Improving the Performance of Septic Tank Soil Absorption Systems

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by

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Abstract

This thesis is primarily a literature review of research and information about septic tank soil absorption systems and their combined performance as on-site sewage treatment systems. The purpose of the review is to analyse the functions of the tank and the soil absorption area and assess how aspects of their design affect their overall performance. Design variations that optimise septic tank soil absorption system performance have been determined so that they can be applied in New Zealand.

An assessment of New Zealand septic tank soil absorption systems with regard to their treatment capabilities shows that they are designed to fail. That is, the tanks are designed to produce a poor quality effluent, and the absorption areas are designed to produce saturated conditions so that contaminants in the effluent are transported to groundwater. Inadequate systems are installed and used because there has been little New Zealand research into the environmental and public health effects of their use and how these effects could be reduced, and there has been limited application of knowledge gained from overseas research.

Septic tank design features that optimise wastewater treatment are dual or multi-chambered tanks, outlet tees, outlet baffles, anaerobic filters, and large sludge storage volumes. These features improve the physical processes of sedimentation and separation, the biological process of anaerobic decomposition, and provide storage for accumulated sludge and scum. Incorporating these features can achieve consistent, significant biochemical oxygen demand and suspended solids reductions and assist in retaining pathogenic organisms and nitrogen in the sludge at the bottom of the tank. Dual or multi-chambered tanks, outlet tees, and outlet baffles are effective because wastewater short-circuiting is avoided or decreased and sludge is retained in the tank. Anaerobic filters provide effective treatment of low strength wastewater and are insensitive to variable loading rates. This makes them suitable for the types of wastewater and loading use generated by individual households. Improved septic tank effluent quality in terms of BOD and suspended solids concentrations extends the life of the soil absorption area.

Soil absorption area design features that optimise wastewater treatment are pressurised effluent distribution in all soils, effluent application at hydraulic loadings appropriate to the soil type, the presence of a biological mat, particularly in sandy soils, and passive de-nitrification components. These features ensure that effluent is applied to the entire infiltrative surface at the design hydraulic loading, assist in removing pathogenic organisms in the effluent, and avoid or decrease adverse effects on groundwater quality from nitrates.

Critical aspects of the soil absorption area design are the hydraulic loading, the method of distribution, and components that promote de-nitrification. Potential adverse effects on groundwater quality and public health are reduced by maintaining unsaturated conditions under the soil absorption areas and removing nitrates. Uniform effluent application at a rate compatible with the infiltration capacity of the soil is essential to avoid saturated conditions. Uniform application is achieved by pressure distribution.

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Chapter One
Introduction

On-site domestic waste systems are used throughout New Zealand where connection to a central collection system is not possible. The systems used include simple disposal options such as pit latrines and soakholes, septic tank soil absorption systems, and more recently, more complex aerobic systems. The most commonly used on-site domestic waste system in New Zealand is the septic tank soil absorption system (ST-SAS).

Septic tank soil absorption systems have generally been regarded as the most cost-effective and convenient method to dispose of domestic sewage from single dwellings. Ordinary household conveniences such as flush toilets, baths, showers and clothes washing machines are all utilised, often with little regard to the ultimate end-of-pipe effluent disposal.

The receiving environment of a septic tank system is the soil surrounding and beneath the soil absorption area, and ultimately ground water. Septic tank soil absorption systems are commonly installed in all types of receiving environments except those restricted by highly impermeable soils.

The primary concern with the use of septic tank soil absorption systems relates to their cumulative effects on groundwater. This is of particular concern where groundwater is used for drinking water. The threat to public health is combined with problems of nuisance and odour if effluent from the system reaches the ground surface. Septic tank effluent can also cause nutrient enrichment of surface water.

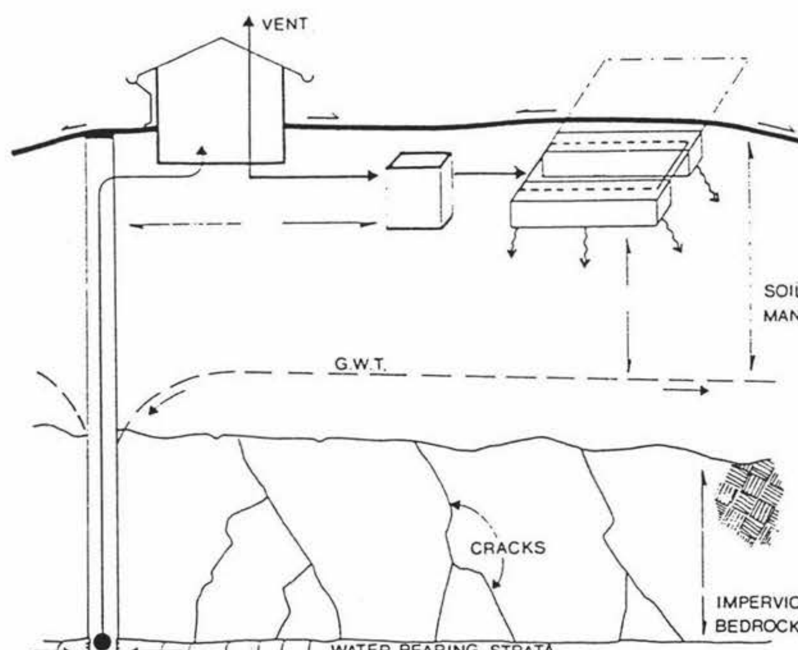
Environments receiving waste from community sewage treatment systems are routinely monitored for compliance with environmental standards to ensure that the discharge does not cause adverse effects. On-site systems, however, undergo little or no monitoring once they are installed, and their construction follows a basic design little changed over the last 100 years. Government subsidies on sewerage systems between 1968 and 1989 directed public health engineers and regulatory authorities into the development of community systems and removed any incentive for the advancement of on-site system technologies as was occurring in other developed countries (Graham, 1992). New Zealand now lags behind modern on-site sewage treatment and disposal technology.

This thesis is primarily a literature review of research and information about septic tank soil absorption systems and their combined performance as on-site sewage treatment systems. The purpose of the review is to analyse the functions of the tank and the soil absorption area, determine the features of their design that affect their overall performance, and recommend design variations that can be applied in New Zealand to optimise septic tank soil absorption system performance.

An introduction to the concerns about the environmental effects of septic tank soil absorption systems, and the regulatory framework governing their use is given in Chapter Two. The function of a septic tank, including the physical and biochemical processes occurring in the tank, its engineering design, and the purpose of its component parts is described in Chapter Three. The function of the soil absorption area, including the physical and biochemical processes occurring there, and its engineering design is described in Chapter Four. In Chapter Five some examples of New Zealand ST-SAS designs and management requirements are assessed with regard to their performance capabilities, and identified ways to optimise ST-SAS performance are discussed. Conclusions and recommendations are given in Chapter Six.

Chapter Two
Septic tank soil absorption systems

Septic tank soil absorption systems (ST-SAS) comprise two sewage treatment processes. First, a tank (usually buried) where waterborne wastes are collected, and scum, grease and settleable solids are separated from the liquid by physical processes of settling and floatation; and second, a sub-surface drainage system (the soil absorption area) where clarified effluent percolates into the soil. A typical system and its interaction with the environment is shown in Figure 1. System performance depends on waste characteristics, the tank design, hydraulic loading rates and methods, soakage field design, soil types and topography, climate, and system maintenance (Canter and Knox, 1985).



Definition sketch: individual subsurface disposal system with a well.

Figure 1. Septic tank soil absorption system (Laak, 1986).

The septic tank operates as a settling tank combined with an unstirred, cold, anaerobic digester. Tanks may have one or more chambers, and can include mechanisms such as anaerobic filters to decrease levels of soluble biochemical oxygen demand (BOD_5) and suspended solids in the tank effluent.

The soil absorption area provides final treatment of the septic tank effluent and a receiving environment for its disposal. Effluent distribution in the soil absorption area can be via surface or sub-surface distribution drains.

2.1 The potential scope for contamination by ST-SAS

2.1.1 New Zealand and overseas studies

There has been considerable research undertaken in New Zealand into the effects of discharging industrial and agricultural waste into surface water (Ministry for the Environment, 1992 and 1994), and into the effects of the disposal of agricultural waste onto land. In contrast, there have been relatively few studies about the effects of septic tank discharges on nutrient and bacterial contamination of surface water (Gibbs, 1977a; Gibbs, 1977b; Hoare, 1984), or groundwater (Sinton, 1982; Sinton, 1986; Close, 1989; Close *et al.*, 1989).

Gibbs (1977a) found that septic tank effluent discharged into a soakhole serving one household travelled considerable distances through pumice soils at Lake Taupo, and that there was a "high possibility" that the effluent reached the lake twenty-three metres away. Further studies at the same site indicated that as much as nine kilograms of phosphorus and thirty kilograms of nitrogen per year could be discharged to the lake from one domestic soakhole, and that the total yearly nutrient input from household soakholes into Lake Taupo could be as much as 19.5 tonnes phosphorus and 65 tonnes nitrogen (Gibbs, 1977b). Sinton (1982) estimated that nitrate-nitrogen entering groundwater from septic tanks in the Yaldhurst area was between 36-46 kg per household per year, and that between twenty and thirty percent of the groundwater nitrate in an unsewered area near Christchurch could be attributed to septic tank contamination.

The Bay of Plenty Regional Council analysed surface water, groundwater and shallow sediments near communities reliant on ST-SAS sewage disposal for the presence of microbial and nutrient contamination. The highest levels of a human specific bacteriophage (96,400 plaque forming units per 100 grams) were present in shoreline sediments of Lake Tikitapu (the 'Blue Lake'), and the next highest levels were at Lakes Tarawera and Okareka. These sites were fifty to seventy metres away from the nearest houses or baches and there was no other likely source of contamination. Sediment bacteriophage contamination in some coastal communities such as Little Waihi and Maketu was up to 1,425,000 pfu/100 grams, while some shellfish had concentrations of 107,100 pfu/100 grams. Groundwater from bores six to eight metres deep at Omokoroa Beach had nitrate-nitrogen levels up to 13.3 g/m^3 (Bay of Plenty Regional Council, 1992).

Elevated nitrate and pathogen indicator levels are also evident in groundwater quality data held by other regional councils, but the source of the contamination is difficult to identify. This is partly because the impacts of agricultural land uses on groundwater nitrate levels has not been quantified, and because baseline groundwater quality monitoring has only been undertaken recently in many regions. The Manawatu-Wanganui Regional Council, for example, has only three years (1993-96) of baseline groundwater quality data for fifteen sites in the Manawatu River catchment and seven other sites in the region.

There have been a considerable number of studies in the United States into the sub-surface behaviour of effluent from septic tank systems (Wilhelm *et al.*, 1994). A review of the American literature (Reneau *et al.*, 1989), found that biological and chemical contaminants from on-site wastewater systems travelled extensively in soils. They advised that further studies were necessary to investigate the possibilities for decreasing nitrogenous contamination. Recent American research indicates that nitrate contamination of groundwater is an underestimated threat from ST-SAS and could be significant enough to challenge their continued use in some areas (Wilhelm *et al.*, 1994).

2.1.2 Examples of at-risk areas in the Manawatu-Wanganui Region

Two methods to address adverse effects from ST-SAS are to improve their design, and to avoid or decrease their use in unsuitable areas. There are fourteen regions, each administered by a regional council or unitary authority, in New Zealand. The extent of the need for on-site sewage disposal in one of these, the Manawatu-Wanganui Region, and some examples of possibly unsuitable areas in this region, are described below.

The percentage of people in the Manawatu-Wanganui Region dependent on on-site sewage disposal can be estimated using 1991 census data. This is shown in Table 1 below.

Table 1. Populations in the Manawatu-Wanganui Region served by reticulated sewerage systems.

District	total population	population served by sewerage	percentage
Horowhenua district	29,500	22,600	77
Palmerston North district	70,300	64,700	92
Manawatu district	27,000	14,700	54
Tararua district	19,500	10,700	55
Rangitikei district	16,700	9,700	58
Wanganui district	45,100	41,700	92
Ruapehu district	18,200	9,400	52
Manawatu-Wanganui Region	226,300	173,500	77

In some districts such as Ruapehu, nearly half the population are reliant on-site sewage disposal, but in the region overall about 23 percent reliant on some form of on-site sewage disposal. This is similar to the United States where about one third of the population have on-site sewage disposal (Canter and Knox, 1985).

Of the coastal communities on the western Manawatu-Wanganui coastline only the Foxton Beach, Waitarere, and Moawhanau communities have a reticulated sewerage system, and only Foxton Beach, Moawhanau and, on the east coast, Akitio have reticulated water supply. Waikawa, Hokio, and Tangimoana on the west coast, and Herbertville and Owawhango on the east coast have neither sewerage or water supply. These coastal communities are reliant not only on on-site disposal of wastewater but on on-site (usually rooftop and/or bore) collection of drinking water as well. In addition, these coastal areas have sandy soils with high permeability. This can lead to problems, for example, groundwater quality monitoring at Hokio Beach and Manakau indicates that nitrate levels occasionally breach New Zealand drinking water standards. Faecal contamination in private drinking bores is also evident (Bekesi, 1994).

East of the Ruahine and Tararua Ranges the predominant land use is agricultural, and residential housing is scattered. Most houses have wastewater disposal by ST-SAS and are generally reliant on bore water for domestic supply as well. The majority of bores in the Pahiatua area tap a shallow unconfined gravel aquifer. There is no suitable deep groundwater there that could be used as an alternative to surface water or shallow groundwater. The soils overlying this aquifer are semi-pervious silts about three to six metres deep. Available evidence shows elevated or higher than expected nitrate and pathogen indicator levels in this groundwater (Bekesi, 1994).

Many areas such as Aokautere and Ashhurst near Palmerston North have been sub-divided into two to five hectare 'lifestyle' blocks. They have no reticulated water supply or sewerage. Soils in these areas are clayey with extremely low permeability and conventional soakage fields have difficulty absorbing the quantities of effluent generated by a typical household without surface ponding, particularly in winter (Lawrence, 1994).

There has been no research, other than water quality monitoring, into ST-SAS effects on groundwater or surface water in any of these areas. The geographical environments indicate that these areas would be vulnerable to contamination, but the degree to which this occurs and the changes necessary to avoid further contamination has not been identified.

2.2 The regulatory framework

The statutory framework for managing water quality and controlling discharges to land and water is the Resource Management Act, 1991. The purpose of the Resource Management Act is to promote the sustainable management of natural and physical resources. Included in the definition of sustainable management is the need to meet the reasonably foreseeable needs of future generations, and to avoid, remedy or mitigate any adverse effects of activities on the environment.

Discharges of contaminants to water, or to land where the discharge may enter water (including surface water, groundwater and coastal water) are restricted by Section 15 of the Resource Management Act (1991). Controlling discharges of contaminants to the environment is a function of regional councils under that Act. Any discharge to water requires a resource consent from the regional council, unless the discharge is expressly permitted by a rule in a regional plan. Regional councils must be satisfied that a discharge from a ST-SAS will not cause adverse effects in the receiving environment before they adopt a rule in a regional plan allowing such discharges as a Permitted Activity.

Controlling the effects of the use and development of land is a function of district councils under the Resource Management Act. District councils have additional responsibilities under the Building Act (1991), which includes requirements about the conveyance of "foul water". The septic tank is classed under the Building Act as being part of the building and is subject to building permit requirements. Further, Section G13.3.4 of the Building Code states that "where no sewer is available, an adequate on-site disposal system shall be provided for foul water". The Building Code further requires that the disposal system is constructed to "avoid the likelihood of contamination of soils, ground water and waterways except as permitted by a resource consent given under the Resource Management Act."

The New Zealand Standards for Household Septic Tank Systems (NZS 4610:1982) include recommendations rather than mandatory requirements for septic tank and soakage field design. There is no legal requirement to abide by the Standards unless a district council requires such compliance as part of a building permit, or a regional council requires compliance as a condition of a resource consent or a rule in a regional plan.

The septic tank and its discharge therefore fall under the jurisdiction of two authorities, each with functions under the Resource Management Act, and one with the additional responsibilities of the Building Act. Effective management of discharges from ST-SAS by these authorities requires that they are aware of the potential effects of these discharges and that they know about appropriate designs to avoid or mitigate adverse effects on the environment.

Chapter Three
Septic tanks

3.1 *Function of the tank*

Wastewater entering the septic tank is treated by the physical processes of sedimentation and separation, and the biological process of anaerobic decomposition of organic matter. The three key functions of the tank are —

- to allow the sedimentation of settleable solids, which are retained as a sludge at the bottom of the tank, and the flotation of grease and other floatable substances, which form a surface scum mat on top of the wastewater;
- to allow some degradation of the accumulated organic matter, and some die-off of any pathogens in the wastewater; and
- to provide storage space for the accumulation of sludge and scum.

The physical and bio-chemical processes occurring in the tank are described below.

3.1.1 Inflow wastewater characteristics and tank environment

Wastewater inflow to a domestic septic tank is not uniformly distributed throughout the day. As with collective systems, there are generally flow peaks in the morning and evening when the greatest water use occurs, and an almost total flow absence during the night (Metcalf and Eddy, 1979). In a septic tank, the flow peaks are sharper than in collective systems because they respond directly to every input from the house. The total volume discharged to the tank depends on the number of people served by the system, the facilities and appliances in the house (such as baths, showers, laundry facilities, and dishwashing machines), water saving facilities such as dual flush toilets, and the nature of the water supply (reticulated town supply, roof-top collection, groundwater).

Daily volumes of wastewater generated in New Zealand households have been estimated as 140 -180 litres per person (Gunn, 1989). This is similar to daily production estimated for on-site systems in the United States, which is 170 litres per person, with a theoretical design flow, which includes a safety factor, for ST-SAS there being 570 litres per day per bedroom (Canter and Knox, 1985).

Domestic sewage entering a septic tank has generally travelled a short distance in the pipes from the source and is therefore little changed in composition from its original form. This contrasts with domestic sewage treated in a collective water borne sewage treatment system where influent composition, particularly after pre-treatment screening of rags and grit, is relatively homogenous.

The constituents of wastewater entering a septic tank depends on whether all household facilities and appliances are connected to the system. In New Zealand this is usually the case although it is possible for separate greywater systems to be installed. Some characteristics of untreated domestic wastewater are given in Table 2. The constituents are primarily organic, with some inorganic contaminants including phosphorus, nitrogen, chlorides and metals.

Table 2. Character of untreated household wastewater

Characteristic	**NZ data (1) (Close, 1989) (g/m^3)**	**UK data (2) (Gray, 1989) (g/m^3)**	**US data (3) (Canter and Knox, 1985) (g/m^3)**
COD		650	
BOD_5	910	326	241
Total solids			1,128
Suspended solids		127	200
Total nitrogen	28	66	
oil and grease			21

(1) averaged data of samples taken from comminuted sewage (without stormwater infiltration) from a military camp of 2,000 to 5,000 people.

(2) typical composition of raw wastewater in the UK.

(3) averaged data of samples of influent wastewater to a septic tank in studies reported by Lawrence (1973) in Canter and Knox (1985).

There is very little data in the literature about septic tank environments with regard to temperature, dissolved oxygen, and pH ranges and variations, The most comprehensive study was undertaken by Winneberger (1984), who spent eight years investigating the operation of American septic tanks during the 1960s. Some of his findings are given below.

Dissolved oxygen (DO) is present in septic tank wastewater and levels vary with depth and proximity to the inlet pipe. Winneberger (1984) measured DO in twelve septic tanks and found that levels ranged from less than 0.1 mg/l up to 0.25 mg/l. DO levels were found to decrease with depth and increase with wastewater inflow. The levels he measured are only about one percent of saturated oxygen levels (9.1 mg/l at 20°C; 10.1 mg/l at 15°C) but they demonstrate that septic tank environments are not entirely anaerobic. They can also be partially aerobic.

Influent flows are generally warm and septic tanks display temperature variations both with depth and with seasons. Winneberger (1984) found that temperature decreased with depth and increased according to wastewater inflow. Temperatures were generally in the range 18.5-22.1°C, and ranged between 16-26°C seasonally.

Measurements of pH show a profile that decreases with depth from between 7 and 8 near the top, to between 5 and 7 near the bottom (Winneberger, 1984). Lower pH levels towards the bottom of the tanks may be caused by dissolved carbon dioxide released in the anaerobic digestion process, and also by the activity of non-methanogenic bacteria (acid-formers), which reduce wastewater pH (Section 3.1.4).

The movement of septic tank effluent from deep disposal wells into alluvial water bearing gravels in the Canterbury Plains was investigated by Sinton (1986), and Close (1989). The influent, which originated from a military camp of 2,000 to 5,000 people, was comminuted prior to being discharged into the tank in hourly 66 litre loads, fifteen times per day (to approximate the discharge from a five-person household). The original influent composition would not be representative of a typical household, and its homogeneity may have resulted in a different sedimentation rate of the settleable solids than would be achieved in a septic tank in 'normal' operation receiving wastewater from a single household.

Septic tank effluent characteristics measured in the Canterbury study, from an Imhoff tank, which serves about thirty households near Eketahuna, and from two overseas studies, are shown in Table 3.

Table 3. Effluent Quality from Septic Tanks.

	Mean concentration from Eketahuna Imhoff tank N=12 (MWRC, 1994-95)	**Mean concentration from one tank - NZ data, N=23 (Close, 1989)**	**Mean concentration from 30 tanks - US data (Converse, *et al.*, 1991)**	**Mean concentration from one tank - Canadian data (Cullimore & Viraraghavan, 1994)**
COD			291 (163)	293 (104)
BOD_5	27.2 (8.9)	560 (380)	150 (54)	178 (71)
NO_3-N	0.25 (0.43)	0.4 (0.6)		
NO_2-N	0.03 (0.03)	0.34 (0.064)		
NH_4-N	9.2 (3.9)	15.11 (5.6)	48 (18)	22.3 (1.1)
TKN		20.63 (10.03)		32.2 (1.7)
P	2.0 (1.0) [DRP]	3.54(1.15)[DRP]	5.0 (1)	6.8 (0.6) [PO_4]
SS	23 (8.9)		99 (102)	76 (54)
Faecal coliform N/100 ml	10^6	# 10^6	10^8	

from Sinton, L. W. (1986). Data from the same study reported separately.

All concentrations except faecal coliform in (g/m^3). Standard deviations are given in parentheses.

The differences in these data are not unexpected. The Eketahuna tank receives domestic sewage from thirty households rather than one, and provides different sedimentation opportunities and more effective hydraulic retention times than a septic tank; Close's data reflect the composition of comminuted sewage from a collective system after treatment in a standard New Zealand septic tank; Converse's data reflect averaged data from thirty different septic tanks in Wisconsin, USA; and Cullimore and Viraraghavan's data reflect averaged data from a septic tank serving a single household in Saskatchewan, Canada. Even tanks from the same studies displayed wide variations in effluent contaminant concentrations (note standard deviations). For example, suspended solids in Cullimore and Viraraghavan's study ranged from 20 to 298 mg/l, and BOD ranged from 34 - 345 mg/l. Winneberger (1984) emphasised that septic tank characteristics and environments vary widely and cautioned the application of 'average' data to the general case.

Winneberger (1984) concluded from his studies that there are considerable variations in wastewater characteristics and volumes entering septic tanks, and treatment achieved in them, even among systems in the same neighbourhood. Factors influencing the wastewater characteristics and environment in the tank include the number of people served by the tank, their particular routines and culture in terms of diet, washing frequency (including themselves, dishes, and laundry), detergent and cleanser use, as well as the physical aspects of the tank and external environmental influences such as climate. Of these factors, only the physical aspects of the tank can be controlled by a design engineer. These aspects should designed to reduce the variability of septic tank effluent and to achieve an effluent quality that is consistently high.

3.1.2 Sedimentation

One of the three primary functions of the tank is to provide an opportunity for sedimentation. This removes the solids and some of the organic matter in the wastewater, making the effluent more suitable for infiltration into the soil. The removal of settleable solids in sewage treatment plant sedimentation tanks depends on the influent suspended solids, the hydraulic retention period, the overflow rate, throughflow and settling velocities, and tank shape (Anderson, 1981). Removal efficiency is increased with increased retention periods and reduced with increased overflow rates. These same factors are relevant to solids removal in septic tanks.

Septic tank influent suspended solids concentrations respond directly to the input from the household and can be reduced by reducing food scrap waste. Garbage grinders, for example, can contribute a high proportion of total solids, and have been estimated to increase the BOD load by fifty percent (Krebs, 1974). Septic tank influent is less homogeneous than influent to sedimentation tanks in collective systems. In effect, much of the settleable solids could behave as flocculated solids, allowing better sedimentation than would occur in a collective system in the same time.

In a septic tank the hydraulic retention time (HRT) is directly related to wastewater inflow. That is, while the average wastewater daily inflow may be 140-180 litres per person, this is discharged into the tank in concentrated shock loads. Canter and Knox (1985) reported that because daily flow into a septic tank actually occurs over only about four hours, the HRT is reduced to only a few hours. Shock loads with high flow rates can also disturb settled solids so that they are carried out with the effluent. The overflow rate from a septic tank responds directly to the inflow rate. Throughflow is affected by the length of the tank, and baffling and compartments within the tank.

Four types of settling can occur in wastewater treatment systems (Metcalf and Eddy 1979). These are discrete particle settling, flocculent settling, hindered settling and compression settling. Discrete particle settling is the sedimentation of particles in a suspension of low solids concentration. Flocculent settling is the improved sedimentation that occurs when suspended particles coalesce in the wastewater and settle with a greater velocity than they would independently. Hindered settling usually occurs in wastewaters with intermediate concentration where particles tend to remain in fixed positions relative to each other and settle en masse. Compression settling usually occurs in the lower layers of a more highly concentrated deep sludge mass.

Discrete settling is likely to predominate in a septic tank because of the dilute nature of the influent flows, but, because of the non-homogeneous composition of the influent, some particles may settle as per flocculent settling. Agents to aid flocculation in septic tanks are available commercially in the United States (Winneberger, 1984), but do not appear to be widely used for on-site sewage treatment in New Zealand. Aluminium sulphate is used in municipal systems to encourage conglomeration of colloidal materials, but it lowers the pH and may not be suitable in septic tanks where there is no professional maintenance of the operation. Sachets of freeze-dried bacteria, reportedly selected for their floccing characteristics, are available for use in septic tanks. These bacteria do not cause the solids to coalesce but work by consuming organic matter before they themselves settle (Contamination Control, 1995). This contrasts with enzyme additives, varieties of which are selected to feed on specific pollutants and which do not generally settle out but are discharged to the soakage field with the effluent. There is no scientific information in the literature confirming the effectiveness of any septic tank additives.

Anderson (1981) analysed the settling characteristics of sewage following preliminary treatment at sewage treatment plants and pilot scale plants in England. He concluded that for retention periods of six hours the settleable solids removal efficiency was only slightly reduced when the flow rate was doubled.

Factors relevant and specific to septic tank sedimentation are the composition of the influent wastewater and the low actual HRT. Septic tank design factors should include methods to reduce flushing out of settled solids, and methods to provide sufficient actual HRT that relates to the short-term hydraulic loading to the tank, rather than the daily average hydraulic loading. Septic tank design factors affecting the concentration of suspended solids in septic tank effluent are the size and shape of the tank, the degree of compartmentalisation, and the frequency of pumping (Troyan *et al.*, 1984). The tank size and shape, and degree of compartmentalisation affect the wastewater hydraulic retention time and the retention of settled solids. The frequency of pumping affects the likelihood of settled solids being flushed through the tank. These design factors are discussed in Section 3.2 below.

3.1.3 Grease, floatables and the scum mat

Retaining solid materials and floatables is one of three key functions of the tank. These constituents accumulate on top of the wastewater in the tank and if they are allowed to be flushed out of the tank they will accumulate in the soil absorption area instead. Soil absorption areas, designed so that clarified effluent can drain into the upper soil profile, then become clogged and cannot perform their functions of treatment and disposal.

The scum mat on the surface of the wastewater in the tank can include grease, oils, fats, soaps, food wastes, hair, paper and cotton, and similar materials (Metcalf and Eddy, 1979). The leathery character of scum mats, thought to be caused by vegetable moulds, principally fungi, can support larger aerobic creatures such as earthworms and insects (Winneberger, 1984). The scum mat in tanks is not broken down ~~by~~ to any great extent by microbial action in the tank.

The tendency for any tank to form a scum layer is highly variable (Winneberger, 1984). There was a complete absence of any scum layer in almost all of fourteen on-site systems, all dual tanks with baffles, monitored over a two year period in the Netherlands (van der Graaf *et al.*, 1989). The build-up of a scum mats in septic tanks is also variable in the Manawatu area. While most tanks have some scum layer, one tank was observed without any mat even after three years of use, and another with a mat ten cm thick after less than six months of use. As in the United States, the tendency of a scum mat to form in a septic tank appears to depend on the culture of the householders (Roswell, personal communication).

Observations by sanitary engineers early this century concluded that the establishment of the scum mat was influenced most strongly by the amount of gas produced by the sludge layer (Winneberger, 1984). In laboratory conditions, large agglomerated particles were observed to be buoyed by gas bubbles and attach to the underside of the scum mat (Winneberger, 1984). This is likely to be CO_2 and CH_4 produced during anaerobic digestion of the sludge (see Section 3.1.4 below) although it is possible that some denitrification may also occur in the tank.

The septic tank design feature affecting the retention of scum is the depth of the outlet tee. This is discussed in Section 3.2.3 below.

3.1.4 Anaerobic digestion

Anaerobic digestion of sewage is the process of decomposition of organic matter in the absence of molecular oxygen. In a septic tank the digestion process is unheated and unmixed. This process is often referred to as standard rate anaerobic digestion. The shift to heated digestion for collective sewage treatment systems means that there is a shortage of consistent operational data available on the performance of cold digesters (IWPC, 1979).

In the predominantly anaerobic conditions in the tank, micro-organisms use electron acceptors such as organic carbon, H^+, CO_2, and SO_4^{2-} to oxidise organic matter and produce CO_2, H_2, CH_4, and S^{2-}. This can be represented by the following chemical equations (Wilhelm *et al.*, 1994):

Organic molecule hydrolysis:

Proteins + $H_2O \rightarrow$ Amino acids

Carbohydrates + $H_2O \rightarrow$ Simple sugars

Fats + $H_2O \rightarrow$ Fatty acids and glycerol

Ammonium release:

Urea $[CO(NH_3^+)_2] + H_2O \rightarrow 2\ NH_4^+ + CO_2$

Amino acids + $H_2O \rightarrow NH_4^+$ + Organic compounds

Fermentation:

Amino acids, simple sugars $\rightarrow H_2$, acetate (CH_3OO^-), other organic acids

Anaerobic oxidation:

Fatty acids + $H_2O \rightarrow H_2$, acetate (CH_3OO^-)

Sulphate reduction:

$SO_4^{2-} + 2\ CH_2O^a + 2\ H^+ \rightarrow H_2S + 2\ CO_2 + 2\ H_2O$

Methanogenesis:

acetate $(CH_3OO^-) + H^+ \rightarrow CH_4 + CO_2$

$CO_2 + 4\ H_2 \rightarrow CH_4 + 2\ H_2O$

The constituents of septic tank gases confirm that these processes occur in the tank (Winneberger, 1984). The biological conversions occur in a staged process. In the first stage, higher-weight molecular compounds are transformed into compounds suitable for sources of energy and cell carbon, which are converted by bacteria into lower-weight compounds. These compounds are then converted by different bacteria into simpler end products, mainly methane and carbon dioxide. The organisms responsible for the first part of the process are collectively called non-methanogenic, or "acid-formers". The second group, which are strict anaerobes, are called methanogenic, or "methane-formers".

In the non-methanogenic phase, organic material is degraded mostly to saturated fatty acids, carbon dioxide, hydrogen, and ammonia, with smaller amounts of alcohols, aldehydes, and ketones produced. During this stage there is an increase in bacterial cellular biomass (IWPC, 1979).

Methanogenic bacteria can use the lower fatty acids containing at least six carbon atoms (formic, acetic, propionic, butyric, valeric, caproic), the normal and isoalcohols containing from one to five carbon atoms (methanol, ethanol, propanol, butanol, and pentanol), and three inorganic gases (hydrogen, carbon monoxide, and carbon dioxide) as substrates. The methanogenic group that degrade acetic acid and propionic acid have very slow growth rates. The anaerobic treatment of organic waste is limited by the metabolic rate of this group (Metcalf and Eddy, 1979).

Only a small amount of the degradable organic matter is synthesised into new cells because of the low growth rate of the bacteria responsible for the fermentation of acetic and propionic acids. Most is converted into methane, which is highly insoluble, and is released from the tank through the inlet and outlet pipes and cracks around the access hole, representing a net loss from the system.

The low cellular growth rate, and the net loss of degraded organic matter from the system as methane or carbon dioxide, result in a sludge that is better stabilised than that from an aerobic treatment system. Sludges from aerobic systems have a higher proportion of cellular material than sludges from anaerobic systems.

Anaerobic treatment is performed by fast growing acid-forming bacteria and slow-growing methane producing bacteria. An efficient anaerobic wastewater system needs to maintain the non-methanogenic and methanogenic bacteria in a state of dynamic equilibrium. Factors adversely affecting the maintenance of such a state include the presence of dissolved oxygen, and pH values below 6.6 or above 7.6 (methane bacteria cannot function below a pH of 6.2). The environment should be free from inhibitory concentrations of heavy metals and sulphides, and should include sufficient nitrogen and phosphorus. The mesophilic and thermophilic temperature ranges (30 - 38°C and 49 - 57°C), are the optimum temperature ranges for anaerobic treatment (Metcalf and Eddy, 1979).

As described in Section 3.1.1 above, dissolved oxygen is present in septic tank wastewater in very small quantities, the pH may be up to 8 at the top of the tank, and as low as 5 at the bottom of the tank, and the temperature in the tank is not in the optimum temperature range.

The lack of optimum temperature for anaerobic digestion in a septic tank is not a strong disadvantage. Operating the system at lower than optimum temperatures can be compensated by increasing the mean cell residence time, in this case the sludge detention time in the tank. The minute quantities of dissolved oxygen present at the bottom of the tank should likewise not present a strong disadvantage. The lower limit of pH values at the bottom of the tank could, however, present a disadvantage for the survival of the methane bacteria. This could result in a lower digestion rate of the sludge and a faster sludge build-up rate in the tank.

Nitrogen and phosphorus requirements for microbial growth are C:N:P ratios of 100:5:1. Available nutrients are not a limiting factor in domestic sewage systems because the ratio in raw sewage is about 100:17:5, and in settled sewage is about 100:19:6 (Gray, 1989).

Inhibitory material such as heavy metals and halogenated hydrocarbons may be present in municipal systems from industrial waste in the waste stream but are not likely to be present in a domestic on-site system. The only inhibiting material likely to be in a septic tank is anionic detergents. These will cause serious inhibition at concentrations greater than 2 percent of dry solids concentrations. The inhibitory effect from non-anionic and cationic detergents is very slight even at high concentrations (IWPC, 1979). Few New Zealand detergents state their constituents. Two that do, Palmolive and Down to Earth, are anionic. Detergent concentrations in New Zealand septic tank sludge have not been reported in the literature.

Wastewater quality monitoring of 14 on-site systems in the Netherlands showed that high COD effluent concentrations and low COD reductions were due to a high content of organic fatty acids in the effluent (van der Graaf *et al.*, 1989). They reported that the "higher than anticipated levels of heavy metals constituted a drawback with regard to the disposal of the sludge" (van der Graaf *et al.*, 1989, page 6). The report does not state which heavy metals were present, or in what quantities, but their presence could have been sufficient to cause inhibition of methanogenic bacteria.

Detention times of between 30 - 60 days are recommended for large scale standard rate anaerobic sludge digesters. In those systems, sludge retention of less than 12 days presents the danger that the slow growing methanogenic bacteria will be washed out with the sludge. A domestic septic tank should be designed to contain the sludge, therefore there would be no loss of these bacteria until desludging. Septic tank design features affecting the retention of sludge are outlet baffles and the degree of compartmentalisation. These are discussed in Section 3.2.2 below.

Anaerobic attached growth treatment by the incorporation of an anaerobic filter in the tank is a modification of the anaerobic digester design. Anaerobic filter technology was developed mainly for wastewaters with high organic strength such as piggery effluent (Hashieder and Sievers, 1984) and food industry wastewater (Ehlinger *et al.* 1988). This is because the high quantities of methane produced during the digestion of high strength wastes can be used to heat and maintain the digester temperature in the mesophilic range, enabling a very efficient operation. The application of anaerobic filters in the treatment of low strength wastewater such as domestic sewage has been investigated and developed more recently (Kobayashi *et al.* 1983; Switzenbaum *et al.* 1984; Viraraghavan, 1986; and Noyola *et al.* 1988).

Anaerobic attached growth treatment in a septic tank is achieved by installing various kinds of solid media in the tank. Suitable media for incorporation in a septic tank include rocks, plastic rings, and screen filters. Anaerobic bacteria attach to and colonise the solid media and high mean cell residence times can be achieved even with short hydraulic retention times. The media provides the biofilm with increased surface area to grow on and reduces the risk of bacteria and suspended solids being washed out with the effluent (Cullimore and Viraraghavan, 1994).

Young (1983) described four "somewhat unique" abilities of the anaerobic filter based on tests from laboratory-scale units. These abilities are the filter's effective treatment of relatively low strength wastewater at low temperatures without the need for solids settling and recycling, their insensitivity to severe shock loads and variable loading rates, more efficient treatment of high organic loads than mixed digesters, and the low net production of biological solids resulting in lower requirements for solids wasting. The first two of these four abilities make anaerobic filters particularly suitable for incorporation into septic tanks.

Kobayashi *et al.* (1983) demonstrated the ability of anaerobic filters to treat domestic wastewater of widely varying strengths to a consistent effluent quality. While influent COD concentrations varied up to four times the mean of 288 mg/l, and influent suspended solids varied up to almost five times the mean of 118 mg/l, effluent COD concentrations were in the range 55-121 mg/l with an average of 78 mg/l and effluent suspended solids concentrations were in the range 15-50 mg/l with an average of 32 mg/l. The anaerobic column was maintained at temperatures of 25°C and 35°C but the results showed that the BOD_5 and COD removal efficiencies for the two temperatures were not statistically different at the 5% level of confidence.

Anaerobic filters were trialed for the treatment of dairy wastewater by Viraraghavan and Kikkeri (1990). Suspended solids, BOD and COD removal increased with increased hydraulic retention times and increased temperature. Suspended solids removal between 70 and 85 percent was achieved at 12.5°C and an hydraulic retention time greater than six days. Organic removal efficiencies were lower at lower temperatures, but the differences in removal efficiency were not significant when hydraulic retention times were greater than four days.

Similar results were achieved with the treatment of septic tank effluent by upflow anaerobic filters in laboratory conditions (Cullimore and Viraraghavan, 1994). Significant biomass was generated at temperatures of 5°C, 10°C, and 20°C, and BOD removal increased with hydraulic retention times and temperature. Overall, longer HRT in all reactors out-weighed the otherwise better performance of the 20°C reactor. The best performance in BOD removal was achieved in all reactors with the highest HRT, which was 3.17 days (75%, 86%, and 86% respectively for temperatures of 5°C, 10°C, and 20°C). The COD removal demonstrated a less regular trend, but was also best with the longest HRT (52%, 62%, and 65% respectively).

The consistent level of treatment makes the anaerobic filter especially useful for low maintenance wastewater treatment, and for the variable waste stream from individual homes. These features are particularly desirable for on-site domestic sewage disposal systems. There is one New Zealand septic tank design, the Humes Ecotank™, which incorporates an upflow anaerobic rock filter. Waste characteristics from an Ecotank™ studied at Lincoln University over a three month period showed about 80 percent removal of suspended solids, with effluent suspended solids concentrations in the range 13 - 20 mg/l; and up to 70 percent reduction in COD, with effluent COD concentrations between 141 -300 mg/l (Dakers and Khan, 1993, unpublished report).

The use of upflow filters has also been investigated for their usefulness in removing nitrogen during the treatment process. Hanaki and Polprasert (1989) investigated methane production and denitrification in upflow filters receiving methanol and nitrate in laboratory conditions. The reactors were kept in the temperature range 25 - 30°C and were fed synthetic wastewater continuously. The reactors achieved nitrate removal efficiencies of 99.8 percent with a methanol:nitrate ratio greater than 2.3:1 and hydraulic retention times greater than 1.5 hours. Surplus methanol, after completion of denitrification, was converted to methane. If used for this purpose as part of an ST-SAS there would need to be a nitrifying stage for the septic tank effluent prior to de-nitrifying in the upflow filter.

The treatment performance of septic tank effluent screen filters is not reported in the literature. Screen filters, installed at the effluent pipe, would provide less surface area for bacteria to colonise than tanks packed with solid media. Further, they may be less able to damp the hydraulic surges that characterise on-site wastewater loads.

3.1.5 Sludge accumulation

Septic tank sludge comprises the settleable solids of the wastewater. A typical domestic primary sludge is greyish black, has an offensive odour, and contains about 5% dry solids of which about 70-80% is organic and volatile matter (IWPC, 1979). Some characteristics of septic tank sludges are given in Table 4 below.

Table 4. Contaminants in septic tank sludge (Brandes, 1978).

Contaminant	Dual tank receiving toilet wastewater only		Dual tank receiving toilet, bathroom, kitchen and laundry wastewater	
	Tank 1	Tank 2	Tank 1	Tank 2
COD	not tested	not tested	35,600	44,200
BOD_5	6,000	380	13,500	15,000
Total solids	23,350	620	33,550	28,495
Total Kjeldahl (as N)	2,200	170	630	650
Ammonia (as N)	19	22	88	92
Total phosphorus (as P)	610	18	170	160
Soluble phosphorus	1.7	2.8	33.0	19.0
Total coliform organisms/100 ml	0.9×10^6	0.42×10^6	16×10^6	16×10^6
Chlorides (as Cl)	50	57	78	83
Sulphates (as SO_4)	28	19	21	17
Aluminium (as Al)	5.3	0.29	14	4
Iron (as Fe)	160	0.75	50	70
Calcium (as Ca)	66	22	56	82
Magnesium (as Mg)	10	6	41	36
Sodium (as Na)	55	53	89	82
Potassium (as K)	22	20	26	28

Note: all data except coliform organisms are given in mg/l.

Providing storage for accumulated sludge is one of the three key functions of the septic tank. The space taken up by accumulated sludge decreases the space available for hydraulic retention of the wastewater, and high levels of accumulated sludge increase the risk of solids carry-over with the effluent. The studies reviewed below show that accurately assessing sludge accumulation rates is an important performance factor because too frequent pumping out may be as undesirable as infrequent pumping.

The rate of sludge accumulation in the tank has been assessed in American studies and reported by Brandes (1978). He found that longer hydraulic retention times resulted in greater decomposition of organic matter, which was eventually removed from the tank either through the vent in gaseous form or with the effluent in liquid form. Gaseous discharges (principally hydrogen sulphide, ammonia, carbon dioxide and methane) represent a net removal from the system.

Brandes (1978) studied three different systems, each with different loading rates and waste composition, to determine the effect of septic tank capacity and wastewater strength on the sludge composition and accumulation rate. The highest accumulation rate of 0.29 litres per capita per day was observed in the tank with the shortest hydraulic retention time (1.9 days compared to 2.4 and 9.7 days). Further, waste in that tank was only partially decomposed.

Brandes (1978) estimated that 0.275 litres per capita per day (100 litres per capita per year) would be representative of a typical accumulation rate. This is higher than average sludge accumulation rates of 0.19 litres per capita per day (69 litres per capita per year) determined from 205 septic tanks in the United States after one year of operation (Weibel, 1949, cited in Brandes, 1978), and two cubic feet (57 litres per capita per year) allowed for by Winneberger (1984).

Philip *et al.* (1993) measured sludge accumulation and composition in thirty-three septic tanks over three years in France. They found that sludge accumulation rates stabilise after the first year of operation, decreasing from 0.35 litres per capita per day to less than 0.2 litres per capita per day (73 litres per capita per year). They also found that soluble COD concentrations in the sludge increased during the first two years of operation then rapidly declined. Methane production showed a corresponding increase after two years.

Gray (1995) measured sludge accumulation rates in twenty-eight septic tank systems over a three week period and calculated the sludge accumulation rate for the specific age of the sludge on a per capita basis. The study was undertaken in Ireland where water usage was estimated to be only 65 litres per person per day. Gray concluded that the rate of sludge accumulation in a tank decreases with time. That is, while accumulation rates for the first six months of tank use would be 92.7 litres per capita per year (0.25 litres per capita per day), they decreased to 64.9 litres per capita per year (0.18 litres per capita per day) after sixty months. The decline in sludge accumulation rates over time was attributed to an increase in solids decomposition, increased compaction of settleable solids, and solids carry-over from the tank.

Sludge and scum accumulation rates in forty operating septic tanks in Perth, Australia ranged from 35 litres per capita per year to 85 litres per capita per year. The 90 percentile rates were 48 litres per capita per year of sludge accumulation and 32 litres per capita per year for scum accumulation (Troyan *et al*., 1984). Theoretical hydraulic retention times in the tanks surveyed ranged from 0.5 to 8.3 days.

Sludge accumulation rates of 0.3 - 0.5 litres per capita per day (110 - 180 litres per capita per year) were estimated for compartmentalised septic tanks in the Netherlands (van der Graff *et al*., 1989). Eighty to ninety percent of this sludge settled in the first compartment, which had twice the volume of the subsequent compartment. The reported high concentrations of heavy metals could have contributed to this high accumulation rate, relative to the other studies. Heavy metals inhibit the activity of methanogenic bacteria, causing faster sludge build-up.

These reported sludge accumulation rates (excluding van der Graaf *et al*., 1989) range from 48 to 100 litres per capita per year, with two studies indicating that the rate would decrease with time. Larger tanks not only provide greater hydraulic retention during the first years of operation, they also provide more storage for sludge during the later years of operation when the sludge accumulation rate is less. The principal design factors affecting sludge retention are the size, shape and amount of compartments and baffling in the tank. These are discussed in Section 3.2 below.

3.1.6 Pathogen removal and die-off in the tank

Disease causing organisms, or pathogens, that can be present in sewage are bacteria, viruses, protozoa, and the eggs of parasites such as roundworm. Water-borne pathogens isolated most frequently in sewage are strains of *Salmonella*, *Shigella*, enteropathogenic *Escherichia coli*, *Francisella*, *Vibrio*, and *Mycobacterium*, human enteric viruses, cysts of *Entamoeba histolytica*, or other pathogenic protozoans, and larvae of various pathogenic worms (Gray, 1989).

Pathogens entering the septic tank can be killed in the tank, discharged in the effluent, or concentrated in the sludge. Destroying pathogens present in sewage, or intercepting their potential transmission routes, are two ways to reduce the spread of disease in the community. The amount of micro-organism die-off in any wastewater treatment process depends on the detention time, wastewater chemical composition, antagonistic forces in the biological flora, pH, temperature, and the application of any disinfection process (Hirn, 1980). There is very little information in the literature about pathogen survival in typical septic tank conditions but some inferences can be drawn from the results of studies of collective sewage treatment plants.

In raw sewage, 50-75 percent of the coliforms are associated with particles with settling velocities greater than 0.5 mm per second (Gray, 1989). These particles would fall one metre in 33 minutes in quiescent conditions and accumulate in the sludge in the tank. Protozoa have poor settleability and often pass through community sewage treatment plants. Viruses are not effectively removed by short-term settling in community plants, but one study showed that between 33-67 percent of poliovirus type 1 were removed after a 24 hour settling time (Berg, 1966 in Gray, 1989). This decrease could be attributed to die-off during that time, rather than removal by settling. These factors indicate that overnight quiescent conditions in a septic tank should assist in bacterial removal but may be less useful for virus and protozoa removal.

Bacteria tend to have shorter survival in wastewater than enteric viruses because their cellular nature makes them more susceptible to the environmental conditions in the tank. Studies show that *S. typhi* die-off in anaerobic digesters is slower at lower temperatures, taking 12 days at 20°C and 10 days at 30°C (Gray, 1989).

Salmonella, a bacteria species with over 1,800 identifiable subtypes, are pathogens of nearly all animals, including humans (Gray, 1989). Because of their widespread incidence in the environment, *Salmonellae* tend to be the most numerous pathogenic bacterial group present in sewage. The occurrence and survival of *Salmonellae, Escherichia coli, Mycobacteria, Leptospires*, and *Brucella abortus* and *Bacillus anthracis* was investigated in eight sewage treatment plants in England (Lewin *et al.*, 1981). All plants incorporated at least secondary treatment by activated sludge or biological filters. Samples were analysed from settled sewage, final effluent, raw sludge, digested anaerobic (mesophilic) sludge, digested aerobic (cold) sludge, and processed sludge. *Salmonellae* were isolated in the highest quantities from the raw sludge, the anaerobically digested sludge, and the settled sewage. The highest numbers were present in the raw sludge indicating that *Salmonellae* are "preferentially co-settled with the solids during primary sedimentation" (Lewin *et al.*, 1981). These results indicate if there are any pathogenic bacteria in the septic tank influent, they will be present in high quantities in the settled sludge. Improving settling opportunities and retaining the sludge should decrease the number that reach the wider environment.

Any organisms retained in the sludge could be expected to die after a period of days or weeks but re-suspension of the sludge, either buoyed by sludge gases or disturbed by influent flows, could allow settled organisms to be flushed out of the tank. The infectious dose for some bacteria is very low, for example, the infectious dose for *S. typhi* is between 1 and 10 /100 ml (Jay, 1992) and so promoting sedimentation and retaining sludge in the tank contributes not only to pathogen destruction but to interrupting potential disease transmission routes.

Enteroviruses and hepatitis types are present in high concentrations in human sewage. Their presence poses a significant health risk when faecally contaminated water is consumed because the infectious doses from these viruses can be very small (WHO, 1981). Many viruses present in raw domestic sewage are believed to be absorbed or embedded in solids and so the removal rate of viruses in septic tanks is strongly associated with solids removal. Virus removal during primary settling is in the range 23 - 81 percent (Gerba, 1984).

Anderson *et al.* (1991) monitored human enteroviruses at eight septic tank systems in Florida. Virus presence in faecal stool specimens from residents served by septic tanks was monitored, as was virus presence in the septic tank effluent, and in the soil systems directly below the infiltration system. The results showed that, in general, viral serotypes present in the stools (*Poliovirus*, *Echovirus*, *Coxsackievirus*, and *Adenovirus*) were also present in the septic tank effluent. The removal efficiency could not be determined from their study because they did not measure influent concentrations, but the Most Probable Number of infectious units per litre of effluent ranged from 0.06 to more than 43.7, with *Coxsackievirus* present in the highest numbers.

Polprasert and Hoang (1983) studied the removal of faecal coliforms and bacteriophages by anaerobic rock filters in septic tanks. They concluded that bacterial removal was in accordance with a first order reaction and increased with the hydraulic detention time in the tank. Their results showed that a four day detention achieved almost complete removal of faecal coliforms but a lower bacteriophage removal.

Cullimore and Viraraghavan (1994) assessed coliform removal by upflow filters at various temperatures and hydraulic retention times. Faecal coliform removal was greater at 10°C and 20°C than at the lowest temperature of 5°C, but total coliform removal rates were less sensitive to temperature.

3.1.7 Treatment performance of the tank

Septic tank effluent quality depends on the influent wastewater quality, and the treatment performance of the tank. The influent quality varies among households and could be improved by educating tank owners about the benefit of reducing input of food scraps into the tank. Tank treatment performances are also variable, being dependent on factors such as hydraulic retention periods, retention of sludge, and the presence of filters.

Poor treatment performance is characterised by high suspended solids and BOD in septic tank effluent. These factors increase the formation of a biological mat in the soil absorption area (see Section 4.1.1 below) and diminish the rate at which the soil can absorb the effluent.

Wide variations for BOD_5 removal rates in septic tanks from zero to seventy percent have been reported in the literature (Martens and Warner, 1995). Some tank designs achieve higher removal efficiencies, in particular, multi-compartment tanks have more potential to contain effluent solids, and reduce short circuiting and turbulence in the tank. Anaerobic filter reactors with hydraulic retention times greater than four days can achieve consistently high removal of BOD, suspended solids and bacteria.

Alhajjar *et al.* (1989) investigated septic system treatment of wastewaters containing PO_4- or CO_3- detergents. Both detergents were "white, granular, and contained normal-sudsing anionic surfactants". Eight householders in the study were supplied with PO_4- detergent, seven with CO_3- detergent. The 17 septic systems were sampled monthly for two years and analysed for physical, chemical and biological parameters. Their analysis showed that concentrations of filtered and unfiltered total solids, total volatile solids, total suspended solids, volatile suspended solids, indicator bacteria, and five-day biochemical oxygen demand were all higher in systems receiving PO_4- detergent, but that total nitrogen was removed more effectively from those systems.

Most organic nitrogen entering the tank is reduced to ammonia in the anaerobic conditions of the septic tank (Brown *et al.*, 1984, Wilhelm *et al.*, 1994) and is discharged with the effluent. Treatment performance should also have regard to the potential to minimise nitrogenous concentrations in septic tank effluent. This can be helped by ensuring that sludge is retained in the tank. About 34 percent of nitrogen in septic tank influent accumulates in the sludge at the bottom of the tank and in the scum mat (Laak 1974, cited in Winneberger, 1984). This is comparable with the 26 percent TKN removal in a septic tank in one New Zealand study (Close, 1989).

Both bacteria and viruses are associated with solid particles in the sewage. Retaining the sludge can also reduce quantities of pathogenic organisms discharged to the environment.

Examination of processes occurring in the tank shows that improving the performance of the septic tank requires —

- improving sedimentation of settleable solids;
- improving floatation of floatable matter;
- reducing short circuiting opportunities;
- improving degradation opportunities by providing optimal conditions, or at least removing inhibitory conditions; and
- improving storage space for sludge and ensuring that the sludge remains in the tank.

Septic tanks should be designed to recognise that —

- the influent comes in shock loads, generally grouped in part of the day;
- the influent is not homogenous; and
- there is a long sludge retention period.

Septic tank designs are discussed in Section 3.2 below.

3.2 Design of the septic tank

The standard septic tank has no moving parts, requires no energy for its operation, and has few maintenance requirements. A typical design is shown in Figure 2. This design has undergone little modification since its early utilisation about one hundred years ago. A septic tank design, the "Mouras Automatic Scavenger", was patented by Louis Mouras in France in 1881 (Winneberger, 1984). Some early designs are reproduced in Figures 3 and 4.

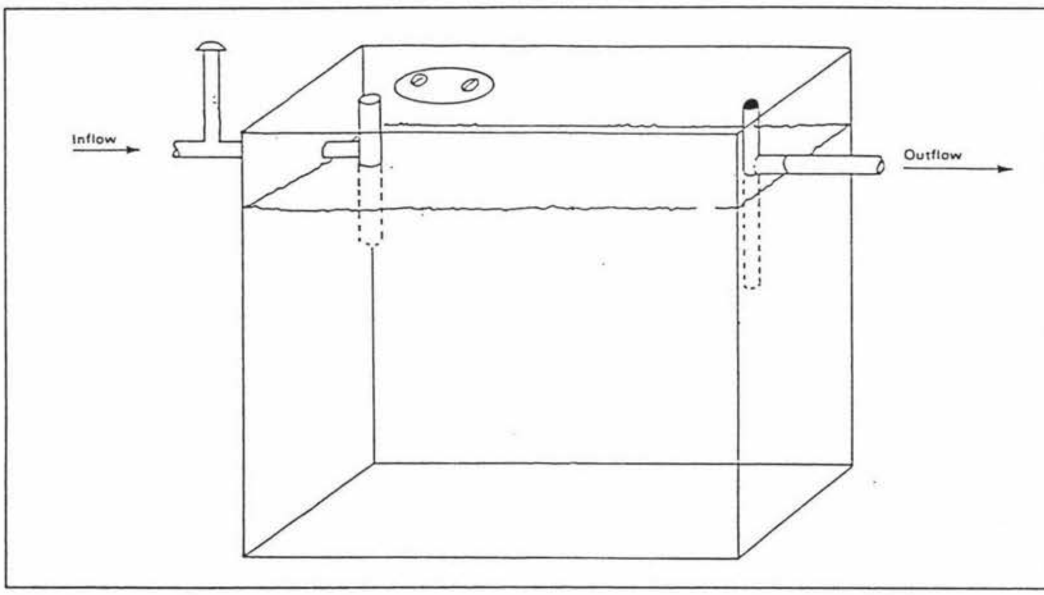


Figure 2. Typical New Zealand septic tank design (NZS 4610:1982). Note this figure is reproduced in full, including dimensions, in Figure 6.

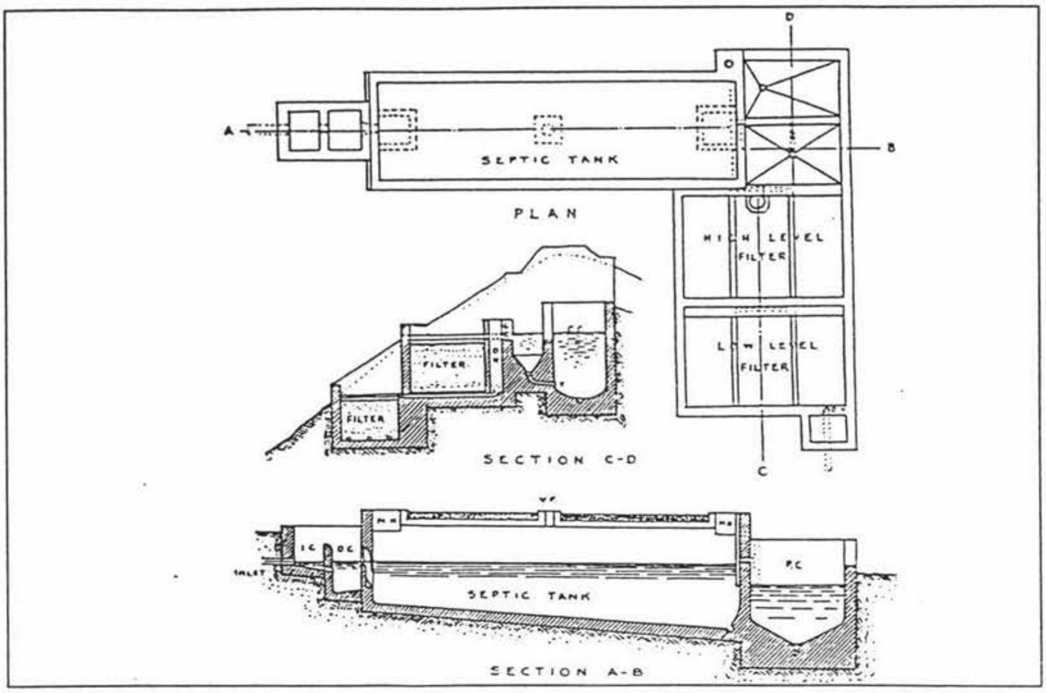


Figure 3. Septic tank system design circa 1920 (Flood, c1928)

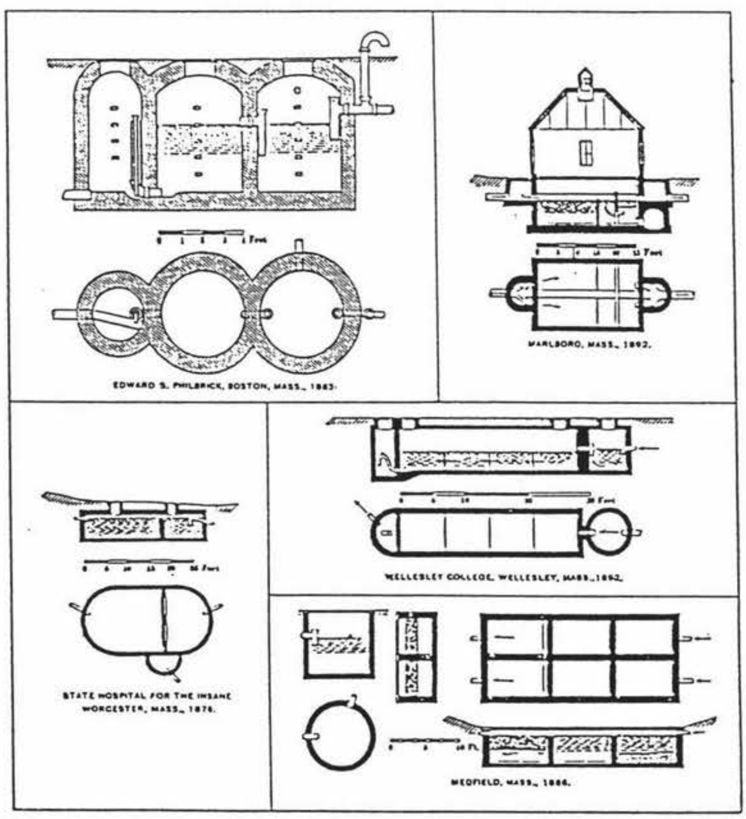


Figure 4. Septic tank design from last century (Winneberger, 1984, taken from Metcalf (1901) "The Antecedents of the Septic Tank").

Variations in tank designs affect its efficiency and effectiveness as part of a sewage treatment system. Winneberger (1984) found that while sanitary engineers at the turn of the century had an excellent understanding of septic tank functions, today's texts contain little rationale for tank designs and little information about the degree of treatment achieved in the tank. The decline in the use of these systems for collective community sewage treatment over the last seven or eight decades has apparently been paralleled by a decline in professional interest in their operation.

The physical engineering elements of the septic tank are discussed below.

3.2.1 Size and shape

New Zealand septic tanks are generally 2,700 litres or 3,300 litres (Gunn, 1989). American requirements for household septic tank volumes vary from state to state but are generally designed according to the number of bedrooms. The minimum volume required is 750 gallons (2,840 litres). Depending on the state, a five bedroom house requires up to 1,500 gallons (5,800 litres) and an extra 150 - 250 gallons capacity is required for each additional bedroom (Canter and Knox, 1985). Contemporary (American) septic tank specialists recommend that tank volumes should be larger than that required by the codes (Laak, 1986). The British Code of Practice requires a septic tank capacity of 2,000 + P180 litres, where P equals the population equivalent (Gray, 1995). 'Standard' British tanks, designed to serve four people, are 2,720 litres.

Large tank volumes, especially when combined with features to avoid short-circuiting, contribute to longer mean hydraulic retention times. Calculating a conservative hydraulic retention time by allowing for only half the tank to be available for wastewater (to discount space taken up by sludge, scum and air space) a 2,700 litre New Zealand tank would theoretically provide 58 hours wastewater retention for a four person household (assuming wastewater production of 140 litres per person per day), while a 3,300 litre tank would provide 71 hours retention for the same household.

Studies undertaken on a single non-baffled tank showed that this theoretical retention is, in fact, unlikely to be achieved in an operating septic tank. A water-filled tank, with a theoretical 72 hour detention availability, demonstrated only about 29 hours and 14 hours detention when subjected to intermittent and continuous feeding respectively (Winneberger, 1984). The shorter actual detention achieved was attributed to wastewater short-circuiting in the tank.

Nottingham and Ludwig (1948) measured septic tank performance as a function of tank length, width and depth by comparing effluent quality from three tanks over three months. The purpose of their study was to establish whether or not there should be an upper limit on the length:width ratio of septic tanks. Local codes required that septic tank lengths were at least two, but no more than three times the width. Many American state codes still have this requirement (Canter and Knox, 1985).

The three tanks examined in their study had length to width ratios of 2:1, 2.7:1 and 14.5:1. The highest ratio was achieved by connecting seven tanks in series. Nottingham and Ludwig found that during the three month period of the study, and with each tank receiving essentially the same sewage influent, the longer tanks removed 96 percent of settleable solids compared to 91 percent in the shorter tanks; and 89 percent of suspended solids compared to 80 percent in the shorter tanks. Biochemical Oxygen Demand removals were 71 percent and 60 percent respectively. They did not consider the difference to be significant and concluded that a tank with a length to width ratio anywhere in this range would suffice. This supported their preference for installing pre-cast tanks in series on-site because doing so at least did not diminish wastewater treatment achieved in the tank (Mark Nottingham and Harvey Ludwig were, respectively, a manufacturer of septic tanks and a consulting engineer). In fact their study shows that several tanks connected in series performed better than shorter tanks. This could perhaps be attributed to the influence of connecting the tanks in series, effectively providing baffles throughout the length and decreasing opportunities for wastewater short circuiting, as much as increasing the overall tank length.

Early sanitary engineers recommended that a septic tank should be large enough to accommodate between one and a half to two days wastewater flow, the length should never be less than six times the width, and the working depth of the tank should be four to six feet (1.2 to 1.8 metres) (Flood, c1928). Laak (1986) recommends that the tank length should be at least three times the width.

The historical reason for using long tanks, particularly when constructed with baffle boards, was to assist sedimentation and improve the uniformity of the effluent (Metcalf, 1901). Winneberger (1984) suggested that their lack of use today does not reflect any design improvement but has developed to reduce construction and installation difficulties with larger tanks. Septic tank fabrication today can take advantage of materials such as reinforced plaster and fibreglass, which were unavailable earlier this century, without decreasing the functional attributes of the tank. Plastic and fibreglass tanks are more resistant to corrosion and decay than concrete tanks, although they can have less structural strength unless reinforced (Canter and Knox, 1985).

Tanks installed in New Zealand in the 1940s and 1950s were often constructed from concrete blocks or bricks. Many were constructed in situ and have a rectangular shape, with a volume of 1,800 or 2,700 litres. The 1,800 and 2,700 litres tanks cost the same price but digging a hole for the smaller tank was easier and so they were commonly installed in preference to the larger tanks. They are about one metre deep, one metre wide, and two metres long with about 200 mm air space (personal observation). Some have lids of plastered corrugated iron or concrete slabs. These concrete tanks have limited structural integrity and many require replacing today because they are breaking up. The Bay of Plenty Regional Council found evidence in their region of septic tanks comprising only two 44-gallon drums (Environment BOP, 1994).

Modern septic tanks in New Zealand are commonly constructed from reinforced concrete, ferro-cement over wire mesh and, more recently, from fibreglass and high density plastic (polyolefin). They are generally constructed off-site. Small, light tanks are easier to transport, manoeuvre and re-site than longer, heavier tanks. Shorter, rounder tanks also have structural advantages in that they can better withstand sidewall pressure than longer, rectangular tanks.

The single tank is universally used throughout the Manawatu-Wanganui Region except in the Palmerston North area where heavy clay soils and low evapo-transpiration rates combine to impede effluent drainage. Because of this the City Council requires dual tanks so that effluent quality is improved and soil infiltration easier.

3.2.2 Baffles and multi-chambered tanks

Inclined baffles inserted in the tank near the outlet pipe were utilised at the turn of the century to help prevent solids buoyed by gases from escaping out with the effluent. An innovation early this century of compartmentalising a community tank with the use of baffleboards reportedly resulted in them achieving "three times as much work as they had previously done" (Metcalf, 1901). Winneberger recommended that their use in household septic tanks today be established (or re-established). Baffles, in his view, "especially gas baffles, improve performance, particularly in the commonly too short, stubby tanks" (Winneberger, 1984, page 54). Laak (1986) recommends that there should be both gas baffles and deflectors to reduce the potential for sludge buoyed by gases to escape from the tank.

Dual tanks, regardless of shape, achieve better reduction in suspended solids and BOD_5 than single chamber tanks (Troyan *et al.*, 1984). Multi-chambered septic tanks provide less opportunity for re-suspended sludge material to be flushed through to the soakage field, increase mean hydraulic retention times by decreasing short circuiting, and provide a more quiescent environment in the second tank for settling. Achieving more than one chamber by connecting single tanks in series combines the construction and installation convenience advantages of short tanks with the treatment opportunity advantages of baffled long tanks.

American research has shown that effluent from multi-chambered tanks has fifty percent less BOD and suspended solids than effluent from single chambered tanks (Laak, 1980). Compartmentalised tanks in the Netherlands achieved 18 - 54 percent reductions in BOD_5 and 48 - 98 percent removal of total suspended solids (van der Graaf *et al.*, 1989). Dual compartment tanks with a minimum detention time of twenty-four hours achieved 49 percent BOD_5 removal and 66 percent suspended solids removal compared to 22 percent and 42 percent in single chambered tanks (Troyan *et al.*, 1984).

A quiescent environment occurs in the second tank because influent wastewater velocity is less than that in the primary tank. The lower influent velocity reduces turbulence, which otherwise disrupts the sedimentation process by disturbing the settleable low density solids.

Laak (1980) reported the results of different studies which compared treatment efficiency, in terms of BOD and solids removal, of single and multi-chambered septic tanks, with and without baffles. Results showed that inlet and outlet baffles on single tanks, and increased hydraulic retention time in multi-chamber tanks, both contributed to improved treatment efficiency. Effluent from multi-chambered tanks had up to 50 percent less suspended solids and BOD than effluent from single tanks, and baffles were particularly effective at retaining suspended solids and settleable solids in single and multi-chambered tanks.

Multi-chamber tanks do not avoid wastewater mixing between the tanks. Wastewater mixing in a dual system occurs by both inter-compartmental mixing and turbulence. Oscillation of the wastewater between the tanks can be reduced or avoided by having the second tank smaller than the first, reducing the flow-through area, and connecting the tanks with an ell (Canter and Knox, 1985).

3.2.3 Inlet and outlet tees, air vents, and access

The design of the inlet and outlet pipes of the tank influence the movement of wastewater through the tank. Submerged inlet tees direct the wastewater flow downwards rather than straight to the outlet end of the tank, and help prevent accumulated scum from blocking the inlet pipe.

Maintaining a minimum 'sludge clear depth', defined the distance from the bottom of the inlet tee to the top of the sludge layer, contributes to efficient solids retention in the tank (Laak, 1980). Conclusions following the comparison of 300 domestic tanks were that this depth, with dimensions in feet, should be equal to 2.7 - 0.08A, where A = the surface area of the sludge (Abwasser-Normen, in Laak, 1980). In typical New Zealand septic tanks (Figure 6) this depth would be 0.849 feet (260 mm) for the 2,700 litre tank and 0.672 feet (205 mm) for the 3,300 litres tank.

Outlet tees on a single tank reduce the potential for scum and floatables to pass out with the effluent to the soakage field. An open top end of the outlet tee allows gases in the tank to pass through to the soil absorption area and disperse from there to the atmosphere.

Flood (c1928) recommended reducing influent velocity to the tank by installing a bell mouth under the inlet and believed that this would assist the sedimentation process in the tank. No methods to reduce influent velocity appear to have been adopted in New Zealand tanks.

Air vents are included for two reasons. First, to prevent wastewater flows from draining all wastewater from the "u" bends in the house plumbing by vacuum pressure. Second, to allow the escape of gases generated during the digestion of the wastewater (Kaplan, 1987). Vents in New Zealand tanks are sometimes installed near the dwelling to vent the drainage line, with a separate vent to allow the escape of gases from the tank. An amendment to the Standards has allowed venting directly from the tank so that the air vent can serve as an inspection hole into the tank. These vents are sometimes attached directly onto the top of inlet tee, thereby preventing the escape of gases directly from the tank. This allows malodorous gases to be released slowly through spaces around the access hole and through the outlet tee to the soil above the soakage field.

Winneberger (1984) conducted studies on the atmosphere of tanks sealed at the air vent and at the access hole and found that it was little different to that of the outside atmosphere. He concluded that the septic tank inlet and outlet tees had provided the ventilation role. These results indicate that venting in the manner described above does not compromise removal of gases generated in the tank.

An access point to the tank is necessary so that accumulated sludge can be periodically removed. Risers are fitted to access holes on buried tanks so that there is still ready access at the ground surface. This also helps owners to locate buried tanks otherwise only indicated by the air vent, which is usually at the inlet. The whereabouts of the second tank in a dual-tank system is likely to be forgotten if the access hole for each tank is buried. De-sludging of both tanks may then become a problem. Fitting risers from each tank to the ground surface allows easy detection and access to the tank.

3.2.4 In-tank filters

Avoiding or reducing solids overflow can be achieved by the installation effluent filters. These can be installed inside the tank near the outlet pipe, or may comprise a separate component of the system, such as an aerobic recirculating sand filter. Re-circulating sand filters, which are installed outside the tank, are discussed in Section 4.2.4.

There are several kinds of in-tank filtration possibilities available. One is to include layers of rock in the tank, another is to install some kind of constructed mesh screen around the outlet pipe. The use of filtration devices is not novel; Flood (c1928) recommended the installation of screens near the effluent outlet early this century.

Anaerobic rock filters provide a solid medium for the attachment of microbial growths. Influent wastewater is usually directed upwards, but can be downwards, or horizontally through the filter. Upwards wastewater flow enables anaerobic conditions to be maintained on the submerged material.

The only New Zealand septic tank to incorporate an upflow anaerobic rock filter is the Ecotank^{TM} (Figure 5). This tank, which has a total capacity of 5,000 litres, includes three chambers, gas baffling of the inlets and outlets, and a long flow path established by the circular first and second chambers surrounding the central chamber, which contains the rock filter. The first chamber is twice the volume of the second chamber, which incorporates stiff plastic mesh. After initial settling in the first and second chambers, wastewater is directed upwards through the rock filter and discharged to the soil absorption area. An alternative smaller 2,000 litre tank is available to upgrade an existing septic tank.

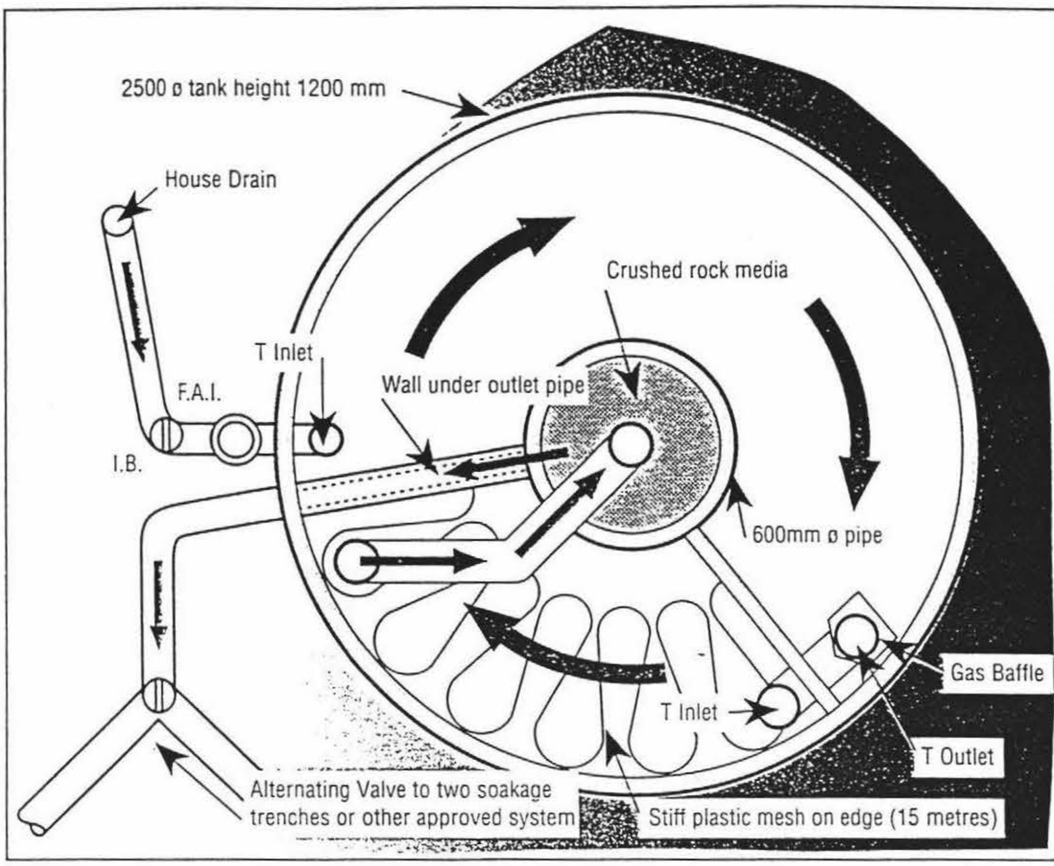


Figure 5. The Humes RD 5000 Ecotank™

The configuration of the first two chambers should ensure that most sludge accumulates in the first chamber, and gas baffling between all chambers reduces solids carry over. The stiff plastic mesh in the second chamber would help reduce turbulence and provide a surface for suspended growth treatment. This decreases suspended solids and BOD levels in the wastestream entering the third chamber, thereby reducing the potential for clogging of the rock filter.

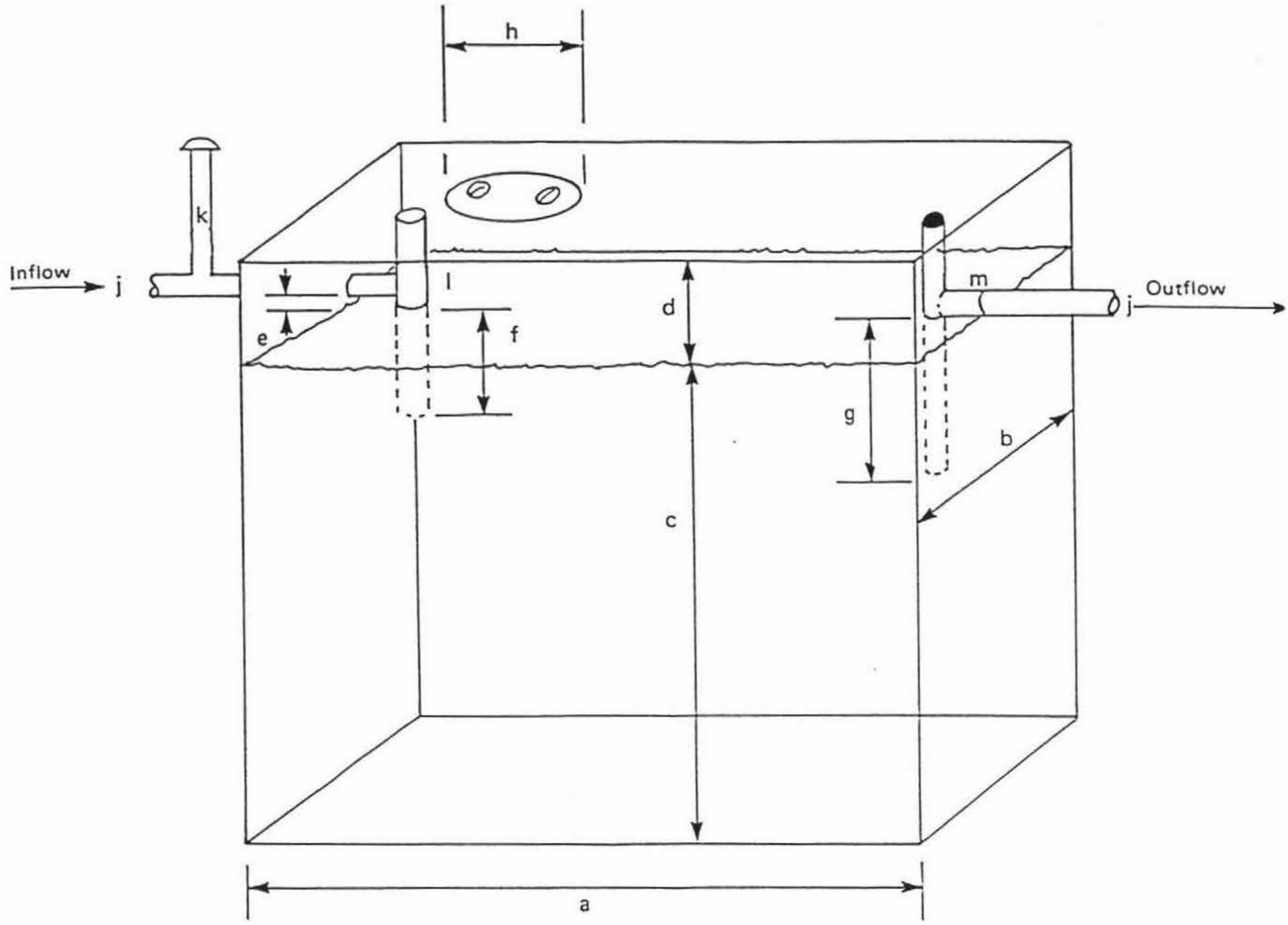
A criticism of the use rock filters in on-site systems is that owners have limited understanding of the need to backwash the rocks periodically to remedy any clogging that has taken place. Apparently some owners have removed the rocks in the central chamber once they clog for the first time (Miller, 1995, personal communication). Even without the rock filter, these systems would still retain considerable advantage over standard septic tanks because of the improved settling conditions and solids retention provided by the large volume, long flow path, compartmentalisation, plastic mesh, and inlet and outlet baffles.

Since 1994, the use of in-tank screen filters at the outlet pipe has been promoted in the Manawatu area where there are heavy clay soils which are particularly prone to clogging by septic tank effluent. Screen filters are a simple device to install in existing tanks, and can be easily removed for cleaning. The screen filter not only decreases the likelihood of wastewater short-circuiting in the tank but also provides a surface for anaerobic suspended growth treatment processes. When installed in a single tank there would be possibilities for the filter to become clogged with sludge and suspended solids. The frequency of screen cleaning requirements is not known because their utilisation is only recent.

3.2.5 Septic tank design in the New Zealand Standards

The New Zealand Standards for Household Septic Tank Systems (NZS 4610:1982), establish recommendations for household septic tanks and effluent disposal systems. The standards include requirements for septic tank construction, recommendations for septic tank design, recommendations for disposal field construction, and recommendations for system operation and maintenance. They are currently under review, first because their universal application even in limiting receiving environments has led to failed systems, and second so that they can be consistent with current New Zealand statutory requirements, such as the Resource Management Act and the Building Act, which have performance based requirements.

There are no requirements in the Standards for tank size, shape, influent baffles or number of tanks. Instead the standards contain recommendations about dimensions, air space etc. These are reproduced in Figure 6 below.



Dimension	Description	Tank capacity 2700 ℓ	Tank capacity 3300 ℓ
a	Liquid length	2150	2355
b	Liquid width	1000	1000
c	Liquid depth	1255	1400
d	Air space	300 (min.)	300 (min.)
e	Inlet invert above top water level	75	75
f	Inlet depth	150 (min.) 550 (max.)	150 (min.) 550 (max.)
g	Outlet depth	500	560
h	Access lids (Flush where tank top at ground level. For buried tanks extend lids to ground level.)	minimum dimension 600	
j	Inlet and outlet diameters	100	100
k	Vent on inlet drain installed before the entry to the tank		
l	Carry the inlet tee to within 25 mm of the underside of the tank top		
m	Plug the top of the outlet tee or carry through tank top and seal		

Figure 6. Septic tank design in the New Zealand Standards (NZS 4610:1982).

Tank construction details are given in Section 5.2 of the standards. The requirements are that the tank is constructed of durable materials, is watertight, capable of withstanding loads on its roof and walls, that inspection openings are brought up to finished ground-surface level, that air is vented, and that the inlet square junction is terminated 25 mm below the underside of the top of the tank. Section 6.2 of the standards requires that there is no stormwater infiltration to the tank or disposal field area. These requirements are discussed below.

Durable materials are essential to prolong the life of the tank. In the United States people have fallen into septic tanks because the roofs have collapsed (Winneberger, 1984). Hydrogen sulfide, which corrodes metal and concrete, is about one percent by volume of the septic tank atmosphere (Winneberger, 1984). Old concrete and brick tanks installed in New Zealand in the 1940s and 1950s, and corroded over time by moisture and hydrogen sulphide, are less durable than some modern materials and after some decades of operation may no longer have the strength to withstand the sidewall or roof pressure.

A watertight tank ensures that the wastewater volumes are not increased by groundwater infiltration into the tank and that wastewater does not leak from the tank directly out into the subsoil. Groundwater infiltration dilutes the wastewater and decreases its detention in the tank. This decreases the treatment achieved in the tank, and also causes the final discharge rate to the soakage field to be greater than the design discharge rate (Canter and Knox, 1985). Tanks are generally buried and the bottom of the tank may be up to two metres below the ground surface. Wastewater leaking from a tank can flow straight to shallow groundwater without the benefit of treatment provided in a soakage field.

The value of a riser and air venting is discussed in Section 3.2.3 above. Without a riser to the ground surface, soil covering the tank not only hides the whereabouts of the access hole, but also makes the de-sludging process less convenient. Air venting of the drainage line at the inlet pipe avoids malodorous gases from the tank being transported back to the house via the plumbing. The requirement to terminate the inlet square junction 25 mm below the top of the tank allows tank air to be vented via the open top end of the inlet tee to the drainage line. This may not be necessary because tank gases can escape through the outlet tee and percolate to the atmosphere through the soil above the soakage field. An advantage of venting the inlet tee in this way is that it can make the inlet pipe less prone to clogging (Laak, 1980).

Preventing stormwater infiltration avoids dilution of the wastewater in the tank. Dilution reduces the retention time of the wastewater, thereby reducing sedimentation opportunity and wastewater treatment in the tank, and can hydraulically overload the soakage field.

The dimensions in the recommended design were based on the kind of tanks typically being constructed in New Zealand in the early 1980s. The tank sizes provided for are 2700 and 3300 litres. This is less than that recommended by early design engineers and by modern specialists such as Laak (1986) and Winneberger (1984).

The standards recommend that the inlet tee depth is between 150 and 550 mm from the top of the tank. This allows the inlet pipe to discharge into the 'clear zone' between the scum and sludge layers. The outlet tee depth is recommended to be 500 mm deep in the 2700 litre tank and 560 mm deep in the 3300 litre tank so that effluent is discharged from the 'clear zone'. These depths reduce the opportunity for scum and floatables to be washed out with the effluent, and provide some clearance between the sludge layer and the outlet pipe. In a 2700 litre tank, which has a height of 1555 mm and an air space of 300 mm, the bottom of the tank would be 755 mm below the bottom of the outlet tee. The average home requires de-sludging about once every three years (NZS 4610:1982). Given a tank surface area of 2.15 square metres, and assuming a sludge accumulation rate of 80 litres per capita per year (Section 3.1.7) the sludge depth after three years in a tank serving a four-person household would be 447 mm. This would leave 308 mm clear zone between the bottom of the outlet tee and the top of the sludge. After four years the clear zone would be only 160 mm.

The length to width ratio in the Standards is about two to one. This is less than that recommended by regulatory authorities overseas. Shorter single tanks increase opportunities for wastewater short circuiting and allow sludge disturbed by influent flows to pass out with the effluent.

3.2.6 Tank design shortfalls

The key shortfall in the standard septic tank design is the lack of parts, such as baffles and multiple compartments, to improve hydraulic retention times and increase solids retention and sedimentation opportunities. Combined with a small tank size of 2,700 litres, which a four-person household could nearly than half fill with sludge after four years, there is a high risk that sludge disturbed by influent flows, and sludge re-suspended during anaerobic digestion, will be carried over to the soil absorption area. This unnecessarily transfers solids and pathogenic organisms from the tank to the environment. The tank design also makes it difficult to determine the sludge depth and assess the necessity for tank de-sludging. Design alterations to remedy tank design shortfalls are discussed in Chapter 5.

Chapter Four
Soil absorption areas

4.1 *Function of the soil absorption area*

Soil absorption areas (SAAs) take advantage of the soil's natural ability to provide treatment of both organic and inorganic pollutants. The organisms involved in decreasing the amounts of organic matter in the effluent include fungi, bacteria, algae, soil animals, protozoa and higher plants. These organisms are present in highest numbers in the top 150 mm of soils and their numbers decrease rapidly with depth (Cheremisinoff *et al.*, 1984).

A good soil system for receiving septic tank system effluent should absorb all effluent generated, provide a high level of treatment before the effluent reaches the groundwater, and have a long and useful life. Ideally, a soil should be able to absorb and treat a pollutant at a rate equal to, or greater than, the rate at which it is added to the soil.

The SAA receives effluent from the septic tank and allows it to soak into the ground via sub-surface drains. The key functions of the SAA are —

- to distribute effluent uniformly into the receiving environment;
- to achieve final treatment of the effluent; and
- to provide a place for the safe disposal of treated effluent.

These functions are strongly inter-related. The effectiveness of the SAA in providing adequate final treatment is dependant on the rate of application of the effluent to the soakage field, the formation of the biological mat in the soil under the leaching drains, and the hydraulic characteristics of the receiving environment (Canter and Knox, 1985). The physical and bio-chemical processes occurring in the SAA are described below.

4.1.1 Hydraulic distribution

Septic tank effluent is distributed throughout the soil absorption area (SAA) via perforated sub-surface pipes. The hydraulic effluent loading to the SAA should allow effective infiltration through the subsoil. Septic tank effluent infiltration rates depend on soil properties and effluent quality (Canter and Knox, 1985; Siegrist, 1987), establishment and permeability of a biological mat (Wilhelm *et al.*, 1994; Laak; 1986), dosing and resting (Siegrist, 1987; Wilhelm *et al.*, 1994), underlying restrictive layers (Reneau *et al.*, 1975), shape and area of the infiltrative surface (Laak, 1988, Clothier *et al.*, 1991), and climate (Jenssen and Siegrist, 1991).

Two forces move liquid through the surrounding soil. These are the downward force of gravity and the capillary attraction of water towards drier soil. The rate at which liquid will drain through soil by these forces is influenced by three factors. These are the dryness of the surrounding soil, which influences the capillary force exerted on the effluent flow, the geometry of the SAA, which affects its surface area, and the saturated hydraulic conductivity, K_{sat}, of the underlying soils (Clothier *et al.*, 1991).

Hydraulic conductivity (K) is the factor of proportionality in Darcy's Law, which states that the velocity of fluid through a porous medium is proportional to the hydraulic gradient (Canter *et al.*, 1987). The K-value of a saturated soil represents its average hydraulic conductivity, which depends mainly on the size, shape and distribution of the pores. It also depends on the soil temperature, and the viscosity and density of the liquid (Ritzema, 1994). Some values for the hydraulic conductivity of water in various soils are given in Table 5 below.

Most SAA systems are designed according to the hydraulic conductivity of the soil (Gunn, 1989; and Clothier *et al.*, 1991), although studies show that there is little correlation between the soil's percolation rate for clean water and that for wastewater. In fact a comparison of non-failing systems and failed systems showed that "the maximum safe Loading Infiltration Rate is fairly independent of soil type, providing the natural soil has an adequate basic permeability" (Troyan *et al.*, 1984, page 75). The reasonably consistent infiltration rate of 15 - 25 mm/day for sands, and about 10 mm/day for clayey soils, arises because of the establishment of a clogging layer, or biological mat at the bottom of the SAA.

Table 5. Hydraulic conductivities of selected materials (Canter *et al.*, 1987).

Granular material	**hydraulic conductivity (m/day)**	**Consolidated material**	**hydraulic conductivity (m/day)**
clay soils (surface)	0.01 - 0.2	sandstone	0.001 - 1.0
deep clay beds	10^{-8} - 10^{-2}	carbonate rock with secondary porosity	0.01 - 1.0
loam soils (surface)	0.1 - 1.0	shale	10^{-7}
fine sand	1.0 - 5.0	dense solid rock	10^{-5}
medium sand	5.0 - 20.0	fractured or weathered rock (aquifers)	0.001 - 10.0
coarse sand	20 - 100	fractured or weathered rock (core samples)	almost 0 - 300
gravel	100 - 1000	volcanic rock	almost 0 - 1000
sand and gravel mixes	5.0 - 100		
clay, sand and gravel mixes (till)	0.001 - 0.1		

The biological mat is made up of effluent solids, mineral precipitates, micro-organisms, and the by-products of decomposition. When viewed under an electron microscope, particles in a biological mat were found to be mostly shellbuilding amoebas and another unidentified organism, rather than paper or fibre materials as had been thought (Jenssen and Krogstad, 1988). The mat typically has an hydraulic conductivity of about 15 cm/day (Laak, 1986), which is similar to the hydraulic conductivity of surface clay soils (see Table 5).

Poor effluent quality, characterised by high concentrations of organic matter and suspended solids, and high hydraulic loading rates accelerate the development of a biological mat, or clogging layer (Simons and Magdoff, 1979; Siegrist, 1987; Jenssen and Siegrist, 1991; Brissaud, 1993; Wilhelm *et al.*, 1994). These same factors, together with underlying soil type and environmental conditions, affect the permeability of the mat (Laak, 1986). The production of polysaccharides by bacteria in the SAA may bind other cellular materials, producing a more flow resistant layer (Reneau *et al.*, 1989).

Soil clogging is significantly correlated with the cumulative mass density loadings of total BOD (ultimate carbonaceous and nitrogenous BOD). The clogging mat develops at the head or the low point of the SAA and progresses along the SAA with time. Reducing the applied mass loading rates, either by reducing the hydraulic loading rate or improving the effluent quality, reduces opportunity for soil clogging (Siegrist, 1987).

Laboratory experiments with sand columns evaluated at different hydraulic loading rates, effluent quality and temperature showed that the maximum safe septic tank effluent loading rate into sand fill was 20 mm/day (Simons and Magdoff, 1979). Clogging of the sand pores was accelerated with increased hydraulic loadings, low temperatures (less than 5°C), and low oxygen concentrations caused by saturated conditions in the column. Recovery time of the infiltrative surface after resting ranged from 10 to 127 days.

Topsoil usually has the highest hydraulic conductivity of all soil horizons (Bouma *et al.*, 1975), but the depth to soils with lower hydraulic permeabilities limits the infiltration of effluent away from the SAA and hence limits the disposal of the effluent. Clay soils have very low hydraulic conductivity and are also subject to swelling and shrinking upon wetting and drying, so that their hydraulic conductivity is variable with the season (Ritzema, 1994). Clay shrinking increases the number of macropores, which can provide a route for rapid drainage from the SAA without the benefit of treatment in and under the SAA.

The method and rate of effluent distribution in the SAA affects the subsequent movement of effluent through and away from the SAA. Effluent from the septic tank is distributed either by flowing freely through perforated pipes as it is displaced from the tank by incoming flows (conventional systems), or is pumped through pressure lines from a pump chamber (Gunn, 1994).

Free-flowing effluent distribution results in extremely ineffective use of the infiltrative surfaces in the SAA. Almost all liquid discharges either through the first hole, or through the lowest hole (Converse, 1974; Machmeier and Anderson, 1987) and the function of effluent distribution is not effectively achieved. In highly permeable soils, a high hydraulic loading at a particular discharge point increases the potential for groundwater contamination because saturated flow conditions in the subsoil allow rapid travel of an effluent plume that has undergone little treatment in the SAA (Reneau *et al.*, 1989). This defeats the second function (effluent treatment) of the SAA. In less permeable soils the local high hydraulic loading causes a localised build-up of the biological mat, and materials near the infiltrative surface fill and block soil pores. This frustrates the third function of the SAA (effluent disposal) because the effluent collects and ponds on the ground surface.

Low pressure distribution systems can achieve uniform effluent distribution throughout the SAA, providing better treatment of the effluent and less clogging in certain soils, particularly sands (Converse *et al.*, 1974; Ijzerman *et al.*, 1993), and clays (Simon and Reneau, 1984). Low pressure distribution systems are described in Section 4.2.2.

As well as achieving effective hydraulic operation in the SAA and allowing a full and economic service life, the function of uniform effluent distribution is essential to the SAA's effective performance as a biological treatment system.

4.1.2 Treatment in the soil absorption area

The quality of septic tank effluent entering the SAA depends on the degree of treatment achieved in the septic tank. Once in the SAA the sewage undergoes further treatment before soaking through the subsoil to groundwater. Biochemical treatment in the SAA is partly anaerobic and partly aerobic. The anaerobic treatment occurs in the biological mat at the bottom of the SAA. The aerobic treatment occurs in the unsaturated zone below the SAA.

The availability of oxygen is a key factor influencing the functioning of the SAA. With recommended hydraulic loading rates of 10 to 50 mm/day (Fig. 8.1, Gunn, 1989; Canter and Knox, 1985), and effluent carbonaceous and nitrogenous oxygen demands in the SAA of between 400 and 1500 mg/l O_2 (Wilhelm *et al.*, 1994), oxygen requirements in the SAA are in the range of 4 - 75 g O_2 /m^2/day.

In an aerobic environment, organic carbon and ammonia are oxidised to nitrate and carbon dioxide. Reactions in an aerobic SAA can be represented by the following chemical equations (Wilhelm *et al.*, 1994):

Organic matter oxidation:

$$CH_2O + O_2 \rightarrow CO_2 + H_2O$$

Nitrification:

$$NH_4^+ + 2\ O_2 \rightarrow NO_3^- + 2\ H^+ + H_2O$$

Sulphide oxidation:

$$H_2S \text{ (or organic sulphide)} + 2\ O_2 \rightarrow SO_4^{2-} + 2\ H^+$$

Oxygen diffusion in water is much slower than in air and so maintaining unsaturated conditions in the SAA, particularly in the sidewalls, increases oxygen availability and provides a more aerobic environment for the degradation of septic tank effluent.

Saturated conditions in and below the SAA, caused by high water tables or by high hydraulic loads, limit the potential for aerobic activity in the SAA. Septic tank effluent treatment in unsaturated conditions compared to saturated conditions has been assessed in terms of COD removal in various studies. Septic tank effluent loaded to pilot scale reactors at 30 mm/day reduced COD to a mean of 20.4 mg/l in saturated sands, and 18.1 mg/l in unsaturated sands from 200 - 250 mg/l (Winkler and Veneman, 1991). A reduction in COD from 200 - 300 mg/l to 10 - 40 mg/l (87 - 95%) was achieved in ninety centimetres of unsaturated aerobic silt loam and sand or sandy loam fill in a Wisconsin mound (Magdoff *et al.*, 1974). In saturated conditions COD was only reduced to 50 - 100 mg/l (66 - 75%). High COD reduction in unsaturated conditions was attributed to the ability of aerobic microorganisms to easily degrade the products of anaerobic decomposition from the septic tank. This reinforces the need for increasing opportunities for wastewater decomposition in the tank. In another study, a predominantly sand Wisconsin mound achieved 70 percent COD removal and 99 percent faecal coliform removal from septic tank effluent in aerobic conditions maintained beneath the biological mat (Willman *et al.*, 1981). Unsaturated conditions were achieved in that mound by limiting the hydraulic loading to 15 mm/day.

The use of peat filters in SAA has been trialed to determine their effectiveness in reducing BOD, suspended solids and bacterial contamination in septic tank effluent (Brooks *et al.*, 1984). Septic tank effluent was gravity fed through 750 mm of sphagnum peat moss at 15 mm/day. Mean effluent concentrations after filtration were 15 mg/l BOD_5, 82 mg/l COD, and 16 mg/l total suspended solids. These represent more than 90 percent reduction in BOD_5, 80 percent reduction in COD, and 90 percent reduction in total suspended solids. Faecal coliform concentrations were reduced by more than 99.999 percent from 7.5×10^4 to less than one colonies per litre.

Peat filters of 200 and 500 mm thickness achieved 88 to 91 percent BOD removal, with final effluent concentrations less than 30 mg/l (Viraraghavan and Rana, 1991). In general, removal efficiency was lowest with the thinnest filter layer, and was also found to decrease when the hydraulic load was increased from 63.7 mm/day to 140.0 mm/day. The thickest filter layer did not always achieve the best removal efficiency. The peat also showed good suspended solids filtration capacity, which was generally best for the thickest peat layer.

High oxygen demands and moist conditions in the SAA produce locally anaerobic conditions in the biological mat, which functions as an efficient biofilter because of the high concentrations of microbial organisms (Laak, 1986). The same anaerobic reactions take place in the mat as take place in the septic tank (Wilhelm *et al.*, 1994).

Unsaturated conditions in and below the SAA enhance treatment potential of septic tank effluent in terms of BOD and COD reductions. Designs to promote these conditions are described in Section 4.2 below.

4.1.3 Nitrogen quantities and movement

Organic nitrogen entering the ST-SAS is mostly reduced to ammonia in the anaerobic conditions of the septic tank (Section 3.2.3). American sources estimate that nitrogen concentration in septic tank effluent is typically 40 mg/l with about 75 percent as NH_4^+ and 25 percent in an organic form (Canter and Knox, 1985; Laak, 1986). These concentrations, applied to the SAA at hydraulic loadings between 10 and 50 mm/day, represent a nitrogen loading to the environment of between 146 and 730 g N/m^2/year. As a comparison, the recommended nitrogen loading for agricultural waste disposal onto land is 200 kg N/ha/year (Manawatu-Wanganui Regional Council, 1995), which equates to 20 g N/ m^2/year. In areas where groundwater may be at risk nitrogen loading is restricted to 150 kg N/ha/year, or 15 g N/ m^2/year.

The fate of nitrogen introduced to soil depends on its form as well as biological conversions capable of taking place in the soil. In soils under the SAA the form of nitrogen is influenced by soil type, the aerobic/anaerobic condition of the soil, pH and cation exchange capacity. Nitrification, the process of changing ammonia (NH_4^+) to nitrite (NO_2^-), then nitrate (NO_3^-) takes place in aerobic conditions. Nitrification is performed primarily by obligate autotrophic bacteria and nitrate is the predominant end product (Canter and Knox, 1985). In most agricultural soils in New Zealand, NH_4^+ is rapidly oxidised to NO_3^- by nitrifying micro-organisms (Speir and Kettles, 1993). Denitrification, the process of completely removing nitrogen by reducing it from nitrate to atmospheric nitrogen (N_2), occurs in anaerobic conditions, and is performed by facultative heterotrophic bacteria. The Nitrogen cycle is shown in Figure 7.

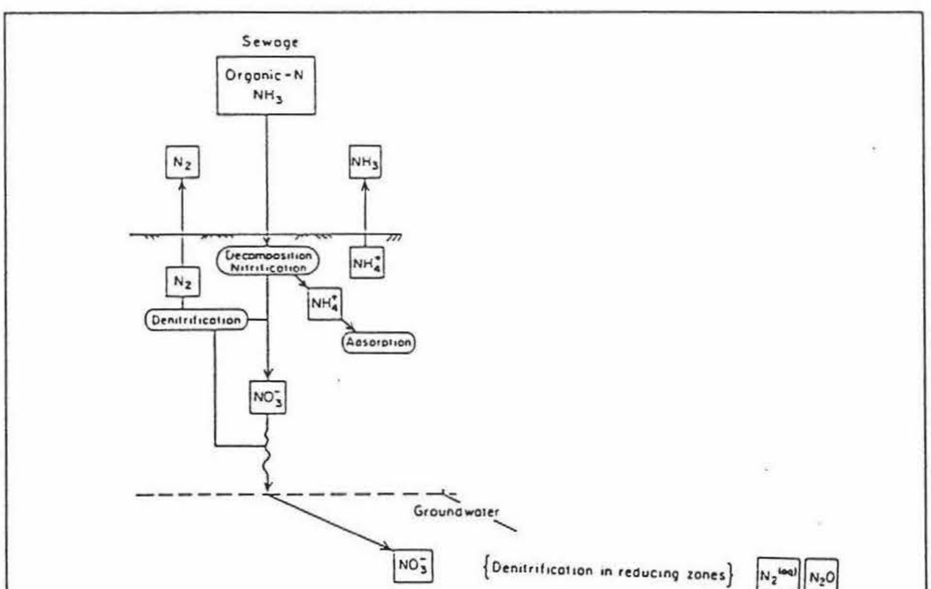


Figure 7. Form and fate of nitrogen in the subsurface environment (Canter and Knox, 1985).

Nitrate-nitrogen and ammonium-nitrogen are both plant available, but behave very differently in soil. While ammonium will bind to soil particles in anaerobic conditions, nitrates (being negatively charged) will not, and are therefore more mobile. This mobility allows nitrate to be readily leached down the soil profile and out of the plant rooting zone. Nitrates migrate long distances from input areas if there are highly permeable subsurface materials which contain dissolved oxygen and are easily transported in groundwater because of their solubility and anionic form. For these reasons nitrates are more mobile than ammonium in both saturated and unsaturated soils (Canter and Knox, 1985).

Elevated nitrate concentrations in surface water can cause prolific and often objectionable growths of aquatic vegetation, and can present a health risk to people drinking the water. Prolific algal growths in surface water promote eutrophication, while high concentrations of nitrates in drinking water can be toxic to people and animals. The New Zealand drinking water standards require that nitrate-nitrogen concentrations in drinking water are not more than 11.3 mg/l (Ministry of Health, 1995).

Studies indicate that nitrate concentrations from between 0.04 and 0.10 g/m^3 and over, together with dissolved reactive phosphorus levels over about 15 mg/m^3, are sufficient to promote undesirable algal growth in surface freshwater (Ministry for the Environment, 1992). Phosphorus is generally the limiting nutrient in New Zealand freshwater environments for algal growths, although rivers draining the central North Island are an exception to this with nitrogen sometimes being the limiting nutrient. Nitrogen is normally the limiting nutrient in marine environments (Manawatu-Wanganui Regional Council, 1995). Lakes can be phosphorus or nitrogen limited (Ministry for the Environment, 1992) and may change according to the season.

The primary health hazard associated with drinking water containing high levels of nitrate is methemoglobinaemia. Young infants (less than six months) receiving milk formula diluted with water containing more than 10-20 mg/l of nitrate-nitrogen have developed methemoglobinaemia (the 'blue-baby' syndrome), which can be fatal (Pandey and Carnus, 1993). Development of methemoglobinaemia in infants ingesting high quantities of nitrates is linked to nutritional practices and possibly genetic factors (Shuval and Gruener, 1977). An additional potential health hazard is the presence of carcinogenic nitrosamines, which have been linked with stomach cancer in adults (Reneau *et al.*, 1989; Pandey and Carnus, 1993).

Up to 40-50 percent of nitrogen added to the ST-SAS may be removed by the septic tank, the biological mat, and the upper layer of soil to a depth of about 60 cm below the SAA (Laak, 1986). Most, about 34 percent, is retained in the sludge and scum layer in the tank (Section 3.1.7). The unsaturated soil conditions under the SAA are not suitable for denitrification, which requires the presence of metabolically available carbon and anaerobic conditions. Denitrification rates under conventional SAA can vary from 0 to 35 percent, and are particularly poor in systems installed over high water tables because of the lack of opportunity for nitrification of the effluent from the tank (Ritter and Eastburn, 1988). Lamb *et al.* (1990) observed a five percent mean nitrogen removal in conventional SAA during a two year study period.

Elevation of nitrate levels in surface water and ground water directly attributable to septic tank contamination has been demonstrated in numerous studies (Sikora and Keeney, 1976; Gibbs, 1977b; Brown *et al.*, 1984; Ritter and Eastburn, 1988; Converse *et al.*, 1991; Wilhelm *et al.*, 1994). A summary of three of those studies is as follows. Three holiday homes on a barrier spit in the United States caused nitrate levels to increase from less than 2 mg/l to averages of 20 and 50 mg/l during summer occupation (Wilhelm *et al.*, 1994). Nitrogen input to Lake Taupo from a septic tank soakhole on the lake shore could be up to 30 kg N per year (Gibbs, 1977b). Ammonium and nitrate leaching from septic tank effluent was higher in sandy loam soils compared to sandy clay, and clay, and was highest in that soil when receiving the greatest hydraulic load (Brown *et al.*, 1984).

The mobility of nitrate in the environment, and its limited removal in standard ST-SAS highlight the importance of improving its removal efficiency in order to allow continued use of ST-SAS in at-risk areas. Several innovations to ST-SAS have been developed to enhance nitrogen removal. These are described in Section 4.2.4. An alternative is to restrict ST-SAS use in at-risk areas.

4.1.4 Removal of pathogens

Pathogen numbers reaching the soakage field depends on the presence of an infected person in the household and will be no more those entering the tank because pathogens do not multiply in a septic tank; they can only survive or be reduced (Laak, 1986). Soils are a hostile environment for most disease organisms excreted by humans, and one in which soil organisms are better suited. The most efficient destruction of pathogens will occur in the topsoil and the biological mat because this is where microbial competition is most intense.

The biological mat is a very significant part of the treatment performed by the system, and has been estimated as accomplishing more than 95 percent of the 'bacterial cleansing' that occurs (Perkins, 1989). Physical straining, particularly in the biological mat and in clayey silty soils, also contributes to bacterial removal. Physical straining is less significant for removal of viruses, which are only 18 - 25 nm compared to bacteria which are about 750 nm (Reneau *et al.*, 1989). The chemical process of adsorption by negatively charged soil particles can occur when pore spaces in the soil are several times larger than the bacteria (Canter and Knox, 1985). Physical and chemical processes help keep bacteria in the upper soil profile, where they are subject to microbial competition, and decrease the potential for them to be transported to groundwater, where their survival rates are better.

Additional mechanisms such as competition for nutrients and the production of antibiotics by high populations of actinomycetes have been suggested to play an important role in the rapid die-off of faecal coliforms and streptococci. The production of actinomycetes occurs in the unsaturated aerobic zone beneath the biological mat (Bouma, 1979).

Various studies indicate that soil moisture is the principle factor determining the survival of bacteria once they leave the soakage field. *Salmonella* species may survive 11 to 280 days in soil and from seven to 53 days on crops, where drier conditions and sunlight contribute to die-off (Dept. of Health, 1992). Although soil temperature, pH, and the availability of organic matter also influence survival time, enteric bacteria have been shown to persist in soils that retain a high amount of moisture, such as loam and adobe peat, for more than 42 days (Canter and Knox, 1985).

The mobility of pathogenic organisms in soils has not been studied although inferences can be made from studies done on coliform bacteria and various viruses. These indicate that organisms travel furthest with rapid water flow, colder temperatures, low soil surface area, such as sand, and weak soil attraction forces, for example, in gravel and fissured rock (Laak, 1986). Some examples of coliform and virus studies are as follows.

In poorly draining fine loam soils septic tank effluent intercepted at tile drains 15.2 metres from the SAA had faecal coliform concentrations up to 10^3 /100 ml (Reneau, 1978). Bacterial and viral removal efficiency is better in unsaturated soil conditions with effluent travelling through the smaller pores because slow average pore water velocities increase contact opportunities with soil surfaces (Reneau *et al.*, 1989).

When antibiotic resistant E. coli were inoculated into three different soil horizons, their presence was detected at concentrations of 10^1/100 ml five metres downslope after 96 hours in soils with the lowest horizontal hydraulic conductivity (0.01 - 1.35 cm/hour depending on depth). Soils with higher horizontal hydraulic conductivity (26.21 - 28.13 cm/hour depending on depth) showed E. coli presence up to 10^3/100 ml 10 metres downslope after only 12 hours (Rahe *et al.*, 1978). Bacteria were reduced from 10^6 to 10^3 /100 ml in clay soils 6.1 metres from the SAA compared to similar reductions after more than twice the distance (13.5 metres) in loam soils (Reneau and Pettry, 1975).

A microbiological survey of the Templeton (NZ) sewage irrigation scheme found that sewage leached through the thin layer of subsoil, and through up to 20 metres of highly permeable shingle below, with only a slight reduction in indicator bacteria numbers. These studies showed that micro-organisms were able to move laterally at least 200 metres and that it took several weeks for a 1-log reduction in indicator numbers once irrigation was ceased (Martin and Noonan, 1977).

Viruses, which may survive treatment in septic tanks more easily than bacteria, are capable of producing an infection even with the smallest detectable numbers (Sproul, 1973; Mahdy, 1979; WHO, 1981). Enteric viruses have been linked to sixty-five percent of the disease outbreaks caused by consumption of contaminated groundwater in the United States (Yates, 1984). Interrupting the transmission routes of viruses entering septic tank soil absorption systems is vital because low concentrations of viruses in water used for drinking may produce carriers in the population leading to endemic viral disease transmission (AWWA, 1979).

The most important mechanism of virus removal in soil is by adsorption onto soil particles. For some viruses the most important factors affecting that absorption are pH and organic matter (Gerba, 1984). Soil pH has been shown to be a controlling variable in bacteriophage sorption and desorption, with increases in pH from 5.7 to 8.3 causing rapid phage release (Bales *et al.*, 1995). Virus adsorption is particularly rapid and effective at pH values below 7.4, and will therefore be effective in most New Zealand soils which are generally acidic. Dissolved organic matter tends to compete with virus absorption sites and so poor effluent quality, which is characterised by high BOD and suspended solids concentrations, will adversely affect virus removal in the SAA.

Adsorption of virus particles by soils also increases with increasing clay content, silt content, and ion-exchange-capacity (Canter and Knox, 1985). Viruses have poor adsorption to sands and can travel long distances in sandy soils. For example, virus presence was detected at distances of up to 250 metres from a wastewater rapid infiltration system in coarse gravel and sand (Vaughn *et al.*, 1983).

Virus absorption is poorest in saturated soil conditions (Yates and Ouyang, 1992; Gerba, 1984), which can be caused by high hydraulic loadings. For this reason, Bouma (1979) recommends an SAA hydraulic loading of less than 50 mm/day.

Establishing conditions that inhibit virus movement and enhance virus die-off would be a more effective way to interrupt transmission routes than determining 'safe' distances between SAA and drinking water wells. This is because setback distances depend on groundwater flow characteristics, which are very site specific, and rates of virus inactivation, which are virus specific. In one study setback distances necessary to ensure a 7 log reduction in virus concentrations ranged from 15 m to 300 m (Yates and Yates, 1988).

These studies indicate that establishing a transmission route for pathogens in soil is easiest in warm, sandy, saturated soils. Human infection is possible if water downstream of an SAA installed and operated in these conditions is used for drinking. A possible way to enhance pathogen removal in sandy soils is to ensure that a biological mat is established at the bottom of the soakage area. Designs to avoid the establishment of saturated conditions are discussed in Section 4.2.1.

4.1.5 Removal of other pollutants

Other pollutants in wastewater that may contaminate soils and water are phosphorus, xenobiotic ('man-made') organic compounds, such as detergents and solvents, and heavy metals. Concentrations of the latter pollutants in septic tank effluent are less than in collective systems, but are still present.

The average concentration of total phosphorus in wastewater entering a septic tank is 25 mg/l, with 8.8 mg/l in the inorganic orthophosphate form and the remainder in the organic form. Anaerobic conditions in the tank convert most of the remaining phosphorus to soluble orthophosphate (dissolved reactive phosphorus). In septic tank effluent, total phosphorus is about 15 mg/l of which about 75 percent is as orthophosphate (Canter and Knox, 1985). Dissolved reactive phosphorus levels from two New Zealand sources (Close, 1989; and in the Eketahuna Imhoff tank, MWRC, 1994) have mean concentrations much less than this, being 3.54 mg/l and 2.0 mg/l respectively (Table 3, Section 3.1.1). These low levels could be particular to those tanks, or could indicate lower levels generally in New Zealand, possibly because of lower levels in detergents and other cleaning agents.

Humans are estimated to excrete between three and six grams of phosphorus per day, and, in the United States, another three to four grams per person is contributed to domestic sewage daily in cleaning compounds (Laak, 1986). These ratios are consistent with New Zealand research which found that more than half of the phosphorus in New Zealand domestic sewage is derived from detergents (White, 1990). There are phosphate free detergents available in New Zealand, however, the choice to use them is still market driven. A ban on phosphates in detergents imposed in Switzerland in 1986 resulted in an overall fall of 4500 tonnes in phosphate reaching the sewage system between 1983 and 1987 (Candinas, *et al.*, 1991).

Soils beneath soakage fields have been found to remove wastewater soluble orthophosphate by allowing the formation of relatively insoluble phosphate compounds of iron, aluminium and calcium. Phosphorus adsorbed in these compounds is generally held very tightly in the soils and is resistant to leaching. Phosphorus leaches through coarse textured soils that are low in hydrous oxides, or in situations where there is poor effluent distribution and rapid flow away from the SAA (Reneau *et al.*, 1989). Dissolved reactive phosphorus is the limiting nutrient for algal growth in many of New Zealand's surface waters (Ministry for the Environment, 1992), but phosphorus is not mobile in most New Zealand soils.

Israeli studies showed that some biodegradable substances, such as detergents, do not break down once they are in the deeper soil layers. The failure to degrade in deep soils is attributed to low microbial populations, and lack of oxygen and supporting organic material. Municipal wastewater exerted a strong mobilising effect on organic contaminants such as toluene and phthalates, while xenobiotic organics decreased significantly with depth under land irrigated with fresh water. The contaminants persisted for more than fifteen years after wastewater application (Muszkat *et al.*, 1993).

Levels of heavy metals in New Zealand domestic sewage are not cited in the literature but heavy metals have been found in septic tank effluent and some, such as copper, may originate from plumbing facilities. Soil type or composition is a very important factor in all heavy metal fixation reactions, with clays and soils containing organic matter capable of absorbing metals more easily than coarser soils because of their high cation exchange capacity (Canter and Knox, 1985). In low pH environments and in anaerobic conditions heavy metals can be very mobile.

Saturated anaerobic conditions and sandy soils can allow the transport of phosphorus and heavy metals away from the SAA. Environmental levels of these pollutants could be decreased by limiting their input to septic tanks and by avoiding saturated conditions under the SAA.

4.2 Design of the soil absorption area

Functional failure of the soil absorption area will cause the release of significant quantities of pollutants to the wider environment. In the United States 66.3 percent of 457 malfunctions in 5,223 ST-SAS were caused by SAA failure (Gross and Thrasher, 1984). Uneven effluent distribution was implicated in almost half of these failures.

The key design elements of the SAA are effluent hydraulic loading, SAA shape, in particular width and depth, and method of distribution. These in turn are dictated by the daily volumes of wastewater requiring disposal, and the subsurface geology and hydrology of the receiving environment. Some attempts to classify receiving environments have been made by rating areas according to limiting factors such as shallow depth to bedrock or frangipan, seasonally high water tables, very high or low soil permeability, stoniness, and steep slopes (Steele *et al.*, 1986; Kumar, 1987). In general in New Zealand, SAA are designed according to the recommendations in the New Zealand standards and some established local practices.

The physical engineering design of the soil absorption area is described below.

4.2.1 Hydraulic loading

The wastewater volume generated in a four-person household is estimated as 720 litres per day (Gunn, 1989). Regardless of the degree of treatment achieved in the ST-SAS, the soil absorption area must allow the volume of liquid generated by the household to be disposed of into the soil without ponding or surface runoff.

As discussed in Section 4.1.1, the long term acceptance rate (LTAR) of the SAA is defined by the hydraulic conductivity of the biological mat, which is typically 15 cm/day, provided underlying soil types have adequate basic permeability. The long term infiltration rate of sewage into permeable soils, except sandy gravels and gravels, eventually declines to about the same quantity, regardless of the difference in soil permeabilities in the beginning (Canter and Knox, 1985; Laak, 1986). The period for the biological mat to reach its LTAR depends on the hydraulic load and the underlying soil type, and could be from one month to a year (Canter and Knox, 1985). Exceedance of the LTAR causes excessive build-up of the biological mat, and the thicker it becomes, the less easily effluent can infiltrate the bottom of the SAA.

The LTAR of soil absorption areas varies from about 32 mm/day for very permeable soils to about 8 mm/day for less permeable soils (Laak, 1986). Applying higher hydraulic loadings than the LTAR increases the thickness of the mat because its development is faster in saturated conditions.

The infiltrative area available is the surface area of the bottom and sidewalls of the SAA. The total sidewall area is only available as part of the infiltrative surface if the SAA is kept full of effluent or if the bottom the trench is totally blocked. Such saturated conditions are undesirable in an SAA (Section 4.1.2) and so it is preferable that the SAA is not designed to operate in this way. While some effluent will percolate through the lower portion of the sidewalls, the added area of infiltrative surface should be regarded as a safety factor rather than part of the design.

Jenssen and Siegrist (1991) developed a framework (the Mean Grain Size and Sorting Diagram) to determine wastewater hydraulic loading rates according to effluent quality and receiving soil types. The application of this process is still in development, in particular with regard to long-term performance. Their recommendations for domestic septic tank effluent are 10 mm/day for clayey, loamy and fine sandy soils; 25 mm/day for sorted soils; and 50 mm/day for coarse sands and gravels. These rates correspond to design loadings in other texts (Gunn, 1989), but, in the case of sands and gravels, are higher than would be required so as not to exceed the LTAR of the biological mat.

Some Tokomaru silt loams in the Manawatu area have saturated hydraulic conductivity of only 0.5 mm/day at 600 mm depth, and this is about half the expected evapo-transpiration rate in this area during winter (Clothier *et al.*, 1991). These soils would require large SAA with at least 480 m^2 infiltrative surface to dispose of 720 litres of wastewater. Soils with very high permeability would still require an SAA with a surface area of 22.5 m^2 for the same wastewater volume because of the hydraulic limitations of the biological mat. In both soils, effluent would have to be evenly distributed to avoid localised exceedance of the LTAR. Uniform effluent application is achieved with low pressure distribution.

Ijzerman *et al.* (1993) evaluated three low pressure distribution systems at three different loadings rates over a period of one year. The sites had hydraulic restrictive horizons of silty clay and bedrock at shallow depths. The system with the lowest loading (4.1 mm/day) had successful hydraulic performance and produced the driest conditions under the SAA. The system with the highest loading (16.7 mm/day) failed even in summer conditions because the loading rate was too high to allow infiltration into the soil. Tests for the presence of microbial tracers showed that the 4.1 mm/day loading rate achieved more than 99.9 percent removal of bacterial and coliphage tracers, and that the system loaded at 7.7 mm/day achieved more than 99 percent removal. They reported that their results supported a similar study they had undertaken in different soil and moisture conditions where tracer removal efficiency also exceeded 99 percent.

Saturated conditions in the SAA can be caused by high hydraulic loading or by high water tables. The phenomenon of groundwater mounding under the SAA causes a shallow depth to groundwater to become even shallower. This reduces, or even eliminates the depth of the unsaturated zone where aerobic treatment takes place. Groundwater mounding is greatest over shallow aquifers with low permeability. Subdividing a single disposal field into widely separated smaller fields reduces mound heights more effectively than having a single long SAA, increasing SAA size (thereby decreasing the hydraulic loading), or intermittent dosing (Finnemore, 1993).

The hydraulic loading should be designed to avoid saturated conditions in and below the SAA. This can be achieved by designing sufficient infiltrative surface area according to the LTAR, promoting the development of a biological mat in porous soils, and ensuring that all surface area is utilised. Saving money by reducing the size of the SAA is not a cost-efficient exercise. Kaplan (1987) demonstrated that, after taking account of soil clogging, the greater the infiltrative surface area available, the longer the service life of the SAA. This relationship far exceeded direct proportion, for example, doubling the leachline length from 25.5 feet to 50 feet increased the time before soil clogging from six years to ten years. Doubling the length again yielded an SAA that theoretically would never fail from soil clogging.

4.2.2 Gravity and low pressure distribution

Conventional systems allow free-flow of effluent, which has been displaced from the septic tank by incoming flows, into the distribution pipes. Flow to the septic tank is intermittent throughout the day, and an overall wastewater production of 720 litres per day will be discharged to the SAA in periodic pulses damped by the septic tank. One study showed that a toilet flush of 23.1 litre volume caused a peak discharge to the distribution pipe of 0.049 l/s and took about 20 minutes to discharge; the washing machine had a peak discharge of 0.124 l/s and took over 30 minutes to complete discharging (Macmeier and Anderson 1987).

Converse (1974) compared septic tank effluent distribution in an SAA through conventional free-flowing perforated pipes with distribution through pressurised pipes. He found that in conventional free-flowing systems most liquid discharges through the lowest holes, regardless of their placement on the pipe, and that a uniform distribution of the effluent was not achievable.

Machmeier and Anderson (1987) investigated free-flow effluent distribution in a perforated pipe by observing the points of discharge at various pipe slopes and with different perforation locations. They found that with perforations at the bottom of the pipe, and flow rates of 0.063 l/s (similar to a toilet flush) only two of the 142 perforations discharged any water, with almost all water discharged from the first perforation. The total hydraulic load was therefore spread over only 25 cm of the total 18 m length. During the period of discharge, this is equivalent to an hydraulic load of 125 mm/day if discharged into a trench 450 mm wide. With flow rates of 0.50 l/s (about four times a washing machine peak discharge) the first four perforations discharged most of the water. Even when pipes were installed on slopes of 10.2 cm per 30 m, over 70 percent of the water discharged from the first perforation.

Changing the location of the perforations to the sides of the pipe did not improve effluent distribution. In this situation, and with the pipe absolutely level, effluent accumulated and discharged at some random point. They found that without the pipe being absolutely level, effluent discharged from the lowest point, and this only needed to be 12.7 mm below any other point.

Uniform distribution can be achieved throughout the field by low pressure dosing. Low pressure distribution systems ensure that effluent is applied at the design hydraulic loading throughout the SAA consistent with the LTAR for the site. Low pressure distribution systems were developed for sites with

- seasonally shallow or perched groundwater
- soils with hydraulic restrictive horizons, such as clays or bedrock, at shallow depths
- sandy soils with high permeability, and
- sites with steep slopes (Ijzerman *et al.*, 1993).

Achieving pressured distribution requires installation of a pump, unless the site can take advantage of suitable on-site slopes. Appropriate pump sizing, providing at least 75 litres per minute at seven metres head, was essential in one set of trials to prevent clogging of the holes and pressured laterals were recommended not to be longer than six metres (Converse *et al.*, 1974). Further, hole diameters must increase along the length of the pipe to account for pressure drop, with the size determined by trial and error (Converse, 1974).

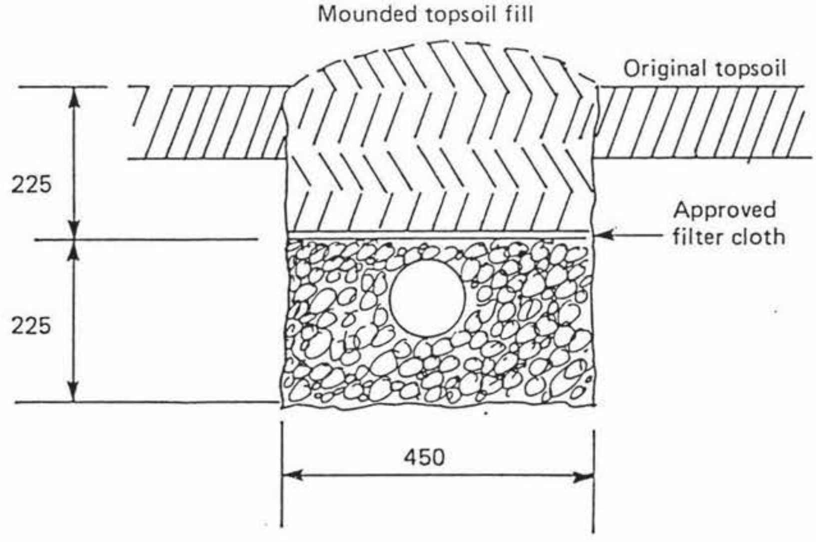
Low pressure dosing not only allows effluent to be uniformly distributed at the design rate, but, on flat sites, allows the effluent to be pumped from the septic tank outlet and discharged into the topsoil, which is the most suitable soil horizon for treatment (Reneau *et al.*, 1989). Better treatment, pathogen removal and die-off is achieved in the unsaturated zone produced under the SAA with low pressure distribution. Conditions are also less suitable for the transport of phosphorus and heavy metals.

4.2.3 Shape and depth of the SAA

Shapes and depths of soil absorption areas vary according to receiving environments. The most common designs are trenches and beds, but alternatives such as mounds have advantages in some situations. These designs are described below.

Trench designs rely on the absorption capacity of the subsoil for the treatment and disposal of the effluent, and some treatment takes place in the biological mat. Trenches are used in most soil types, from rapid draining coarse sand to slowly draining clayey loams and silts.

The New Zealand Standards describe the typical trench design installed as soil absorption areas (Section 4.2.5). A typical design is shown in Figure 8 below. These are recommended to be 450 mm wide, 450 mm deep, with the effluent distribution line (100 mm diameter field tile pipes) laid in the lower half of the trench. The distribution line is placed in a filter media, typically washed gravel. The purpose of the gravel media is to support the sidewalls and prevent their collapse into the absorption area, to dissipate energy from incoming effluent flow, to support the distribution pipe, and to provide a storage place for the effluent before it soaks away (Amerson *et al.*, 1991). In one ST-SAS in the Manawatu, a low pressure distribution pipe was placed inside nova flow drainage pipes, instead of gravel, without a trench. The effectiveness of this alternative in performing the functions of the gravel has not been reported.



Note: all measurements in mm. 1. Distribution drains to be 100 mm diameter and may be field tile pipes of 300 mm length laid at spacing 6 to 12 mm, or alternatively a perforated pipeline of approved material with perforations comprising at least 2% of surface area. 2. Distribution pipes to be laid flat or at gradient not greater than 1 in 200. 3. Sides and base of trench to be carefully scratched with a pointed tool before laying filter media.

Figure 8. Soakage trench design (NZS 4610:1982).

An important benefit of the trench design is that the narrow width allows more oxygen diffusion through the sidewalls than wider bed designs of the same area, providing a more aerobic environment. It is important not to lay the distribution pipe too near the bottom of the trench. First, because this would distribute the effluent into the lower soil horizon too far below the plant rooting zone (grass, for example, has a rooting zone of only about 200 mm), and second, there is less microbial activity in the lower soil horizons.

Evapo-transpiration seepage beds (ETS beds) are installed in moderate to poorly draining soils. The standard width is 1500 to 1800 mm, with a depth of 450 mm (Gunn, 1989). The lower half of the bed, where the distribution pipe is placed, is filled with 'no fines' gravel, while the upper half is filled with sand of 0.5 to 1.0 mm diameter. The purpose of the sand overlaying the gravel is to draw effluent up from the gravels by capillary action. Effluent can then be utilised by plants, selected for their high transpiration rates, planted over the bed. The design is shown in Figure 9 below.

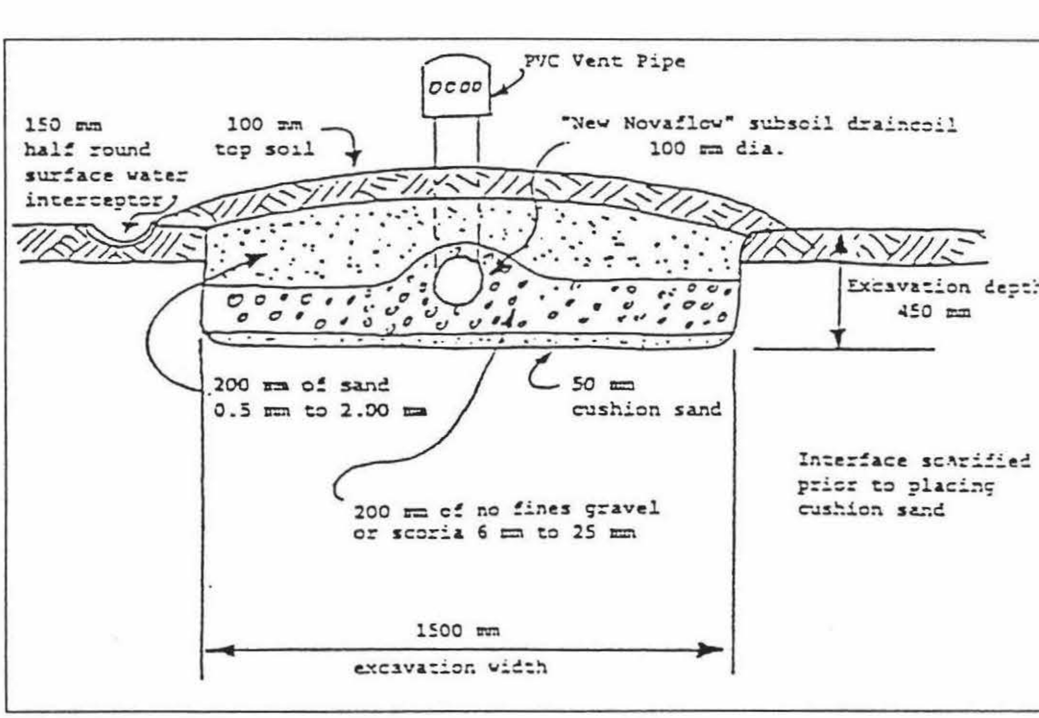


Figure 9. Evapo-transpiration seepage bed design (Gunn, 1989).

An ETS system was installed in Leigh (NZ) for disposal of a high wastewater load into silty clay (Gunn, 1987). The design incorporated dosed loading between four groups of beds, so that hydraulic load per bed was 44 mm/day, and the overall areal load was 11 mm/day. The combined bed area was able to absorb all effluent discharged, even during wet weather, primarily because of the three week rest period between loadings. The periodic high hydraulic loading of this design produced periodic saturated conditions in each bed because of the accumulation of effluent. Grass growing on the beds was assumed to help in the removal of effluent but treatment opportunities would have been reduced by the saturated conditions.

Experimental research on evapo-transpiration beds has been carried out in Australia to establish their usefulness in remote very dry areas with impermeable soils (McGrath *et al.*, 1991). Vegetation was found to be an important component of the system, with evapo-transpiration rates from planted beds considerably higher than from unplanted beds. The particularly low rainfall in the area (200 - 300 mm per year) and high evaporation rates (over 3,000 mm per year) is not applicable to most New Zealand climatic situations. Evapo-transpiration beds here are more likely to operate as soil infiltration systems, with some assistance from plant transpiration, as the Leigh system did.

The Wisconsin mound was developed for sites limited by soils of low permeability (less than 60 cm/day) and high water tables (shallower than one metre). A pressure distribution system is placed in gravel within the mound fill, which is sand. The theory of constructing a mound above the original ground surface is that it provides a volume of unsaturated soil medium for the treatment of effluent before the effluent infiltrates into the native soil. A cross-sectional diagram is shown in Figure 10 below.

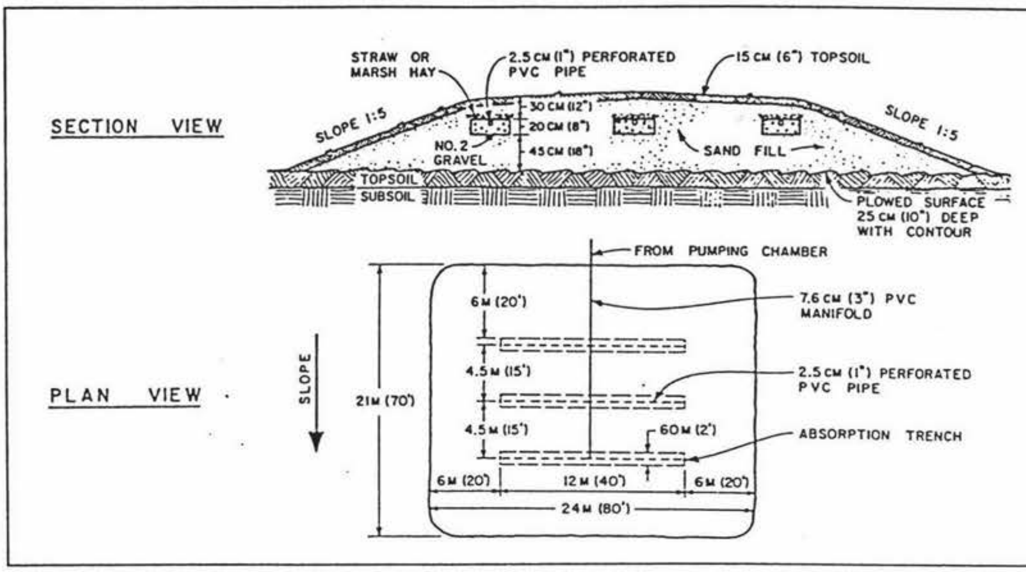


Figure 10. Wisconsin mound design (Bouma *et al.*, 1975).

Wisconsin mound systems would be appropriate in the Manawatu area because of the restrictive soil permeabilities and high water tables. Despite their suitability, the relevant district councils are not aware of any mound type soil absorption areas being installed in their districts.

4.2.4 On-site nitrogen removal systems

Methods to enhance nitrogen removal in ST-SAS have been investigated since the association between septic tank location and elevated nitrate concentrations in groundwater was observed in the early 1970s. Possible options include separation of toilet wastes, which contain about 75 percent of total nitrogen in domestic wastewater, to a separate treatment system such as a composting toilet; physical/chemical separation systems, developed for specific industrial purposes and not usually used in on-site systems; and biological denitrification, which has received the most investigation for on-site utilisation. Nitrogen removal performance comparisons show that source separation of toilet wastes achieves 60-90 percent removal, physical/chemical separation systems achieve over 90 percent removal, and biological systems achieve 50-60 percent removal (Whitmyer *et al.*, 1991).

Separately treating toilet wastes in a composting toilet removes nitrogen before it reaches the septic tank system. Some household composting toilets are operating successfully in New Zealand but coping with public use in National Parks is more difficult (Odell, 1993). Problems can arise if the aerobic composting process in the chamber fails. A commitment to ongoing maintenance of the system is usually necessary and this is a disadvantage of the system compared to a conventional ST-SAS.

Whitmyer *et al.* (1991) identified nine on-site systems in the United States that incorporated a biological denitrification component. These are the extended aeration package plant, the aerobic/anaerobic trickling filter package plant, peat filter, the RUCK system, recirculating sand filter, recirculating sand filter with anaerobic filter, recirculating sand filter with anaerobic filter and carbon source, recirculating sand/rock storage filter, and mound with constructed wetland. Best ranking systems according to nitrogen removal efficiency, removal consistency, and service reliability were the peat filter, various configurations of the recirculating sand filter, and the RUCK. Of these, the peat filter and the RUCK are mechanically simple and require little maintenance. The designs of these systems, and the more recently developed sawdust filters, are described below.

Laak (1988) developed an underground treatment system, called the RUCK system, where toilet wastes are separated from the wastewater flow and directed to a septic tank followed by an underdrained vented sand filter where ammonia is nitrified. Effluent from the sand filter is then added to separately collected kitchen and laundry wastewater in a second septic tank. In the anoxic conditions of the second tank, the greywater serves as an organic carbon source for biological denitrification. The additional advantages of this system is that the oxidation of carbon compounds in the sand filter lowers the pH, thereby facilitating removal of phosphorus and accelerating the rate of die-off of bacteria and adsorption of viruses.

Lamb *et al.* (1991) assessed the performance of a buried sand filter/greywater system designed according to Laak's RUCK system specifications. They found that the combination of a buried sand filter and an anaerobic tank using greywater as a carbon source can remove about 50 percent of the nitrogen in household wastewater prior to discharging to a SAA. Incomplete nitrification in the sand filter was a major limiting factor in the system. This could have been caused by insufficient oxygen.

Methanol has been used as a carbon source for denitrification of septic tank effluent in experimental batch and continuous flow packed columns (Sikora and Keeney, 1976), and has been compared with using wastewater generated on-site to reduce costs. Lamb *et al.* (1990) constructed a re-circulating sand filter/rock tank system (RSF system) and compared the denitrification rates using untreated septic effluent, methanol, and ethanol as a carbon source. The RSF system comprised an above ground re-circulating sand filter, receiving a total hydraulic loading of 155-195 litres per square metre per day (mm/day), a re-circulation ratio of 4:1 to 5:1, and a forward flow of 39 l/m^2 per day; and a buried rock tank, designed with a three day retention period. They found that methanol and ethanol achieved mean total N-removals of 78-82 percent compared to 36 percent removal using septic tank effluent as the carbon source. Overall, they recommended the use of ethanol because it does not have the toxicity risks of methanol, but cautioned that the maintenance requirements may be too high for general householder use.

Peat filters can be installed in a layer over sand below the distribution pipes in the SAA. A 30 percent net loss in Nitrogen was achieved in reactor trials where saturated peat was dosed at 30 mm/day (Winkler and Veneman, 1991). The same reduction was not achieved in unsaturated conditions. The advantages of these nitrate reductions could be outweighed by possible high transport of bacteria and viruses in saturated conditions.

Installation of reactive porous media barriers under SAA has recently been investigated for passive denitrification of septic tank effluent (Robertson and Cherry, 1995). A layer of silty material mixed with a solid organic carbon source such as sawdust is placed under a coarse grained sand layer in the SAA. Alternatively, the layers can be added to existing systems vertically downgradient of the effluent plume flowpath. Both configurations achieved 60 - 100 percent attenuation with input nitrate levels of 125 mg/l as N. Advantages of this system are that they require no energy input, they are simple to construct, and require no alteration of plumbing fixtures. The rate of carbon uptake in the sawdust barrier indicated that sawdust has the potential to last for decades without any need for replacement.

Effluent dosing may increase denitrification potential up to 40 - 45 percent because of the alternating dry/wet conditions (Ritter and Eastburn, 1988). Specially designed and operated systems could take advantage of this, and remove the need for add-on components, although timing the periods for wet and dry conditions requires development.

Designing ST-SAS to improve nitrogen removal is in its infancy in New Zealand. The use of wetland systems following a batch type aerobic system has been developed recently (Bickers *et al.*, 1995 unpublished report) and the use of sphagnum moss has been investigated at Lincoln University (Dakers, 1995, personal communication).

Nitrate contamination of ground water is the key factor influencing the design of systems for the land treatment of agricultural waste. Two methods of reducing overall nitrate loadings in that situation are by crop uptake and harvest, and by denitrification. Barkle *et al.* (1993) concluded that the potential removal rate of nitrogen, applied to soil surfaces in the form of agricultural waste, by volatilisation and crop uptake mechanisms are limited to relatively low values. The exception is if the crop, such as hay is grown, and removed off site. While some nitrates may be used by grass and other plants over shallow SAAs, nitrate removal rates from soil absorption areas by crop uptake has not been investigated.

4.2.5 Soakage area design in the New Zealand Standards

Soakage area designs recommended in the New Zealand Standards (NZS 4610:1982) are shown in Figure 6 in Section 4.2.3 above. The standards are currently under review because it has been found that recommendations in the standards are being universally applied with very little regard to site specific conditions such as depth to groundwater and likely household wastewater volumes. The more recent technical publication from the Auckland Water Board (Gunn, 1989) is generally used throughout New Zealand for SAA design criteria. The New Zealand standards recommend that the soakage area is designed according to the quantity of wastewater produced, the soil profile, soil percolation rates, the levels and rates of movement of groundwater and surface water, and local knowledge.

Guidance is provided about design wastewater quantities - 140 litres per person per day, or 700 litres per day per household. Other design criteria, including recommended trench sizes for the different soil types, is included but is only specific to the soil characteristics, in particular, the percolation rate, and does not vary according to varying design loadings from the tank.

For rapid to very rapid draining environments with percolation rates of 150 mm/hour or greater, soakaway systems or shallow seepage trenches are recommended. Alternatively, seepage trenches no less than 20 m long filled with graded media of 50 mm to 70 mm can be used. For a four person household with rooftop water supply, a 20 metre trench 450 mm wide (nine square metres) would have an hydraulic loading of 62 mm/day provided effluent was uniformly distributed in the trench.

Sandy loams and silt loams with percolation rates of 60 mm/hour to 150 mm/hour are recommended to have shallow seepage trenches of 40 metres and 60 metres length respectively. Corresponding hydraulic loading rates for the same four person household would be 31 and 20 mm/day.

For slow draining environments with percolation rates of less than 25 mm/hour the standards recommend that the SAA be designed based on a full engineering investigation. In this situation, however, there is very little design information for an engineer to make use of. Established ST-SAS operation and performance in New Zealand conditions has received very little attention and so the (widely used) recommendations in the Auckland Water Board's technical publication (Gunn, 1989) are mostly based on overseas investigations.

The standards recommend that effluent is distributed through field pipes laid 6 to 12 mm apart, or perforated pipeline with perforations comprising not less than 2 percent of the surface area. The pipes are to be laid flat, or at least at a gradient no more than 1 in 200, in gravel media in the trench. Flexible 'nova flow' field drainage pipes are commonly used, although it is not possible to ensure that they are installed level or remain level.

4.2.6 Soakage field design shortfalls

The key shortfalls of soil absorption area design in the Standards are the lack of consideration for keeping the system shallow, lack of direction for even distribution, high recommended hydraulic loading rates, no consideration to maintain unsaturated conditions below the SAA, and no mention of denitrification possibilities in at risk areas.

Aerobic, unsaturated conditions in the soil absorption area are important to its treatment performance. These conditions are unlikely to be present in trenches 400 mm deep where distribution is gravity fed at such high hydraulic loadings. Deeper soakage areas have less potential for microbial activity, and less opportunity for oxygen diffusion into the layers beneath the soakage area, particularly in saturated conditions.

Trench designs for SAA in all soils, and particularly sandy soils, would initially allow rapid drainage to groundwater without any process to remove or reduce nitrogen or pathogens. Even if uniformly applied, the hydraulic loading is greater than that required to avoid clogging of the infiltrative surface determined by Simons and Magdoff (1979).

Nitrate loadings to soil absorption areas hydraulically loaded at 10 to 50 mm/day are 146 to 730 g N/m^2/year (Section 4.1.3). This represents a nitrate loading at that particular place ten to fifty times the required areal nitrate loadings onto land where there is risk of groundwater contamination (15 g N/m^2/year, or 150 kg N/ha/year). But these high loadings only occur within the trench, which has an area between nine and eighteen square metres, or 0.09 and 0.18 percent of one hectare. To determine the collective potential nitrate loadings from groups of ST-SAS, the actual loaded areas need to be assessed with regard to the recommended hydraulic rates in the standards. The hydraulic loadings for the soakage areas in the standards are between 20 and 62 mm/day (292 g N/m^2/year and 905 g N/m^2/year). Thus, in at risk areas, for one hectare with sandy soil a total of 18.4 four-person houses served by ST-SAS each having nine square metres of soakage trench would contribute the allowable nitrate loading for that hectare. In areas where groundwater is not considered at risk, there could be 24.6 similar households.

The number of households per hectare calculated in this way takes no account of surrounding land uses, such as pastoral farming, which may contribute additional nitrate loadings and further, are based on the assumption that there will be opportunity for nitrate uptake by plants, as happens when effluent is applied onto land. In fact, the trench is well below the plant rooting zone of grass, the most common plant grown over SAA, and most nitrate will migrate from the trench to groundwater. Safe concentrations of ST-SAS per hectare could be determined by calculating the available dilution capacity of the groundwater but this depends on groundwater quantities and movement, which vary considerably from place to place. While these calculations indicate that nitrate is likely to leach to groundwater if there are more than 18.4 systems per hectare, they do not determine the concentration that would avoid leaching, or even what concentration would not cause groundwater nitrate concentrations to be elevated above health standards. Canter and Knox (1985) suggest that contamination of groundwater can occur in American conditions if there are more than 40 systems per square mile (one per 10 hectares). This extremely low concentration is not possible in coastal communities or in some 'lifestyle-block' rural areas, and so systems that remove nitrates by denitrification would be the most practicable option for reducing nitrate contamination.

Chapter Five
Improving ST-SAS performance

5.1 *Recognising the need to improve ST-SAS performance*

Despite the findings of investigative research undertaken overseas during the last three decades about design features capable of enhancing ST-SAS performance, there is little evidence that the knowledge gained has been applied by regulatory authorities in New Zealand. There are two main reasons for this. First, because professional interest here has focussed on reticulated community systems, and second, because there has been little recognition that existing designs pose potential adverse effects on the environment and public health. Most households reliant on on-site sewage disposal are either in low density rural environments where the cumulative effects from septic tank contamination have been assumed to be minor, or they are in coastal or lakeside holiday home communities where there is often excellent drainage, intermittent household occupancy and no incentive to install 'state-of-the-art' sewage disposal facilities. A situation has developed where rural communities are reliant on groundwater that is not protected as far as practicable from contamination by ST-SAS, and where small 'bach areas' previously occupied seasonally are now permanent communities with probable associated effects on groundwater. There has been some New Zealand research to estimate the level of groundwater contamination likely to occur from ST-SAS, and some regional councils suspect that nitrate and bacterial contamination of groundwater in their regions may be caused by ST-SAS, but actual contamination levels directly attributable to ST-SAS has not been established in any study. The lack of investigation into the environmental effects of ST-SAS may explain why there has been little incentive for regulatory authorities to require improvements to the standard designs.

The standard single chambered tank provides low actual hydraulic retention time, poor settling opportunities, and allows re-suspended sludge to be washed out with the effluent. The standard soil absorption area promotes saturated anaerobic conditions where there is little further treatment and pathogens and nitrates are released to the environment. The combination of these features is that the ST-SAS appear to be designed to fail.

The Ecotank™ is available throughout New Zealand and includes all the essential components to improve effluent quality to a standard suitable for soil infiltration, except in regard to nitrogen concentrations. This type of tank is specifically required by the Dunedin City Council for on-site sewage disposal but there is little promotion of it elsewhere in New Zealand. The engineering technology relating to low pressure distribution systems and their suitability for sandy soil conditions is described in the widely available technical publication No. 58 of the Auckland Water Board (Gunn, 1989). The value of uniform effluent distribution in soil absorption areas of all soil types is not widely recognised, however, and there are no requirements to design soil absorption areas that promote unsaturated conditions. Site constraints, such as high water tables and poorly draining soils, are common throughout the country. Regulatory authorities should be requiring advanced designs that optimise ST-SAS performance capabilities in these areas but this is unlikely to happen while the standards support the use of the standard single tank, and while there is no New Zealand design manual for designing systems for adverse conditions.

5.2 *Some examples of regulatory requirements and current practice*

Regulation of ST-SAS installation and the discharges from them falls under the jurisdiction of both regional and district councils. Examples of the different management requirements of a regional and district council, and an assessment of those requirements, are given below.

Discharges of septic tank effluent to land or water in the Manawatu catchment are controlled under provisions in the Manawatu Catchment Water Quality Regional Plan (Manawatu-Wanganui Regional Council, 1995). The Plan allows most discharges from domestic on-site sewage systems as Permitted Activities provided specified conditions are met. This essentially allows district councils to apply site-specific requirements about the tank and the soil absorption area with their powers under the Building Act. The relevant rule in the regional council's plan is as follows:

MCWQ Rule 11 On-site sewage discharge into ground

Any discharge into land of effluent from an on-site septic tank system or of effluent from an on-site aerobic sewage treatment system is a **Permitted Activity** provided:

a. the design discharge to the treatment system is not more than 1620 litres per day [calculated according to design flows in Gunn, 1989];

b. there is no effluent seepage to the ground surface and there is no objectionable odour at the property boundary;

c. the discharge is sited at least 20 metres from any river, lake, natural wetland or artificial watercourse, where the distance to the nearest river or lake will be measured as follows:

   i. from the edge of the bank contiguous with the bed of the river or lake;

   or, where there is no bank.

   ii. for any river, from the limit of the bed covered by the annual fullest flow; or

   iii. for any lake, from the limit of the bed covered by the annual highest water level;

d. there is no discharge of effluent to water, including groundwater; and

e. there are no more than five systems per hectare.

This rule does not include any requirement about the design of the septic tank itself, and relies on district councils requiring designs specific to the particular household and site. Any discharge that does not meet the conditions in this rule requires a discharge permit from the regional council, or is subject to enforcement under the Resource Management Act. Condition (a) limits the volume of effluent to the quantity expected from a large household. Any volume greater than this, such as would be expected from a school or community facility, requires a discharge permit. Condition (b) is 'effects-based', and relies on district councils applying appropriate restrictions on septic tanks, soil absorption areas, and method of distribution. This condition also ensures that householders can be required to restore or replace a system that fails because of a clogged soil absorption area. Condition (c) ensures that there is additional treatment in the soil before any effluent percolates to surface water, but takes no account of soil type. Condition (d) primarily addresses Maori concerns regarding direct discharges of human waste to water and ensures that soakholes are not installed over shallow groundwater. Condition (e) limits the areal concentration of systems providing some protection of groundwater from nitrate contamination. The rule provides little incentive or requirements for improving effluent quality from the tank or the soil absorption area. This is left to the discretion of the district councils.

In most cases, because of the relatively permissive conditions allowed by the regional council, the responsibility for ST-SAS design rests with plumbing and drainage staff, environmental health officers, and building inspectors at the district council. This is appropriate because those staff are considered to have more detailed knowledge of local conditions and the performance of various systems in those conditions. District council staff have sufficient powers under the Building Act and the Health Act to stipulate site specific requirements for every system on a case by case basis.

District council requirements are generally based on the New Zealand standards and local experience. The standard single septic tank of 2,700 or 3,300 litre capacity without compartments or baffles is installed universally in the Manawatu area, except in the area around Palmerston North within the boundaries of the Palmerston North City Council, where dual tanks are required. Soil absorption areas are designed according to the New Zealand standards and local experience. Specialised tanks such as the Ecotank™ with the anaerobic filter, or aerobic package plants still represent a very small section of the market. Specialised soil absorption areas such as pressure distribution are rarely required, except by Palmerston North City Council, and a Wisconsin mound type system has not been installed anywhere in the area (Bland, 1996, personal communication; Lawrence, 1996, personal communication).

In the beach communities of Himatangi and Tangimoana, for example, the Manawatu District Council allows standard single tanks and requires ten to twenty square metres soakage area. This soakage area, which is sealed over the top with concrete, comprises graded sand contained within concrete blocks. Septic tank effluent is discharged to the soakage area without any method of distribution. That is, effluent displaced from the tank by incoming flows discharges to the beginning of the soakage area and percolates to groundwater. When the soakage area clogs up, the sand is removed and replaced. Installation of the standard single tanks, which are unbaffled and without filters, combined with unpressurised distribution into a small sand box is likely to result in high hydraulic loadings of poor quality effluent. The likely environmental effects are contamination of groundwater by pathogens and nitrates. This can pose a public health risk, particularly in Tangimoana where many residents are reliant on bore water supply. Frequent system failures from blocked soakage are also likely, resulting in objectionable odours.

For on-site disposal in less permeable soils, the Manawatu District Council requires soil absorption areas at least sixty metres long. A typical design is shown in Figure 11 below.

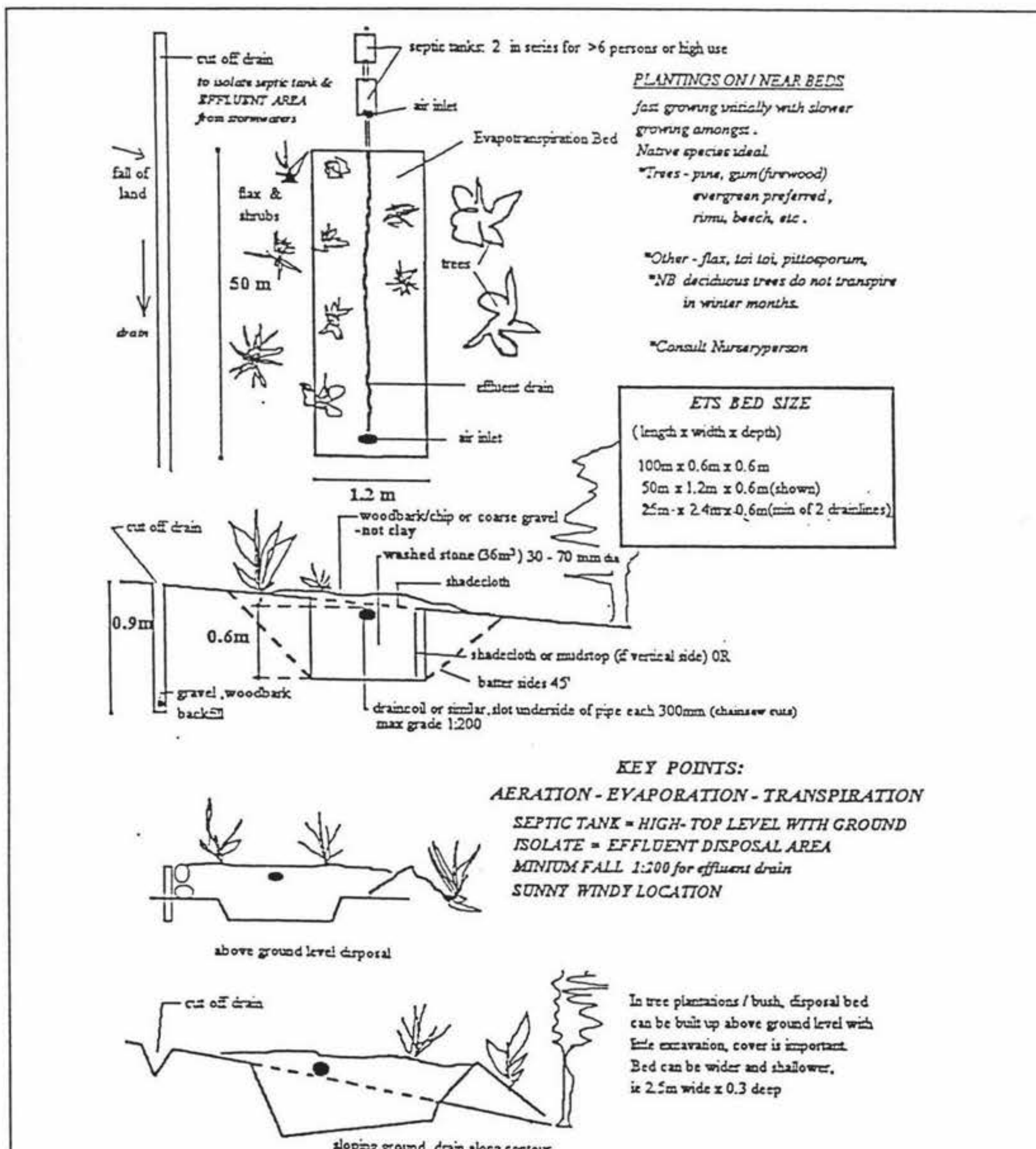


Figure 11. Soil absorption system (Manawatu District Council, 1995)

The design in Figure 11 allows effluent to be gravity-fed through flexible 'nova flow' drainage lines. The design shows an evapo-transpiration seepage bed 50 metres long by 1.2 metres wide but minimum sizes currently allowed in silty loams are usually 60 metres by 1.2 metres (Chris Bland, personal communication). The hydraulic loading from a four person household producing 140 litres of wastewater per person per day on an SAA of this area would be 7.7 mm/day, but only provided the total soakage area of 72 square metres is taken as the infiltrative surface. This design may cause saturated conditions in and under the SAA. First, because of the extremely low saturated hydraulic conductivity of many silt loams in the Manawatu area (Section 4.2.1); second, there is likely to be very high localised hydraulic loading at the beginning of the bed because effluent is gravity-fed rather than distributed under pressure; and third, because there is no sand layer above the distribution line to draw moisture towards the plant rooting zone.

Saturated conditions limit aerobic activity in the bed and reduce treatment opportunities. The 1.2 metre wide bed design also allows less oxygen transmission through the sidewalls than a trench design of the same area but this is compensated to some extent by venting the SAA. Finally, the bed depth is 600 mm, which is well below the depth where most microbial activity in the soil environment takes place and so there will less destruction of pathogenic organisms than there would be in a shallower bed.

Soils of low permeability are particularly prone to clogging and this is accelerated by high hydraulic loadings combined with poor effluent quality. There is no requirement to install a tank designed to reduce effluent suspended solids and BOD, such as would be achieved with the incorporation of anaerobic filters and outlet baffles, although two tanks in series are required for occupancies greater than six people, or for high use. This type of design installed in slowly permeable soils is likely to result in a build-up of the biological mat, eventually allowing a transmission route for disease to become established by vectors such as flies and household pets through surface ponding.

The above examples demonstrate that while information is available in overseas literature about the benefits of increasing sedimentation opportunities and retaining sludge in the tank, about potential nitrate and pathogen contamination of groundwater from saturated soil absorption areas, and about specialised designs such as the Wisconsin mound which are appropriate to install in heavy soils, this knowledge is not necessarily applied to ST-SAS requirements at a regulatory level.

5.3 *Optimising performance capabilities*

It is not necessary for soil absorption areas to fail because they are blocked by poor quality effluent or high hydraulic loadings. Septic tank design features can improve effluent quality by decreasing effluent BOD and suspended solids. Soil absorption area design features can improve effluent distribution and avoid creeping failure caused by localised high hydraulic loading rates. A range of options are available to decrease nitrogen loadings from the system. A septic tank design that enables the best possible treatment performance must recognise the following factors.

- septic tank design features affecting septic tank effluent quality are the size and shape of the tank, the degree and effectiveness of compartmentalisation, and the frequency of de-sludging;
- dual or multi-chambered tanks have consistently better suspended solids and BOD removal than single tanks;
- retaining septic tank sludge in the tank reduces effluent BOD, suspended solids, nitrogen, and bacterial and viral concentrations;
- baffles installed under the pipes connecting septic tank compartments and the outlet pipes decrease opportunities for re-suspended sludge to be carried out with the effluent;
- outlet pipe tee joints reduce the possibility for scum and floatables to be carried out with the effluent;
- phosphate free detergents can produce better septic tank effluent, in terms of BOD and suspended solids, than detergents with phosphate;
- anaerobic filters are suitable for treating the range of wastewater quantities and qualities generated by domestic households;
- tanks incorporating anaerobic filters can achieve consistently better effluent quality than tanks without filters;
- the best performance, in terms of BOD and suspended solids removal, in tanks incorporating anaerobic filters is achieved by high hydraulic retention times (more than three or four days) rather than high temperatures;
- anaerobic sludge digestion in the tank may be inhibited by low pH values at the bottom of the tank, or by the presence of heavy metals;
- there is better sludge decomposition after the first year of operation;

- the rate of sludge build-up in the tank is slower when the wastewater retention time is increased; and
- in dual or multi-compartment tanks most of the sludge is retained in the first compartment.

Septic tank designs that recognise all these factors have dual or multi-chambered tanks, with baffles and outlet tees, and large volumes for sludge storage. Providing sludge storage capacity for at least five years takes advantage of the lower accumulation rates that occur after the first year and allows better decomposition of the stored sludge.

Increasing sludge storage capacity should be achieved by increasing the tank length and width rather than increasing the height. This allows a slower rate of increase in sludge depth with a corresponding longer period for the sludge level to approach the outlet pipe.

Sludge storage space can be calculated using an estimated sludge accumulation rate of between 50 and 100 litres per person year (Section 3.1.5). A four person household with a sludge accumulation rate of 80 litres per person per year would require 1600 litres storage for five years. At that time the sludge would be 744 mm deep in the standard 2700 litre tank, or 680 mm deep in the standard 3300 litre tank. If the larger 3300 litre tank is used as a primary tank in dual tank system, and the outlet pipe is 500 mm deep, the depth from the top of the sludge to the bottom of the outlet pipe would be 220 mm after five years. Without baffles, the settled sludge would easily be disturbed by the incoming wastewater flow and carried through to the second compartment. If there is no second compartment, sludge would be carried over to the soil absorption area.

A soil absorption area design that enables the best possible performance must recognise the following factors.

- poor septic tank effluent quality increases the build-up of the biological mat and reduces virus adsorption;
- almost all effluent gravity-fed to the soil absorption area discharges at the beginning of the trench and is not distributed to the available infiltrative surface;

- poor septic tank effluent distribution causes saturated anaerobic conditions under the soil absorption area;
- saturated soils provide poor BOD reduction, and worse bacterial and virus die-off than unsaturated soils;
- topsoils are more aerobic than deeper soil layers, and is where microbial competition with pathogens is more intense;
- anaerobic saturated conditions under the soakage area allow phosphorus leaching, heavy metal leaching, and help transport bacteria and viruses to groundwater;
- physical straining, either in the biological mat or in soils with low hydraulic conductivity, provides an important removal mechanism for bacteria and viruses;
- nitrates are not removed in the soil absorption area unless specific denitrifying components are included in the design;
- there is no known 'safe' concentration of systems per hectare in terms of protecting groundwater from nitrate contamination.

Soil absorption area designs that recognise all these factors incorporate pressurised effluent distribution lines; allow maximum hydraulic loading rates of 30 mm/day even in sandy soils; have a depth no more than 400 mm; ensure maximum possible distances between groundwater levels and the bottom of the soakage area; and allow the development of a biological mat in sandy soils.

Establishing a passive denitrification zone in the system is a more user-friendly option for decreasing the quantities of nitrate reaching groundwater than constructing separate components such as re-circulating sand filters. Where groundwater is at risk, ST-SAS designs incorporating passive de-nitrification should be installed, or the areal concentration of ST-SAS should be limited to low numbers.

Chapter Six
Conclusions and recommendations

Septic tank soil absorption systems are intended to function as on-site domestic sewage treatment and disposal systems. Standard New Zealand ST-SAS designs can be substantially improved to enhance their performance of these functions. There is very little householder maintenance of, or attention paid to, on-site wastewater treatment systems after they are installed unless there is a soakage field failure or problems with drainage from the house. Septic tanks that require low energy input, in terms of both power and householder time, and that can consistently achieve a level of effluent quality suitable for infiltration into the soil, are desirable for providing long-term reliable performance. Wherever possible, the entire design should avoid the use of complex technology that requires dedicated maintenance.

Effective treatment of domestic wastewater in a septic tank soil absorption system requires removal of biochemical oxygen demand and suspended solids. As far as possible, the treatment should also remove pathogens and nitrogen compounds, which are the pollutants of primary concern in on-site disposal.

High septic tank effluent quality, in terms of low biochemical oxygen demand and suspended solids concentrations, allows better infiltration in the soil absorption area. Septic tank design features that affect effluent quality are the size and shape of the tank, the degree and effectiveness of compartmentalisation, the incorporation of anaerobic filters, and the frequency of de-sludging.

The most significant contaminants present in septic tank effluent in terms of their threat to groundwater quality are pathogens and nitrogen compounds. Many bacteria and viruses are attached to or embedded in solid matter and will settle in the sludge, which also comprises nitrogen compounds. Retaining sludge in the tank is therefore a key function of the tank, because not only are effluent biochemical oxygen demand and suspended solids concentrations reduced, but the release of pathogens and, to a lesser extent nitrogen compounds, to the environment is also reduced. To achieve this function, tanks require at least two compartments to keep most of the sludge away from the outlet pipe; they require baffled connections between the compartments to avoid re-suspended sludge being carried over to the second chamber; and they require long wastewater hydraulic retention times.

To achieve a long hydraulic retention time, even with large amounts of accumulated sludge, tanks require a volume up to five times the daily volume of wastewater expected from the household. The volume should be achieved so that the tank shape is longer rather than deeper to reduce the rate at which the top of the sludge approaches the outlet pipe. Dual compartment tanks increase actual mean hydraulic retention times by reducing wastewater short-circuiting, and allow better conditions for settling. Anaerobic filters, particularly packed media filters, consistently reduce effluent biochemical oxygen demand and suspended solids concentrations.

The outlet pipe from the second chamber should be a tee joint, open at the top end to allow tank gases to escape through to the soil absorption area. The bottom end of the outlet pipe should be deep enough to avoid scum passing out with the effluent.

Large sludge storage capacity takes advantage of the better decomposition that occurs over time. Sludge accumulation rates reported in the literature are generally in the range of sixty to one hundred litres per person per year. This accumulation rate decreases over time, particularly after the first year.

Soil absorption areas provide the best effluent treatment and the lowest failure rates from clogging when the effluent is uniformly distributed at such a rate that unsaturated conditions are maintained beneath the soakage field. This is best achieved in permeable soils by low pressure distribution at rates less than 30 mm/day and in slowly permeable soils at rates less than 10 mm/day. Unsaturated conditions in and below the soil absorption area allow better biochemical oxygen demand reduction, and better bacteria and virus removal than saturated conditions. An established biological mat at the bottom of the soil absorption area helps provide additional treatment, particularly in the removal of pathogenic organisms. Failure to establish a biological mat, as could happen if the hydraulic loading is extremely low, deprives the system of a valuable component of its treatment capacity.

The physical and chemical processes of straining and absorption in the biological mat keep pathogenic organisms in the upper layer where microbial competition and predation is most intense. This then avoids their transport to groundwater where their survival rates are better and their transport to humans more likely. Soil absorption areas should therefore be installed as shallow as possible to take advantage of more aerobic conditions and more microbial activity.

Uniform distribution, which is advantageous in all soil types because it ensures that the entire soakage area is utilised, can be achieved with pressure distribution. Passive de-nitrification of septic tank effluent prior to discharging to the environment avoids potential elevation of nitrate levels in groundwater. Systems suitable for on-site de-nitrification are the peat filter, the RUCK system, and sawdust filters.

A revision of the New Zealand standards that incorporates all these design features is necessary before any change in regulatory requirements is likely to be implemented by the regulatory authorities. In addition, there needs to be a septic tank soil absorption system design manual developed for installations in adverse conditions.

There has been very little study of the operation and performance of septic tank soil absorption systems in New Zealand conditions, or the environmental effects directly attributable to them. The actual and potential effects of discharges from these systems could be better assessed if New Zealand research was undertaken about the quality and quantity of wastewater loads placed on on-site systems, the tank environment, and the quality of the discharge from the tank. Specifically, the following research should be undertaken:

1. the variations in New Zealand wastewater production according to numbers of people and types of plumbing fixtures, and an assessment of actual wastewater hydraulic retention times in single tanks and dual tanks.
2. environmental conditions in New Zealand septic tanks, in particular dissolved oxygen and pH variations over time and in the sludge layer, and the presence of inhibitory conditions such as heavy metal concentrations in the sludge layer.
3. volumes of sludge produced per person per year in single tanks, dual tanks, and tanks incorporating anaerobic filters.
4. de-nitrification possibilities from various on-site passive systems.

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