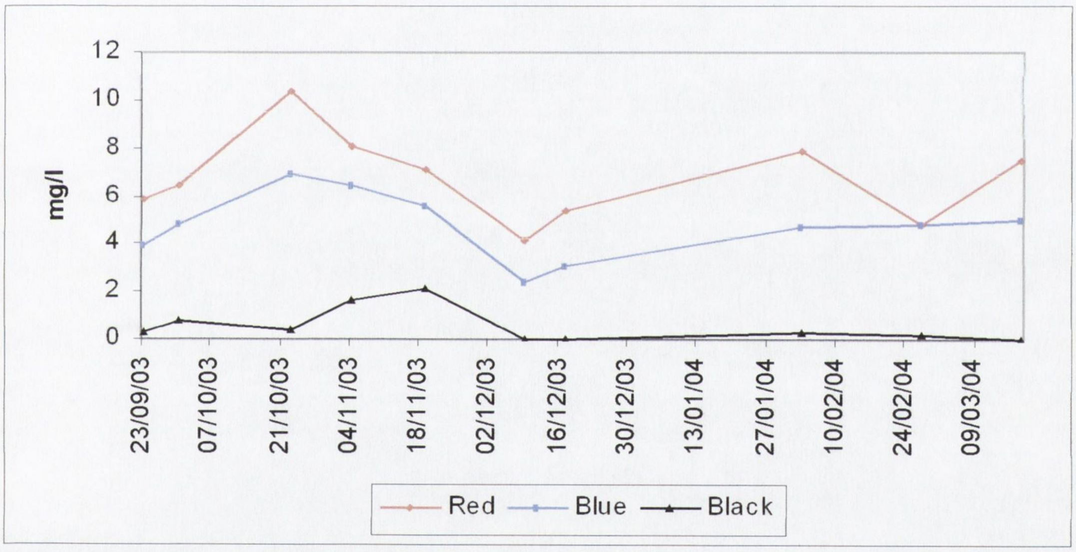


Figure 10.20 PO_4-P fixation within the subsoil matrix.

Sample Position	Concentration mg/l	Load g/d	Load Removal (g/d)
SE	8.1	2.0	
Red Depth Plane	6.8	1.7	0.3
Blue Depth Plane	4.8	1.3	0.4
Black Depth Plane	0.6	0.2	1.1

Table 10.8 Removal of PO_4-P load attributed to the specific treatment steps.

capacity for phosphate fixation and it can be seen that there is only a small reduction in load between the SE and the red depth plane and also between the red and blue depth planes. As the soil moisture samples were acidic (pH 6.0 – 6.6) this suggested that the ability of the soil to fix PO_4-P depended not only on the clay content but also on the presence of Al, Fe and/or Mn as dissolved ions, oxides or hydrous ions. In the absence of X-ray diffraction analysis of the subsoil samples it was not possible to determine whether there was a change in mineralogy within this uniformly classified subsoil layer which would increase affinity for PO_4-P removal.

10.4.4 Results of Bacteriological Analysis

Due to financial constraints it was only possible to send two sets of samples from Site 3 for bacteriological analysis. Both sets of samples were obtained from Trench 1 only. Assuming that these samples are representative of the sample position from which they were obtained it can be seen from Table 10.9 that, allowing for the factor of safety introduced due to sample dilutions, there was complete removal of enteric bacteria by the black depth plane. Analysis of the first sample which was obtained on the 02/12/04 showed the presence of enteric bacteria on the red and black depth planes at the 10m sample position and on the blue and black depth planes at the 20m sample position. However, as the chemical analysis highlighted the absence of SE at these locations it suggests a non-anthrophic source.

	Number of Samples	Number of samples with concentration (cfu/100ml)			
		<10	10-100	101-1000	>1000
SE	2		1	1	
Red Plane	2	2			
Blue Plane	2	2			
Black Plane	1	1			

Table 10.9 Concentration of enteric bacteria measured at the 0m sample position on Site 3.

10.5 Summary

As the T-value of 52 measured on Site 3 was outside the permissible range, and in the absence of a specified loading rate, it was necessary to use the recommended loading rate of $25l/m^2d$ for soils with a T-value between 21 and 50 during the construction phase to determine the area of percolation trench required (EPA, 2000). When this was applied to the adjusted average daily flow of 452.1 litres measured on Site 3, which again was less than the EPA estimation for wastewater generation of 180l/pd, it was found that 40m of percolation trench rather than the recommended 64m would be required. As the SE was evenly split between a stratified sand filter and the percolation area this meant that, based on the hydraulic loading rate, only 20m of percolation trench was required. However, as the Cl analysis showed, less than 10m of each percolation trench was utilised and it is clear, therefore, that due to the secondary treatment step and associated impedance of biomat formation that a loading rate greater than the design loading rate was experienced.

As was the case on Site 1 the installation of a Puraflo® system greatly reduced the bacterial and organic load on the percolation area and also brought about a degree of denitrification. This reduction in organic load also had the effect of inhibiting biomat formation and thus the distribution of effluent over a larger trench area.

The analysis of soil moisture sample results was difficult due to the location of the lysimeters. The loading rate on the red and blue depth planes was estimated, therefore, by guestimating a 2% reduction in concentration between each depth plane due to dilution. When results of the chemical analysis were examined it was found that the greatest reduction in COD load occurred between the SE and the red depth plane while the greatest reduction in PO_4-P load occurred between the blue and black depth planes. It was also found that while there had been complete removal of enteric bacteria and a reduction of COD to almost background levels the effluent still contained a substantial nutrient load (Table 10.10).

Sample Position	Load (g/d)		
	COD	Total N	PO_4-P
SE	48.9	12.0	2.0
Red	28.0	13.1	1.7
Blue	22.0	13.6	1.3
Black Depth Plane	22.0	12.7	0.2

Table 10.10 Average load measured in SE and on the three depth plane.

11 ANALYSIS OF RESULTS OBTAINED FROM SITE 4

11.1 Introduction

A successful site assessment was completed for Site 4 on 13th May 2003. While the septic tank treatment system was commissioned on 9th July 2003 sampling did not begin until 23rd September. This was due to the fact that the sampling instrumentation had to be removed from Sites 1 and 2, cleaned and then installed on Site 4. The lag between commissioning and sampling was also affected by the experience gained from Sites 1 and 2 and financial constraints as outlined in Section 10.1. As the dwelling on Site 4 had just been completed and surface water had, on the word of the builder, been diverted to a soakaway, diversion works were not required. The installation of sampling equipment commenced on 27th August and all equipment was in place by 29th August. Sampling continued until the 18th of March 2004.

11.2 Analysis of Flow Data

The Lascar Easylog datalogger with attached ultrasonic probe, which had previously been installed on Site 1, was installed in the sump downstream of the distribution box on Site 4. The sump was required to facilitate the installation of a pump for effluent distribution onto a stratified sand filter which was being monitored as part of a parallel study. As the effluent was evenly split between the sand filter and the percolation trenches measured flows had to be doubled to give the actual flow emanating from the septic tank. Flows were measured from 15th September 2003 until 19th March 2004 (Appendix D). Apart from a few brief periods where the storage capacity of the datalogger had been reached it represents a continuous record of flow from the septic tank over the sampling period.

The average daily flow recorded on Site 4 (Table 11.1) equates to a hydraulic load of 80.4l/person.day which is again less than the typical daily hydraulic load of 180l/person specified by the EPA (EPA, 2000). As can be seen from Table 11.1 the calculation of the average daily flow included incidents of zero flow. Unlike the flow calculations for Site 1 these were not excluded as they were isolated incidents, i.e. did not represent prolonged periods of absence such as holidays.

	Maximum	Minimum	Average
Daily Flow (l/d)	1054.0	0.0	329.2
Total Flow (l/d)	N/A	N/A	334.1

Table 11.1 STE flows measured on Site 4.

While the dwelling at Site 4 had only been completed in June 2003 and the builder had guaranteed separation of surface runoff and domestic wastewater it appears from Figure 11.1 that this was not the case. While there is variation in the STE flow at times of low rainfall, and also in the absence of rainfall, it appears that there seemed to be an enhancement of peak flows during rainfall events. Allied to this is the fact that flow was observed at the distribution box during rainfall events when there was no activity within the dwelling. This means that the measured flow includes a contribution by rainfall and therefore overestimates the average domestic wastewater production. When the STE flow was averaged over the days when no rainfall was recorded it was found to be 312.2l/d or 78.1l/pd which would be a better estimate of the average daily wastewater production.

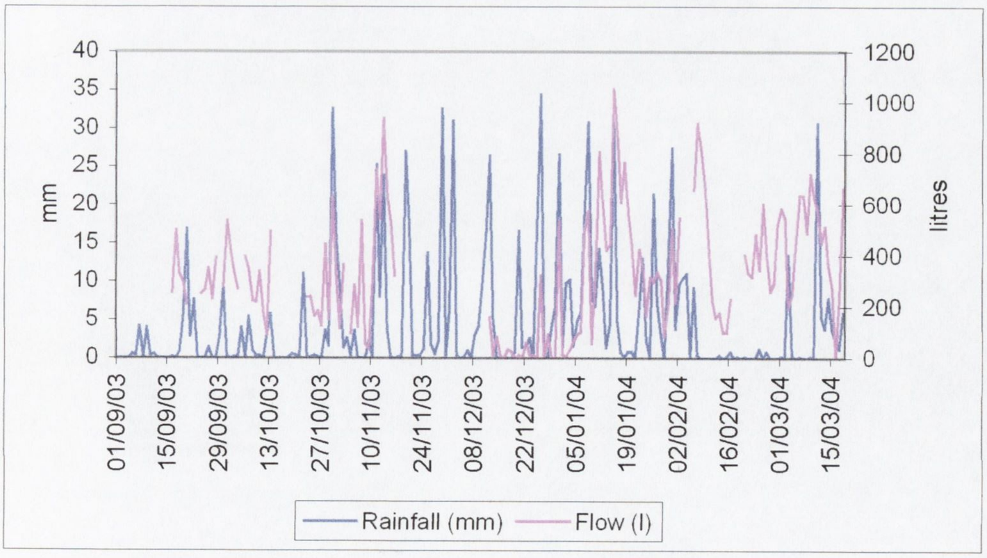


Figure 11.1 Graph of measured flow against rainfall in Site 4.

There were 15 days over the research period when zero flow was recorded. When these records are excluded the frequency distribution of the recorded flow rates (Figure 11.2) shows 70% of the flows recorded fall in the range 20l/d – 480l/d. The average flow over this range was 215.2l/d. When the daily flow regime emanating from the septic tank was examined in greater detail it was found that 80% of flows were in the range 0 to 1.8 l/min (Figure 11.3). As the same modification used on Sites 1 and 2 was used to achieve even distribution within the distribution box, and was shown to be effective over this flow range, it is reasonable to assume that even distribution was achieved.

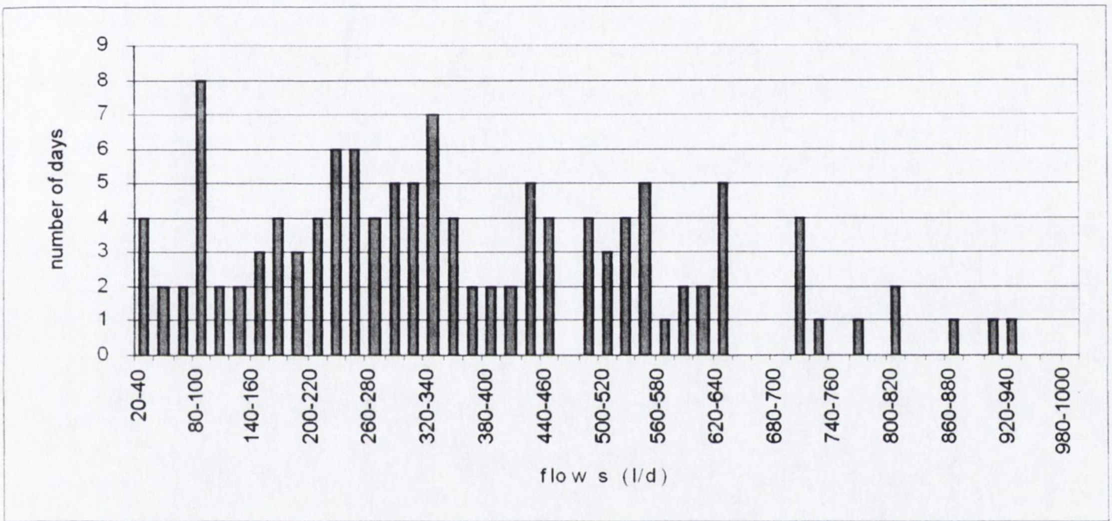


Figure 11.2 Frequency distribution of STE flows recorded on Site 4.

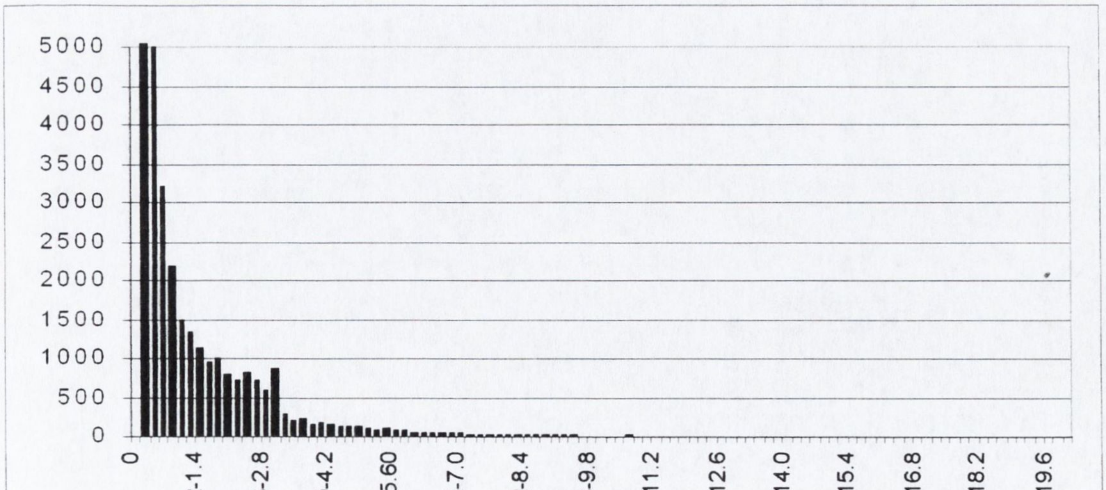


Figure 11.3 Frequency distribution (l/min) of flows recorded on Site 4.

11.3 Results of Analysis of Septic Tank and Soil Moisture Samples

11.3.1 Method of Analysis

To enable the representation of effluent attenuation by the subsoil below the percolation trench the subsoil was again divided into three sections: 0 to 0.4m, 0.4 to 0.8m and 0.8 to 1.2m. Three nominal depths of 0.2m (red), 0.6m (blue) and 1.0m (black) where then defined to represent the middle of these sections and thus the location of the lysimeters porous cups. As can be seen from Table 7.2 it was not possible to install the black lysimeter at the 20m sample position to the desired depth and it had to be reclassified as a blue lysimeter prior to chemical and bacteriological analysis.

The results of soil moisture samples for Cl were again used to determine which of the two methods outlined in Section 8.4.1, planar average or depth average, was the better method for representing the distribution of STE within the percolation area. When the average Cl concentrations at the three sample positions were graphed (Figure 11.4) it suggested that the effluent was distributed over less than the first 10m of trench as reduced Cl concentrations were measured at the 10m and 20m sample positions (Table 11.2) and therefore the depth average method, and not the planar average method, was the more representative method of reporting the attenuation of the percolating effluent. When the average Cl concentrations for each depth plane at the 0m sample position were plotted it was discovered that they were very similar (Figure 11.5). While there were isolated incidents of elevated Cl concentration at 10m it appears that biomat formation along the base of the percolation trench on Site 4 had not been as successful in effluent distribution as on Site 2. These incidents on the 03/02/04 and 18/03/04 were preceded by periods of high STE flow and it is possible that this increased hydraulic load facilitated the distribution of STE along a greater length of percolation trench. These increases were mirrored on the blue and black planes also. It is also possible that increases were due to the development of the biomat along the base of the trenches but as no further sampling was possible this could not be corroborated. There was no perceived explanation for the elevated concentrations measured on the 23/09/03

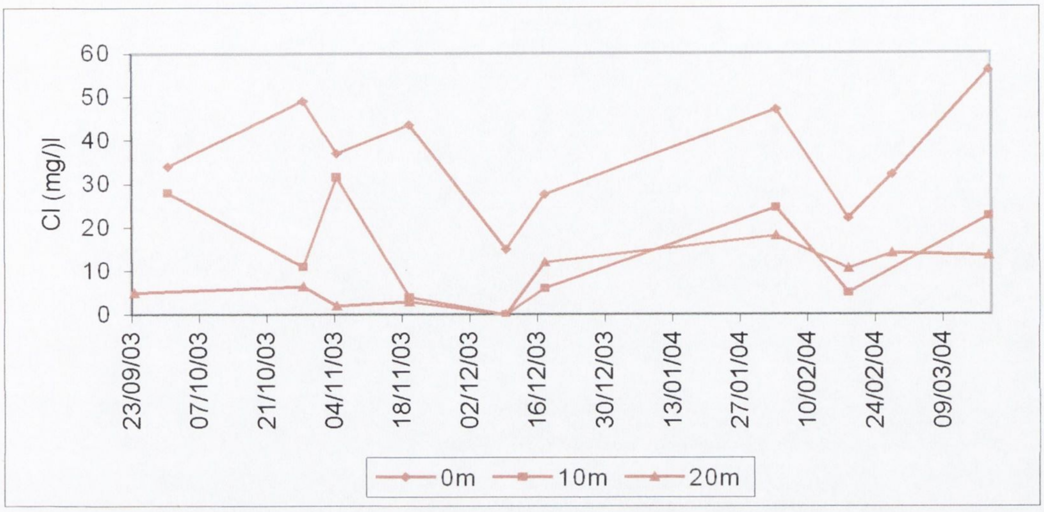


Figure 11.4 Average Cl concentrations on the red depth plane at the 3 sample positions.

Sample Point	1	2	3	4	5	6	7	8	9
Average concentration	29.5	35.2	39.3	9.29	11.3	9.8	9.3	9.7	9.8
Sample Point	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
Average concentration	39.4	32.9	39.0	18.6	8.3	8.6	8.0	8.4	6.6

Table 11.2 Average Cl concentrations (mg/l) measured under percolation area on Site 4.

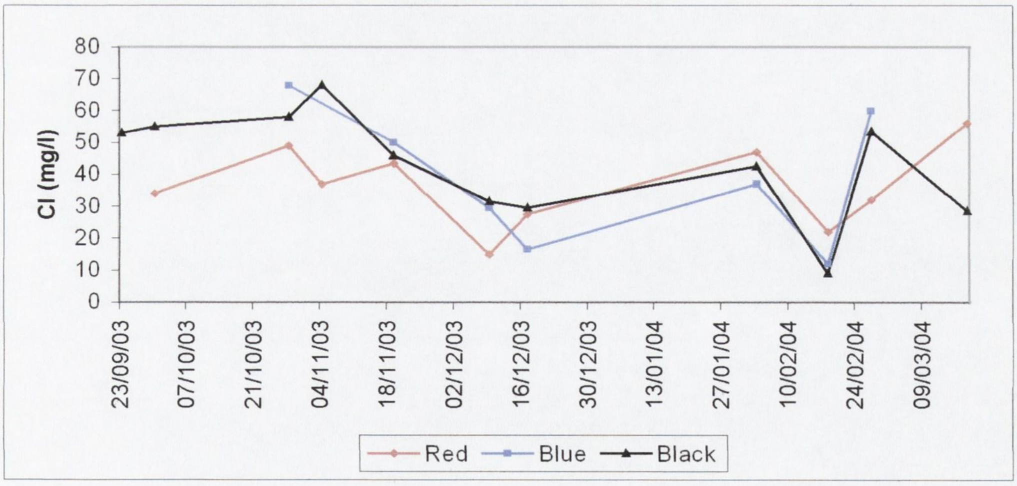


Figure 11.5 Cl concentration measured on the 3 depth planes at the 0m sample position.

and 04/11/03. While the reduction in Cl concentration between 04/11/03 and 03/02/04 appears to be due to an increased contribution of effective rainfall (Section 10.3.2) the drop in measured Cl concentration on 19/02/04 is thought to be due to analysis error.

Although the T-value of 33min/25mm recorded is twice that of the T-value recorded on Site 2 it is possible that it is not a true reflection of the overall percolation characteristics of the subsoil. As the subsoil has been described under BS5930 as very gravelly clayey Sand with some cobbles (Section 5.4.4) the abundance of gravel and cobbles within the subsoil matrix lends itself to the possibility of the presence of preferential flowpaths. The presence of such preferential flowpaths over the first 10m of percolation trench could prevent distribution of effluent along the base of the trench. It would also reduce the residence time of the effluent within the trench thus impeding biomat formation. However, the location of the lysimeters, as outlined in Section 11.3.2, also has to be considered.

11.3.2 The Effect of Dilution on Effluent Attenuation

The quantification of effluent dilution within the subsoil and estimation of the zone of contribution proved problematic for Site 4. Due to the dense nature of the subsoil on Site 4 it was not possible to install the lysimeters directly below the percolation trenches (Section 7.3.2). As a result the ceramic cups, especially those on the red depth plane, might not have been fully immersed in the effluent plume. With effluent dispersion the deeper lysimeters would be expected to be more centrally located within the plume resulting in lower Cl concentrations at the shallower lysimeters as highlighted in Figure 11.5. This is borne out by the soil moisture tension measurements. As with the lysimeters, the tensiometers were installed adjacent to, rather than in, the percolation trenches. As no effluent was recorded at the 20m sample position it appears from Figures 11.6 and 11.7 that both sets of tensiometers responded to effective rainfall rather than the percolating effluent. This suggests that the tensiometers at the 0m sample position were installed outside the effluent plume. While the tensiometers on Site 3 were also installed adjacent to the percolation trenches it is possible that the higher T-value of the subsoil on Site 3 promoted

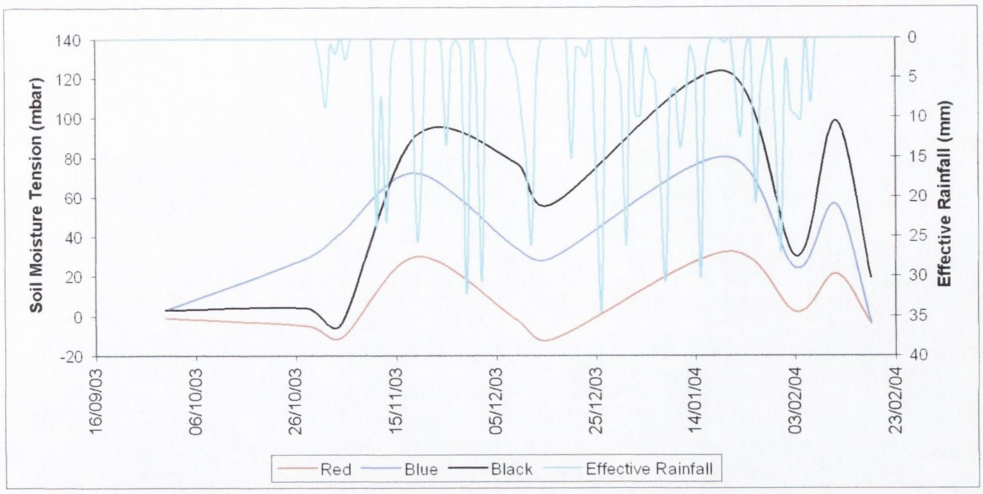


Figure 11.6 Soil moisture tension plotted against effective rainfall for the 0m sample position on Site 4.

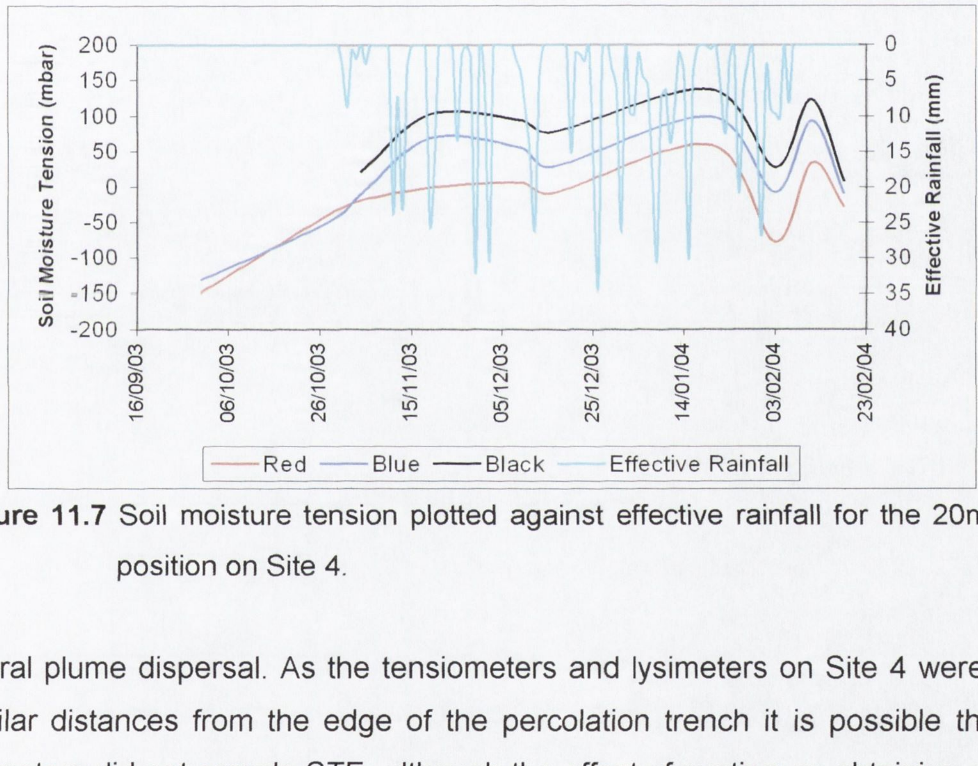


Figure 11.7 Soil moisture tension plotted against effective rainfall for the 20m sample position on Site 4.

lateral plume dispersal. As the tensiometers and lysimeters on Site 4 were located similar distances from the edge of the percolation trench it is possible that some lysimeters did not sample STE, although the affect of suction on obtaining a sample must be considered. It is therefore possible that as the lysimeters at the 10m and the lysimeters and tensiometers at the 20m sample positions were installed further from the trench edge than those at the 0m sample position, due to the inability of the Minute Man to obtain the correct depth at those sample positions, and due to the narrow width of the effluent plume, as highlighted by the tensiometer readings, that the effluent actually reached the 10m and/or 20m sample positions but was not sampled.

While a raingauge was installed on site no other local meteorological data were available, and for reasons outlined in Section 10.4.2, it was decided to use data obtained from the weather station on Pollardstown Fen in calculating the effective rainfall. The potential evapotranspiration was again calculated using the Hargreaves method (Equation 8.1). The rainfall at Site 4 for the period 01/09/03 to 18/03/04 was measured as 922.8mm and using the effective rainfall for this period was calculated as 697.3mm. As expected the influence of effective rainfall was greatest during the winter months when there was no soil moisture deficit, rainfall was highest and temperatures and plant respiration, hence evapotranspiration, were lower. This effect on soil moisture content, and hence dilution, is seen in the reduction in soil moisture tension highlighted in Figures 11.6 and 11.7.

As Cl concentrations measured on the black depth plane were, in general, greater than those measured on the other two depth planes (for reasons outlined above) it was decided to use the difference in Cl concentration between the filtered STE and the black depth plane to estimate the contribution of the effective rainfall to effluent dilution and, as was the case for Site 3, to guestimate the contribution to the other two depth planes using a 2% difference between each depth plane. However, it can be seen from Figure 11.8 that the relationship between effective rainfall and effluent dilution was not as well defined for this data set as it was for Sites 1 and 2 (Figure 11.8). That is to say there appears to be no correlation between the volume of effective rainfall and the difference in Cl concentration between the STE and the black depth plane. As it was not possible to quantify the contribution of effective rainfall to effluent dilution, and therefore the loading rate of the various wastewater constituents at the three depth planes, it had to be estimated using the STE flow rate and a guestimation of the dilution effect, which was calculated using the average dilution effect calculated for Sites 1 and 2. This equated to a dilution affect equivalent to, on average, the addition of 0.13 l/l (or 11.6% reduction in concentration) for the red depth plane, 0.16 l/l (or 14.1% reduction in concentration) for the blue depth plane and 0.20 l/l (or 16.6% reduction in concentration) for the black depth plane.

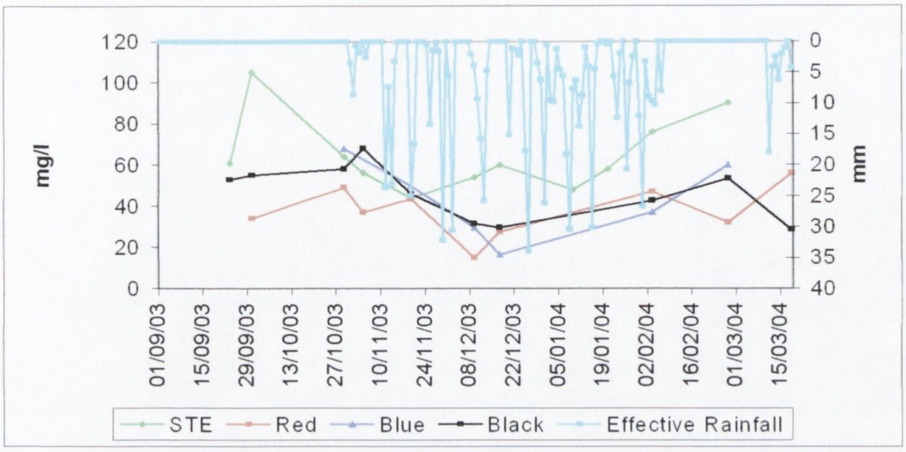


Figure 11.8 Graph showing Cl concentrations and effective rainfall between 01/09/03 and 18/03/04.

When the Cl concentration of the STE was graphed against the rainfall measured on Site 4 for the project duration it was found that as rainfall increased there was a corresponding decrease in the Cl concentration of the STE (Figure 11.9). This is reflected in the variation of the other STE constituents over the sampling period (Figures 11.10 to 11.12). This appears to corroborate the findings of the flow data analysis (Section 11.2) which suggested that domestic wastewater effluent was being diluted by surface water.

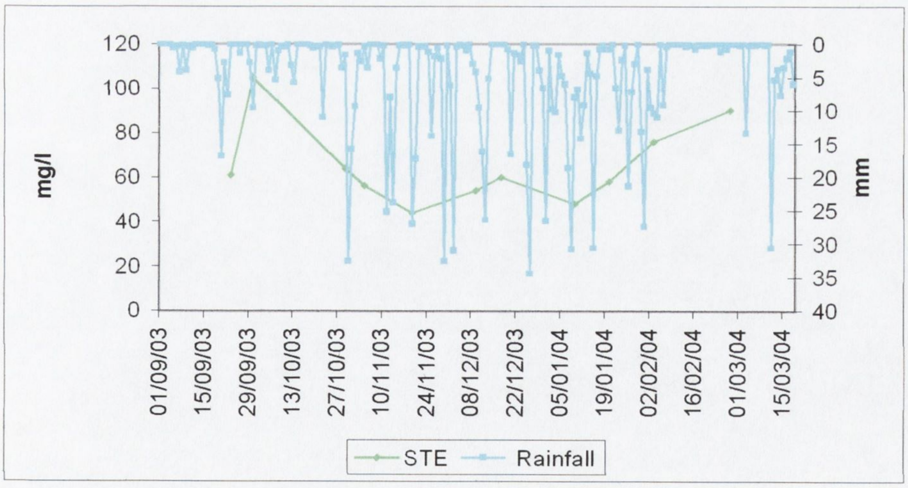


Figure 11.9 The effect of rainfall on STE Cl concentrations at Site 4.

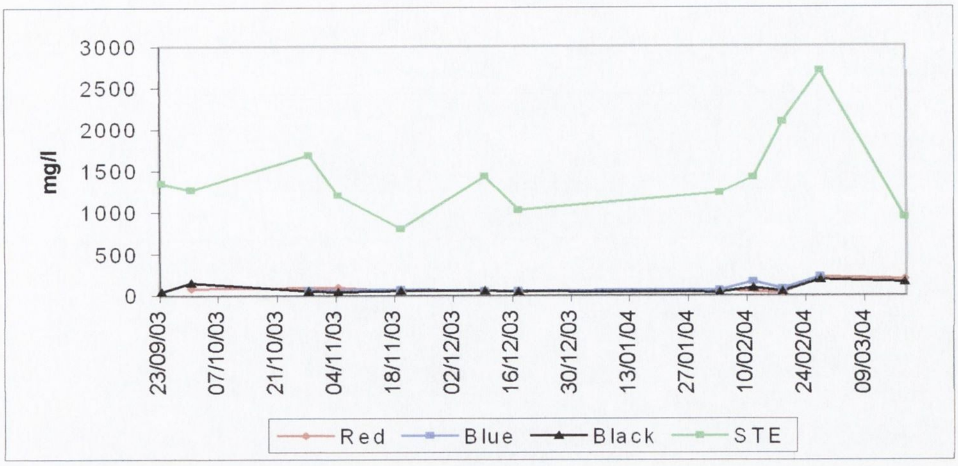
11.3.3 Results of Chemical Analysis

The results of all chemical analysis carried out on STE and soil moisture samples are contained in Appendix E. Tables 11.3 summarises the results of the STE chemical analysis. For reasons outlined in Section 10.3.1 unfiltered and filtered samples of nine out of twelve STE samples obtained underwent chemical analysis for Cl in the laboratory. It was found that filtration resulted in an average reduction of 28.4% on Site 4. When the reduction in Cl concentrations due to sample filtration were averaged for both Sites 3 and 4 it was found that filtration resulted in a 22.6% reduction in Cl concentration.

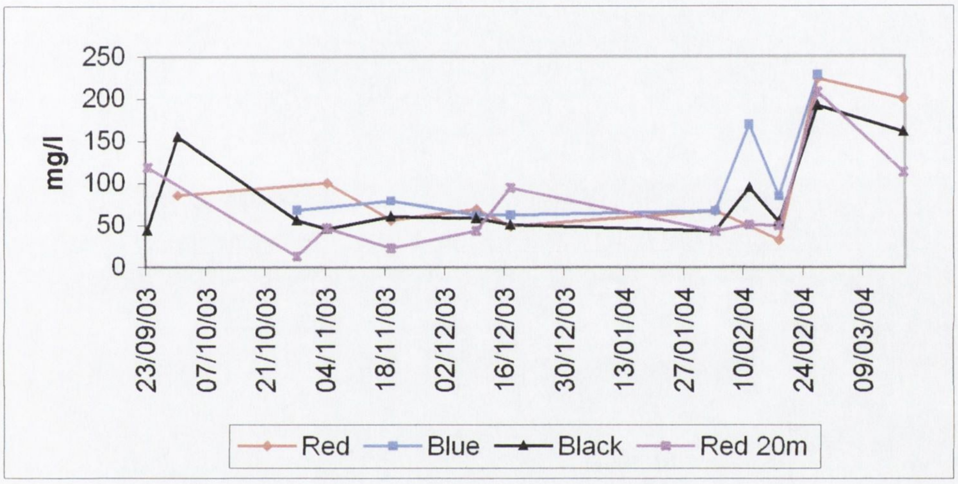
	Concentration (mg/l)					
	COD	NH_4-N	NO_2-N	NO_3-N	PO_4-P	Cl
Maximum	2703.0	71.1	0.7	4.3	16.7	135
Minimum	540.0	19.8	0.2	0.5	3.0	56
Average	**1307.8**	**41.7**	**0.5**	**2.0**	**7.4**	**88.4**

Table 11.3 Summary of chemical analysis of STE on Site 4.

As the lysimeters are slightly offset from the percolation trenches and therefore not centrally located within the effluent plume it is difficult to quantify the attenuation above the red depth plane and between the red depth plane and the other depth planes. It is clear from Figure 11.10 (a) and (b) and Table 11.4, however, that there is, on average, a 94% reduction in COD concentration between the STE and the black depth plane. Due to the similarity between the COD concentration on each depth plane it is clear that the majority of the reduction in COD concentration occurs above the red depth plane. It is also clear from Figure 11.10 (b) that when COD concentrations measured at the 0m sample position were compared with those measured at 20m, where STE was not present, that they are very similar. This suggests that the organic content of the STE had been reduced to background levels by the black depth plane. It appears from Section 9.3.4 that the majority of this reduction occurs within the percolation gravel rather within the subsoil. The O_2 required to achieve this reduction in COD concentration between the STE and the



(a)



(b)

Figure 11.10 Comparison of COD concentration (a) between STE and the subsoil and (b) between the three depth planes at 0m and the red depth plane at 20m.

Sample Position	Concentration	Load	
	(mg/l)	g/d	% Reduction
STE	1307.2	215.1	
Red Depth Plane	128.5	23.9	191.2
Blue Depth Plane	102.7	19.6	4.3
Black Depth Plane	84.7	16.7	2.9

Table 11.4 Average reduction in COD concentration on Site 4.

red depth plane, 1303.7mg/l/d or 0.4kg/d on average, is present within the distribution gravel in the percolation trench, which is aerated by means of a ventilation pipe. As only two percolation trenches were in use on Site 4, as opposed to four on Sites 1 and 2, there is 1.1kg of O_2 available within the percolation trenches to meet demand. Further reduction in COD concentration within the subsoil would result from a combination of physical, chemical and biological processes.

The reduction in total inorganic nitrogen concentration of STE from November 2003 corresponds to a period of increased rainfall, as outlined in Section 11.3.2. This period of increased rainfall resulted in an increase in effective rainfall although this had less of an influence on the total inorganic nitrogen concentration measured on the red and black depth planes in the subsoil (Figure 11.11). Although it was only possible to calculate total inorganic nitrogen concentrations for four occasions on the blue depth plane, it appears from Figure 11.11 and Table 11.5 that the greatest reduction in nitrogen concentration occurred between the blue plane and the black plane. While nitrification was evident from the high concentration of NO_3-N measured

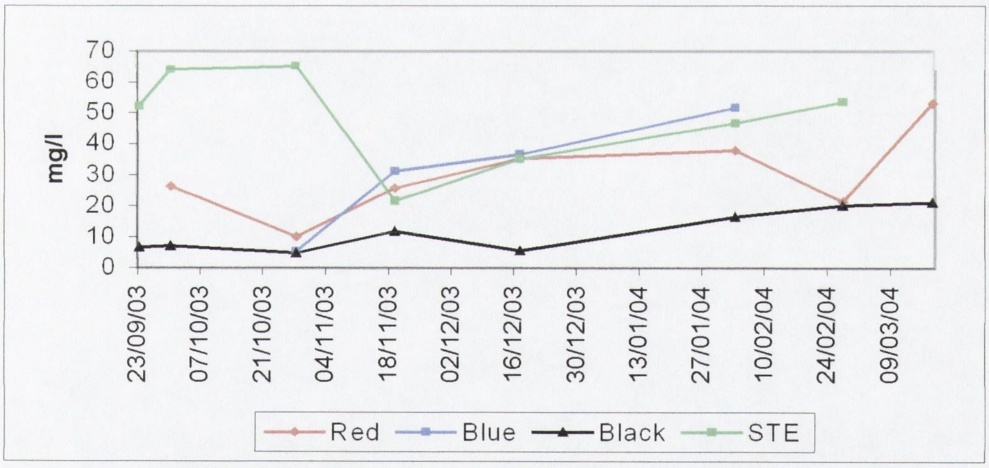


Figure 11.11 Total inorganic nitrogen (mg/l) measured on Site 3.

in soil moisture samples obtained from the red and blue depth planes there was no parallel denitrification over the same subsoil thickness. It is possible that due to the location of these lysimeters within the fringe of the effluent plume that the hydraulic load was not sufficient to produce saturated conditions which would promote denitrification. It is therefore possible that denitrification would occur on the same

Depth Plane	NH_4-N		NO_2-N		NO_3-N		Total N	
	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)	Conc. (mg/l)	Load (g/d)
STE	41.7	6.9	0.5	0.1	2.0	0.3	44.2	7.3
Red	5.1	0.9	0.3	0.1	22.0	4.1	27.4	5.1
Blue	7.2	1.4	0.7	0.1	20.9	4.0	28.8	5.5
Black	3.8	0.8	0.4	0.1	6.1	1.2	10.3	2.1

Table 11.5 Average NO_3-N, NO_2-N, NH_4-N and Total inorganic N concentration and loading rate measured on Site 4.

more central location within the plume where conditions would be more favourable, as seen on Site 2. It is also possible that the increase in soil moisture content with depth, as highlighted by the tensiometer readings in Figure 11.6, led to the creation of anoxic conditions thereby promoting denitrification between the blue and black depth planes. It is also possible that the saturated conditions experienced directly below the percolation trench promoted removal of NH_4-N from the percolating effluent by cation exchange.

The orthoPO_4-P concentration in the STE was, on average, very similar to that outlined in Table 2.2. However, it must be remembered that the STE had been diluted by rainwater. As was the case with COD reduction it can be seen from Table 11.6 and Figure 11.12 that the greatest reduction in effluent PO_4-P load occurred above the red depth plane.. The high clay content of the subsoil below the percolation area on Site 4, revealed by the particle size analysis contained in Appendix A, suggests that the removal of phosphate from the percolating effluent was controlled by soil adsorption (Section 4.2.2). As the subsoil only has a finite capacity to adsorb PO_4-P and as soil moisture samples were only analysed over a eight-month period this depth of subsoil active in PO_4-P fixation will increase with time.

Sample Position	Concentration (mg/l)	Load	
		g/d	Load Removal (g/d)
STE	7.4	1.22	
Red Depth Plane	0.4	0.07	1.15
Blue Depth Plane	0.2	0.04	0.03
Black Depth Plane	0.1	0.02	0.02

Table 11.6 Average reduction in PO_4-P load on Site 4.

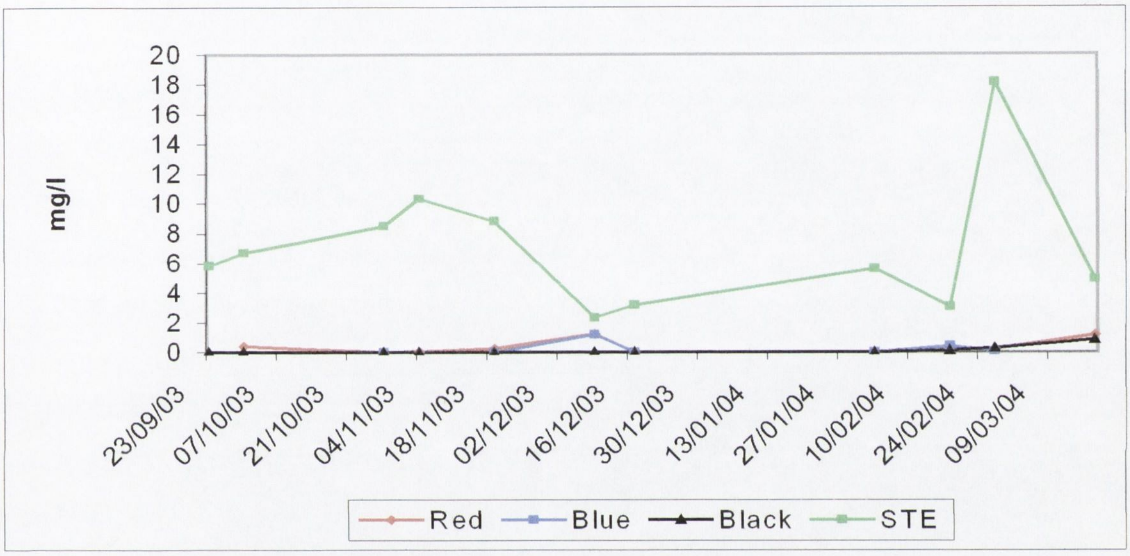


Figure 11.12 PO_4-P concentrations measured on Site 4.

11.3.4 Results of Bacteriological Analysis

Due to financial constraints it was only possible to send two sets of samples from Site 4 for bacteriological analysis. On both occasions samples sent for analysis were obtained from trench 2 only. Assuming that the results obtained from the bacteriological analysis of these samples are representative of the sample position from which they were obtained it can be seen from Table 11.7 that, allowing for the factor of safety introduced due to sample dilutions, there was complete removal of enteric bacteria by the black depth plane.

	Number of Samples	Number of samples with concentration (cfu/100ml)			
		<10	10-100	101-1000	>1000
STE	2				2
Red Plane	2	1	1		
Blue Plane	1	1			
Black Plane	2	2			

Table 11.7 Concentrations of enteric bacteria measured on Site 4.

11.4 Summary

As a result of observations during sample analysis on Sites 1 it was decided to analyse both unfiltered and filtered STE samples for Cl. It was found that, on average, there was a 22.6% reduction in Cl concentration due to filtration with 1.2μm filter paper.

The analysis of sample results obtained from Site 4 was more difficult than experienced for either Site 1 or Site 2 due the problems outlined relating to the installation of the lysimeters and the dilution of STE by surface water runoff. These problems inhibited the calculation of the loading rate at the different depth planes by the method used in Chapters 8 to 10 as it was not possible to quantify the effect of effluent dilution by effective rainfall. However, loading rates were guestimated using the dilution calculations from Sites 1 and 2 and half the average daily flow. The average hydraulic load measured on Site 4, which included a contribution of surface water runoff, was again less than the 180l/pd cited by the EPA (EPA, 2000). As a result the percolation area, design based on a hydraulic load of 20l/m^{2}d, occupied a larger footprint than was necessary.

The chemical analysis of soil moisture samples revealed that less than 10m of each percolation trench was utilised over the project duration. It appears from samples obtained at the 0m sample position that while the greatest attenuation of PO_4-P and COD occurs above the red depth plane the greatest reduction in nitrogen load was measured between the red and black depth planes. Bacteriological analysis revealed that most of the reduction in enteric bacteria concentration occurred above the red depth plane and, allowing for the factor of safety introduced due to the sample dilutions, no enteric bacteria were recorded at the black depth plane.

12. DISCUSSION

12.1 Introduction

With the domestic wastewater effluent of approximately 400,000 dwellings being discharged to groundwater annually and about 25% of all water supplies in the country provided by groundwater, the protection of groundwater resources from contamination by domestic wastewater effluent is imperative. The effect of domestic wastewater effluent treatment systems on the integrity of surface water bodies, due to the contribution of groundwater to surface water, is also an important consideration. By examining the attenuation capacity of the subsoil on four research sites this thesis aimed to review the effectiveness of the guidelines laid out in the EPA manual *Treatment Systems for Single Houses* in the treatment of domestic wastewater effluent and, hence, in the protection of water resources.

The four sites chosen had infiltration properties (measured as T-values) that fell within specified ranges, an occupancy of at least four people and conformed to the other EPA recommendations relating to site suitability. Sites 1 and 3, which had T-values of 28.8 and 52.3 respectively, both had a septic tank and Puraflo® system installed, together with a percolation area downstream built to EPA 2000 specifications. Sites 2 and 4, which had T-values of 14.6 and 32.7 respectively, both had a septic tank installed followed by a similar percolation area. Sampling spanned a 12-month period on Sites 1 and 2 and a 9-month period on Sites 3 and 4. Septic tank effluent (STE) and secondary effluent (SE) samples were obtained from composite samplers while soil moisture samples were obtained from lysimeters, nine of which were installed on each trench.

12.2 Discussion

12.2.1 Hydraulic Load

The EPA manual defines the typical daily hydraulic load to an on-site treatment system for single houses as 180l/pd. This is used in conjunction with specified loading rates of 20l/m^{2}d, for subsoils receiving STE, and 25l/m^{2}d, for subsoils with T-values between 21 and 50 receiving SE, in determining the required size of a percolation area. The hydraulic load on each of the four research sites was recorded using ultrasonic probes. As can be seen from Table 12.1 all of the sites monitored experienced a hydraulic load less than the design load and, as percolation area design is presently based on hydraulic loading alone, the footprint required for percolation areas could be reduced if these measurements of hydraulic load are representative of domestic wastewater generation nationwide.

Site	Average Daily Hydraulic Load (l/d)	Percolation Trench Length (m)	
		Constructed	Required
Site 1	281.7	80	25
Site 2	418.8	80	47
Site 3	246.1	40*	22
Site 4	164.6	40	18

*while a percolation trench length of only 32m was required 40m of trenching was constructed as a precautionary measure due to the T-value falling outsize the acceptable range.

Table 12.1 Comparison of length of percolation trench constructed and that required based on loading rates of 20 l/m^2/d for STE and 25 l/m^2/d for SE.

It was also found that less than 10m of percolation trench were utilised on each of the two sites receiving SE due to the reduction in organic load resulting from the secondary treatment step. It was possible to estimate the length of each percolation trench in use on Site 1 as 4m due to the installation of small lysimeters under the invert of the trench. 5m was used to estimate the length of each trench in use on Site 3. The hydraulic loading rate experienced over the effective lengths of trench can therefore be estimated as 39l/m^{2}d for Site 1 and 55l/m^{2}d for Site 3. The desired loading rate of 25l/m^{2}d could be achieved by constructing a greater number of shorter percolation trenches and installing a distribution box with more outlets so that effluent would be evenly distributed over a greater area. It is also suggested that the minimum and maximum acceptable T-values for a subsoil receiving SE should be so that the reduced permeability of the acceptable subsoil could be used to facilitate the distribution of SE over a greater area.

An estimated 70m of the 80m of percolation trench constructed on Site 2 was in use, the under-utilisation of the system resulting from uneven distribution of the effluent, resulting in a loading rate of approximately 13l/m^{2}d, rather than the design loading rate of 20 l/m^{2}d, being experienced. As the system was operating below the design hydraulic loading rate further research would be required to determine the ability of the system to perform successfully under the design load. Due to the problems outlined in Chapter 11 it was not possible to determine the loading rate on the percolation trenches on Site 4.

It can be seen from Table 12.2 that the per capita domestic wastewater generation on all sites was less than the EPA calculated figure. While the average daily hydraulic load outlined in Table 12.1 defines the hydraulic load on the percolation area it does not give an accurate reflection of the per capita wastewater generation. The average daily hydraulic load for Sites 2 to 4 includes a contribution by surface run-off thereby over-estimating the average domestic wastewater effluent generation, which was estimated for Sites 3 and 4 by determining the average flowrate on the days when there was not rainfall. As the flowrate on Site 2 included a contribution of surface run-off other than rainfall it was not possible to estimate the per capita wastewater generation. However, if the average daily hydraulic load is used it can be approximated at 104.7l/pd which is still less than the EPA design figure. As the average daily hydraulic load on Site 1 included a contribution when the family was on holidays, the per capita wastewater generation was calculated using an adjusted average daily flow which excluded these periods.

Site	Adjusted Daily Hydraulic Load (l/d)	Per Capita Domestic Wastewater Generation (l/pd)
Site 1	294.9	59.0
Site 3	452.1	113.0
Site 4	312.2	78.1

Table 12.2 Per capita wastewater generation on Sites 1, 3 and 4.

12.2.2 Attenuation of Chemical Constituents of STE and SE

The EPA recommend the presence of 0.6m and 1.2m of unsaturated subsoil below the invert of percolation trenches receiving SE and STE, respectively. While it was possible to install lysimeters to 0.6m on all sites it was not always possible to attain a depth of 1.2m. Three nominal depth planes, red, blue and black, were defined to represent the average depth to which the lysimeters were installed (Table 12.3). In using lysimeters, it must be remembered that samples were obtained from the soil matrix, which was assumed to be homogeneous and isotropic, and the effects of preferential flow would therefore also have to be considered.

Depth Plane	Distance Below PercolationTrench Invert (m)			
	Site 1	Site 2	Site 3	Site 4
Red	0.3	0.2	0.2	0.2
Blue	0.6	0.55	0.6	0.6
Black	0.9	0.9	1.0	1.0

Table 12.3 Distance of depth planes below the invert of the percolation trenches.

Chloride analysis of the soil moisture samples was used to determine the distribution of the STE and SE within the percolation areas. It showed that, as outlined in Section 12.2.1, the reduction in organic load resulting from the secondary treatment of effluent on Sites 1 and 3 inhibited the formation of a biomat along the base of the percolation trenches resulting in less than 10m of trench being utilised on both sites. This means that highly pre-treated wastewaters could be applied at rates greater than that recommended for STE without the occurrence of hydraulic failure of the system.

The organic load delivered to the percolation area on Sites 1 and 3 was greatly reduced as a result of the secondary treatment step afforded by the Puraflo® systems. An average 76%, or 167.5g/d, reduction in STE COD load was recorded on Site 1 while a similar reduction of 75%, or 142.9g/d, was recorded on Site 3. This is in agreement with the literature review which reported a 70-90% reduction in COD load across peat filters. Within all the percolation areas COD was reduced to background levels, with most of this reduction occurring above the red depth plane.

In fact, it appears that most of this reduction occurred within the distribution gravel of the percolation trenches and that the distribution gravel operated, by default, as a horizontal flow aeration bed (Section 9.3.4). The availability of approximately 0.6kg of O_2 per percolation trench for oxidation lends credence to this theory. Table 12.4 highlights the reduction in COD load attributed to the specific treatment steps.

Sample Position	COD Removal							
	Site 1		Site 2		Site 3		Site 4	
	%	g/d	%	g/d	%	g/d	%	g/d
SE	83.3	167.5	N/A	N/A	84.2	142.9	N/A	N/A
Red Plane	12.2	24.6	93.7	119.8	12.3	20.3	96.4	191.2
Blue Plane	4.5	9.0	0.0	0.0	3.5	6.0	2.2	4.3
Black Plane	N/A	N/A	6.3	8.1	0.0	0.0	1.4	2.9

Table 12.4 Reduction in COD load attributed to the specific treatment steps.

The installation of the Puraflo® systems also resulted in a reduction in the nutrient load delivered the percolation area on to Sites 1 and 3. Denitrification within the systems reduced the total inorganic nitrogen load to the percolation area by 51.6% on average, or 19.2g/d, on Site 1 and 38% on average, or 7.7g/d on Site 3. It appears, however, that the subsequent treatment of STE by the Puraflo® system prior to its discharge to the percolation trenches had a detrimental effect on the attenuation capacity of the subsoil treatment system, with respect to inorganic nitrogen, on both sites. It is postulated that reduction in the organic load inhibited the occurrence of the clogged or saturated zones required for denitrification and also, where isolated saturated conditions occurred, deprived the bacteria of the carbon source essential for denitrification. As a result the total inorganic load at the minimum recommended depth of unsaturated subsoil, 0.6m for SE and 1.2m for STE, was greater for sites with secondary treatment systems installed than it was for sites with septic tank treatment systems installed (Table 12.5). It can be seen from Table 12.6 that there was, in fact, little further reduction in total inorganic nitrogen within the subsoil on the secondary treatment system sites.

Sample Position	Total Inorganic Nitrogen Load (g/d)			
	Site 1	Site 2	Site 3	Site 4
STE	37.2	22.6	20.3	8.0
SE	18.0	N/A	12.6	N/A
Red Plane	17.0	8.8	13.7	5.6
Blue Plane	**16.8**	6.6	**13.2**	6.0
Black Plane	N/A	7.4	12.5	2.3

Table 12.5 Average total inorganic nitrogen load measured on the four research sites.

Sample Position	Site 1		Site 2		Site 3		Site 4	
	%	g/d	%	g/d	%	g/d	%	g/d
SE	94.1	19.2	N/A	N/A	86.5	7.7	N/A	N/A
Red Plane	4.9	1.0	86.3	13.8	0.0	0.0	42.1	2.4
Blue Plane	1.0	0.2	13.7	2.2	5.6	0.5	0.0	0.0
Black Plane	N/A	N/A	0.0	0	7.9	0.7	57.9	3.3

Table 12.6 Reduction in inorganic nitrogen load attributed to the specific treatment steps.

It is important to note that the denitrification experienced within the Puraflo® systems resulted from adjustments to the Puraflo® modules which were required to enable discharge of SE to the percolation trenches and would therefore not occur within the standard Puraflo® system or indeed within most other packaged treatment plants on the market. The nutrient load at minimum recommended depth of unsaturated subsoil, i.e. 0.6m, would therefore have been greater should the standard Puraflo® system have been installed. On Site 2 it was found that the greatest reduction in inorganic nitrogen load occurred above the red depth plane, with little reduction thereafter. While the greatest reduction in inorganic nitrogen on Site 4 occurred between the blue and black depth planes it is suspected that the location of the red and blue lysimeters on the periphery of the effluent plume under-estimated the reduction in inorganic nitrogen load between the STE and those depth planes.

The attenuation of PO_4-P within the subsoil treatment system is a function of subsoil texture and mineralogy. Table 12.7 shows the average PO_4-P load entering the distribution box and that measured on each of the depth planes. The higher clay content of the subsoils on Sites 2 and 4 facilitated a greater percentage removal of PO_4-P above the red depth plane than for Sites 1 and 3 (Table 12.8). It was also found that the red depth plane on Site 1 removed more PO_4-P than the blue depth plane and this was attributed to its higher clay content. The greatest reduction in the PO_4-P load on Site 1 was recorded above the black depth plane. This fixation of PO_4-P was thought to result from the presence of calcite in the subsoil matrix and alkaline conditions. The greatest reduction PO_4-P on Site 3 was also recorded between the blue and black depth planes. However, as acidic conditions were prevalent it is possible that PO_4-P fixation resulted from the presence of Al, Fe,

Sample Position	PO_4-P Load (g/d)			
	Site 1	Site 2	Site 3	Site 4
STE	N/A	5.9	N/A	1.22
SE	9.5	N/A	2.0	N/A
Red Plane	6.9	1.2	1.7	0.07
Blue Plane	5.8	0.7	1.3	0.04
Black Plane	N/A	0.6	0.2	0.02

Table 12.7 Average PO_4-P measured on the four research sites.

Sample Position	PO_4-P Removal							
	Site 1*		Site 2		Site 3		Site 4	
	%	g/d	%	g/d	%	g/d	%	g/d
Red Plane	70.3 (41.0)	2.6 (3.2)	88.7	4.7	16.7	0.3	95.8	1.15
Blue Plane	29.7 (0.0)	1.1 (0.0)	9.4	0.5	22.2	0.4	2.5	0.03
Black Plane	N/A (59.0)	N/A (4.6)	1.9	0.1	61.1	1.1	1.7	0.02

* figures in brackets represent four occasions where samples were available on the black depth plane.

Table 12.8 Reduction in PO_4-P load attributed to the specific treatment steps.

and/or Mn in the subsoil. The installation of secondary treatment systems on Site 1 and 3 had little affect on the PO_4-P load of the STE.

12.2.3 Attenuation of Bacteriological Constituents of STE and SE

Four sets of samples from both Sites 1 and 2 and two from both Sites 3 and 4 underwent bacteriological analysis. While the literature reported greater than 99% removal of total coliforms and faecal coliforms by peat filters there was at least a 98% reduction in enteric bacteria concentration across the Puraflo® system for Site 1 and at least a 97% reduction on Site 3. However, the presence of a considerable concentration of enteric bacteria in the SE emphasised the importance of subsequent treatment within the subsoil. As was the case with most of the chemical constituents, the greatest reduction in bacteriological concentration in the subsoil occurred above the red depth plane on all sites. Biomats have been observed to retain as much as 99.9% of the coliform load over a distance of less than a foot (Section 2.3.2). Of a combined total of 18 samples taken from the red depth plane on all four sites, 17 samples had an enteric bacteria concentration less than the limit of detection (10cfu/ml). The other sample had a concentration of 20cfu/ml. It can be seen from Table 12.8 that almost complete removal of enteric bacteria was achieved by the point of discharge to groundwater. While a sample obtained from Site 2 showed a concentration of 10cfu/ml on one of the four occasions on which samples underwent bacteriological analysis, it was shown in Table 9.12 that all samples from the red depth plane analysed had concentrations <10cfu/ml. This indicates that there is a potential for bacteriological contamination of groundwater by STE and SE because of the heterogeneity of subsoil.

Site	No. of Samples	No. Samples at Point of Discharge* with Concentration (cfu/ml)			
		< 10	10 – 100	101 – 1000	> 1000
Site 1	5	5	0	0	0
Site 2	11	10	1	0	0
Site 3	2	2	0	0	0
Site 4	2	2	0	0	0

* where point of discharge is the blue depth plane on sites receiving SE and the black depth plane on sites receiving STE.

Table 12.8 Concentration of enteric bacteria sampled at the discharge point on the four research.

12.2.4 Definition of Target

The EPA manual *Treatment Systems for Single Houses* appears to define the target as groundwater at a depth of 0.6m below the invert of the percolation trench, where domestic wastewater effluent is discharged from a secondary treatment system, and as groundwater 1.2m below the invert of the percolation trench, where domestic wastewater effluent is discharged from a septic tank. However, with respect to the risk of bacteriological contamination it may be more realistic to identify the target as an adjacent water source, such as a borehole or surface water, as further reduction of the bacteriological load will occur within the saturated zone due to filtration, sorption and inactivation/die-off. With respect to chemical pollutant attenuation, while further denitrification could occur within the saturated zone, provided a sufficient carbon source is present reliance on passive attenuation processes such as dilution and dispersion to reduce the contaminant load could result in an overall reduction in regional groundwater quality.

12.2.5 Site Assessment

While the T-test forms an essential part of the site suitability assessment it is felt that there is an over-emphasis on its importance. It appears that local authorities rely heavily on the results of the T-test when gauging the suitability of a site and fail to see if they tally with the subsoil classification, as determined by the trial hole inspection, or with previous experience in the area. As a result they are often manipulated and inaccurate. It is important to remember that the assimilation capacity of the subsoil is not the only concern as the subsoil characteristics are important with respect to effluent attenuation, especially in the removal of PO_4-P and bacteria.

It is also felt that the T-test holes and the trial hole should be excavated outside, but adjacent to the proposed percolation area to minimise the disturbance of the treatment medium. While a T-test result is currently acceptable if either one of the t-values falls outside the acceptable T-value range, as long as the T-value falls within this range, it is suggested that this should not be allowed. It appears, from limited research in the field, that the adapted T-test, outlined in Chapter 5, which is less time consuming than the standard T-test, gives a good approximation of the standard T-test and should therefore form an acceptable part of the on-site assessment procedure.

12.2.6 Site Construction

It is currently specified that the invert of a percolation trench receiving STE should be 0.8m below ground level. However, this militates against borderline sites where the watertable is marginally above the threshold depth of 2m below ground level. As it is the subsoil below the percolation trench, rather than above it, that is active in the attenuation of the percolating effluent, it is recommended that this trench depth be decreased. The depth of the invert below ground level should be determined by the maximum load that the pipes would have to bear and also take account of penetration of ground frost when present. This should also be applied in the construction of percolation trenches receiving SE.

The only certification currently available to the manufacturers of secondary treatment systems is the AGRÉMENT Certificate which certifies that the system is built to acceptable standards. It does not assess the ability of the system to reduce the organic, inorganic and bacteriological load and thus improve effluent quality. It is recommended that all secondary treatment systems should have to meet specified criteria relating to the quality of their effluent.

13. CONCLUSIONS AND RECOMMENDATIONS

13.1 Conclusions

The main conclusions of this study are as follows:

- LOADING RATE
  (i) Hydraulic Loading Rate:
    - The daily per capita wastewater generation experienced on all four sites was less than that specified by the EPA (EPA, 2000) of 180l/person/day. Therefore, based on the EPA recommended hydraulic loading rates for STE and SE (EPA, 2000), it would be possible to reduce the footprint of the percolation areas.

  (ii) Organic Loading Rate:
    - The reduced organic load of SE, relative to STE, resulted in less than 10m of percolation trench being utilised on Sites 1 and 3. This concurred with the findings of the literature (Section 2.3.2) which highlighted that, as soil clogging is a function of organic and solids loading rate from the effluent, where subsoil receives highly treated effluent a biomat will not form because the organic load is very low. As a result loading rates greater than the deign loading rate of $25 l/m^2/d$ were experienced.
    - Secondary effluent can be applied at hydraulic loading rates greater than the EPA recommended (EPA, 2000) loading rated without the occurrence of hydraulic failure of the system.

- ATTENUATION OF DOMESTIC WASTEWATER EFFLUENT.
  (ii)Attenuation of Chemical Constituents of Effluent:
    - A 75% reduction in COD was measured across the Puraflo® systems
    - In the septic tank treatment systems COD attenuation occurred within the distribution gravel rather than the subsoil with the percolation trench operating as a horizontal flow aeration bed.

- The modifications to the Puraflo® modules, required to insure even distribution of effluent between the percolation trenches and which resulted in flooded conditions, facilitated denitrification. This resulted in, on average, a 40% reduction in nutrient load to the percolation area.
- The inorganic nitrogen load at the minimum recommended depth of unsaturated subsoil (EPA, 2000) was greater for the sites on which Puraflo® systems were installed.
- Attenuation of PO_4-P was dependent on subsoil properties.

(iii) Attenuation of Bacteriological Constituents of Effluent:

- A 97% reduction in enteric bacteria concentration was experienced across the Puraflo® systems.
- The greatest reduction in the bacterial concentration of the percolating effluent occurred in the first 0.3m of subsoil on all sites. Of a combined total of 18 samples taken from the red depth plane of all four sites, 17 samples had an enteric bacteria concentration less than the limit of detection (10cfu/ml).

13.2 Recommendation for Further Study

(i) The attainment of even effluent distribution between the percolation trenches is of utmost importance in achieving a uniform loading rate over the percolation area, and hence maximising the attenuation capacity of the subsoil. The design and installation of a distribution box that successfully achieves this goal is therefore imperative. Currently such a distribution box is not available and it is recommended that research be undertaken to rectify this.

(ii) The nutrient load discharged to groundwater from secondary treatment systems is quite significant and far greater than that discharged from septic tank treatment systems. This could be rectified should the secondary treatment system have a capacity for denitrification. The design of a denitrification system that could be installed downstream of secondary treatment systems already in operation, and all future installations, would reduce the nutrient load on groundwater thus improving its quality.

(iii) While this research examined the attenuation capacity of a number of subsoils with T-values greater then 14, the capacity of a subsoil with a lower T-value, especially those with a T-value just above the minimum acceptable value of 1 should also be examined. The ability of such subsoils to reduce the organic, inorganic but especially the bacteriological load of the percolating effluent is of utmost importance in the protection of groundwater resources due to the exploitation of such sandy and gravelly subsoils as waster sources.

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