UNIVERSITY OF SOUTHERN QUEENSLAND
Faculty of Engineering and Surveying

Applied Research on On-site and Small Community Wastewater Treatment and Effluent Disposal in Australasia

A Dissertation Submitted by

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Abstract

The writer recognised in the early 1970s that there was a need to focus on improvements to on-site wastewater treatment technology, effluent disposal and associated aspects of water pollution control. At the time there was considerable potential for applied research in the development of improved treatment and effluent disposal technologies since some 16 to 20% of the populations in New Zealand and Australia relied on on-site or decentralised wastewater treatment and effluent disposal.

This thesis covers applied research projects undertaken in New Zealand, during the period 1974-1990, and in Queensland from 1992 to the present day. These experiences have been written up in the form of a guide to the dissertation (contained within the initial three chapters) and eight technical portfolios.

During University of Auckland post-graduate studies in 1974 the writer reviewed the unique problems of widely fluctuating wastewater loads from coastal and lakeside communities, camping grounds and resort areas. The review evaluated improvements to septic tanks, alternative effluent disposal systems, upflow pebble filters, lagoons, rotating biological discs, and composting toilets. Solutions for handling fluctuating wastewater loads have been further extended.

Due to failed effluent trenches in Northland New Zealand, associated mainly with problem clay type soils, the writer undertook experimental work on the determination of soil permeability by field and laboratory techniques, investigated soil properties impacting on effluent disposal and researched alternative methods of on-site effluent treatment during the period 1976-1979. This included experimental work on evapotranspiration (ET) systems and the compilation of selected plants, shrubs and trees for planting within ET systems in the Northland and Auckland areas. This work has been directly applied to much of Queensland.

An experimental *Clivus* Type domestic composting toilet was established in 1976 and monitored in Kerikeri, Northland, New Zealand. The design of associated greywater and disposal systems consequently took place in New Zealand and Queensland.

The then Queensland Department of Primary Industries (Water Resources) identified the need to develop alternative wastewater treatment technologies in the early 1990s. The writer set up an Artificial Wetlands Research and Advisory Committee to develop this technology within the range of climatic conditions in Queensland. This thesis focuses on the design rationale, nitrification and de-nitrification of effluent in reed/gravel beds or sub-surface wetlands.

A postal survey of Australian effluent disposal systems was undertaken during the period 2000-2001. This showed that a broad range of disposal techniques were being used.

A range of innovative (more non-conventional) treatment and disposal systems have been presented for use in Australasia.

Raised sand mounds for effluent treatment have not been used on a wide spread basis in New Zealand and much of Australia. The performance of an existing sand mound, the design based on AS/NZS 1547:2000, located in Morayfield, Queensland has been assessed.

A survey of deep shaft disposal was undertaken over much of the North Island of New Zealand over the period 1976-1977. This unique method of post-treatment and effluent disposal has merit in locations where shallow trenching is not appropriate and in deeper more permeable soils, that are clear of the groundwater.

Experimental work on the use of lime and solar salt brines with seawater and magnesium salts, for treating municipal wastewater and a range of process waters was undertaken in New Zealand over the period 1984-1986. This work was extended as a joint research project with the Queensland University of Technology in the mid-2000s.

This thesis reflects on past experimental and innovative work on on-site wastewater treatment and effluent disposal in Australasia and it recommends improvements and/or alternative technologies that can used to achieve higher standards of treatment. It consists of three chapters and eight portfolios covering separate topics. The connections between the chapters and the portfolios are shown in Figure 2a of Chapter 2. The conclusions drawn from each portfolio are collated and summarized in Chapter 3.

Copies of selected papers and technical reports are compiled in a separate Volume 2.

Certificate of Dissertation

I certify that the ideas, experimental work, results, analyses and conclusions reported in this dissertation are entirely my own effort, except where otherwise acknowledged.

I also certify that the work is original and has not been previously submitted for any other award, except where otherwise acknowledged.

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Signature of Candidate   Date

ENDORSEMENT

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Signature of Principal Supervisor   Date

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Signature of Associate Supervisor   Date

Acknowledgements

Over the past 35 years several people have given me support with my studies and research projects.

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Finally, the support of my wife Kay has been most appreciated, both during the research projects since 1981and during the compilation of this dissertation.

I wish to dedicate this dissertation to my late father Stan, who took an interest in my work and research involvement, and passed on his innovative skills.

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Glossary of Terms and Abbreviations

Absorption trench
The land application and/or uptake of effluent into the soil by infiltration and capillary action.

Activated sludge
A dark brown suspension of bacterial flocs produced by the aeration of wastewater.

Aerobic digestion
Occurs in conditions where high dissolved oxygen exists.

Aggregation
The process whereby small particles cluster together, due to particle attraction forces.

Ammonia
Ammonia is a characteristic breakdown product of organic matter and it accounts for 90 % of the nitrogen in wastewater. It is expressed in mg/L.

Anaerobic digestion
Anaerobic (oxygen free) digestion occurs in a septic tank. Organic matter is broken down or digested by anaerobic bacteria to form methane and other soluble organic products.

Anoxic
The process by which nitrate nitrogen is biologically converted to nitrogen gas in the absence of oxygen.

Aspect ratio
The ratio of wetland or lagoon length to width.

Attenuation
The progressive reduction of contaminations in effluent. It is the result of physical, hydrological, biological and chemical processes including plant uptake, absorption, precipitation, dilution, effluent losses by evapotranspiration and nitrogen gas loss through de-nitrification.

Bacteria
Single celled organisms that have no nucleus.

Biofilm
An organic layer typically composed of algae, microfauna and bacteria, which adsorb small particles and nutrients.

Biological filter
A porous bed of suitable, graded and inert material. Bacteria and other organisms flourish on the surface of this material to bring about oxidation of the organic matter in the settled wastewater applied to the filter.

BOD_5
Biochemical oxygen demand consumed over 5 days at 20 degrees C by a unit volume of liquid during biological oxidation. It is expressed in mg/L.

Chlorination
The application of chlorine to water and wastewater generally for the purpose of disinfection.

Chlorine
The two forms are generally described as free and combined chlorine. Free chlorine is much more reactive than combined chlorine, and is a far more effective bactericide.

Clarification
The clarification process is essentially settling or *sedimentation*.

Chemical oxygen demand (COD)
A measure of the organic content of domestic and industrial wastewater. The COD test is used to measure the oxygen equivalent of organic material in wastewater that can be oxidised chemically. Expressed as mg/L.

Coagulation
The process whereby chemicals are added to water or wastewater resulting in the neutralization of charges allowing particles to come together.

Constructed wetland
A wetland that has been purpose built to achieve a set of design objectives. They may be in the form of an open surface wetland and a sub-surface or reed/gravel bed wetland.

Contamination
Effluent contamination refers to the corruption, either chemically or biologically, of effluent standards.

Disinfection
A process which destroys, inactivates or removes pathogenic (harmful) microorganisms.

De-nitrification
The process reducing nitrate or nitrite to nitrogen gas, in the absence of freely available oxygen.

Detention time
Also know as retention time and hydraulic retention time (HRT). This is the design time wastewater or effluent must undergo within a unit process to achieve a target standard of treatment.

Distributor
A device which distributes the tank effluent evenly over the surface of the filter media.

Dissolved oxygen (DO)
The factor that determines whether biological changes are being brought about by aerobic or by anaerobic organisms. Expressed as mg/L.

Dry weather flow (DWF)
The wastewater including ground water infiltration, flowing into a sewer in dry weather.

Escherichia coli (E coli)
Bacterial species present in polluted water and wastewater. Specified numbers in water legislation and standards.

Effluent
The water that discharges following a wastewater treatment process, usually to be irrigated.

Emergent plants
Plants that are attached to the substrate and whose leaves and stems either float or protrude above the water surface.

Equivalent person (EP)
The number of persons who could contribute the same quantity and/or quality of domestic wastewater as the establishment or industry being considered.

Eutrophication
Is defined as heavy organic growth pollution of a water body stimulated by the addition of inorganic nutrients.

Evapotranspiration trench of bed
A system designed to dispose of effluent by the combined mechanisms of evaporation and transpiration (ET).

Faecal Coliforms
Faecal coliforms are bacteria of the coliform type that can only exist within the gut of a warm blooded animal. It is for this reason that the presence of faecal coliforms is an indicator of sewage pollution. E.coli is a bacterium that comes from t*he faecal coliform group.*

Final effluent
The liquid finally discharged from a wastewater treatment plant.

Filter medium
The material, such as broken stone or sheets of plastic, with which a biological filter is filled.

Flocculation
This is a variety of mechanisms whereby small particles are grouped together or agglomerated into larger particles than can settle by gravity.

Greywater
Kitchen, laundry and bathroom waters. Also referred to as sullage.

Hydraulic conductivity
The rate at which soil or a substrate can transmit water or effluent. Also known as soil permeability (Kvalue).

Hydraulic Flow
The influent amounts of water into a treatment unit. It is usually expressed in litres per day (L/d).

Hydraulic loading rate
Influent discharge into a treatment unit.

Humus tank
A tank, through which biological filter effluent passes, to settle solids which should be removed periodically.

Infiltration (soil)
The process of water or effluent moving into the surface, sides or base of a trench or bed.

Infiltration or adsorption trench or bed
A system designed to dispose of effluent solely by infiltration or adsorption into the soil.

Influent
Refers to the wastewater flowing into a treatment unit.

Inorganic waste
Waste that is not from an organic living source and is generally not biodegradable. Examples include metals and plastics.

Irrigation
Refers to the surface distribution of effluent. Associated with irrigation is an appropriate irrigation area which is dependent on local evapotranspiration conditions.

Macrophyte
These are plants which are macroscopic or able to be seen by the naked eye. The term used to describe larger aquatic plants.

Monoculture
A system that is dominated by one plant species.

Medium
The stone aggregate or wood chip placed in effluent trenches and beds.

Nitrate – Nitrogen
Nitrate is a major nutrient. It is expressed in mg/L.

Nitrification
The bacterial conversion of ammonia to nitrate - nitrogen and nitrite – nitrogen. Nitrite is associated with a number of health risks.

Nutrients
Chemical elements that are essential for sustained plant or animal growth. The most important of these elements are nitrogen and phosphorus and many of their compound derivatives. Nutrients are prevalent in most wastewaters.

Oxidised nitrogen
Oxidised nitrogen represents the sum of all oxidized forms of nitrogen.

pH
A measure of hydrogen ion concentration in a solution, indicating the presence of acidic, neutral or alkaline conditions.

Precipitation
Chemical reaction causing substance in a solution to be deposited as a solid.

Preliminary treatment

Covers the pre-treatment processes of screening, grit removal and flow balancing.

Primary treatment
The processes that removes a substantial amount of suspended matter but little or no colloidal and dissolved matter.

Reed/gravel bed
Sub-surface wetlands designed such that the flow moves through a soil or gravel matrix which is planted with emergent macrophytes.

Retention period (otherwise known as detention period)
How long a liquid and solids are retained within the treatment unit. This may refer to an individual compartment (such as a septic tank) or a treatment unit as a whole.

Reuse or reclaimed
The beneficial reuse of effluent and sludge.

Rhizome
Any fleshy stem that grows horizontally in the ground and enables the plant to reproduce itself.

Secondary treatment
The treatment of primary effluent by biological aerobic processes to remove organic matter.

Sedimentation
The separation of suspended matter from wastewater by gravity. This is also known as clarification.

Sludge
The accumulated indigested waste at the bottom of an anaerobic unit. It is composed of both organic and inorganic components as well as water.

Substrate
The term for material that forms the bed of a wetland and provides the base for wetland planting.

Supernatant liquor
The layer of liquid overlaying the settled solids which have been separated from it.

Suspended solids (SS)
This is also known as non-filterable solids (NFS). Suspended solids are solids which are suspended in sewage or effluent. It is usually expressed in mg/L.

Tertiary treatment
Follows primary and secondary treatment and it is used to reduce BOD_5, suspended solids, bacteria and other pathogens and nutrients. This process is also referred to as advanced or polishing treatment. Tertiary treatment can include disinfection.

Total coliforms
Total number of bacteria or colonies of the coliform type present within an effluent or water sample. Expressed in bacteria or colonies per 100 millilitres (count/100ml).

Total Dissolved Solids (TDS)
Those solids that pass through a filter and then evaporated and dried.

Trade waste
The liquid discharge, with or without matters in suspension, resulting wholly or in part from any manufacturing process.

TWL
Top water level of a tank or chamber.

Water balance
Water volume changes in a wetland, ET trench or bed in response to variations in wastewater effluent discharges, rainfall, seepage and other hydrological factors.

Wastewater
The discharge from domestic and sanitary appliances within individual houses and communities. Also referred to as sewage.

Chapter 1 Thesis Introduction

1.1 Overview

In non-sewered urban and rural residential developments wastewater is usually treated and disposed of on-site.

In New Zealand about 20% of the population relies on on-site wastewater treatment and effluent disposal (AS/NZS 1547:2000; M for E, 2007).

The development of technical documents as AS/NZS 1547:2000 (Standard, 2000) and the Auckland Regional Council TP 58 (ARC, 1994) over the past decade has seen major advances in the design and installation of on-site wastewater systems. However, many issues still remain. Several studies carried out in recent years have revealed that a large number of on-site wastewater systems are not performing in a way that provides acceptable levels of treatment (M of E, 2007). The New Zealand Ministry for the Environment aims to further improve the performance and management of domestic on-site wastewater systems, to reduce risks to public health and the environment (M for E, 2007).

The set up of the On-site Effluent Treatment National Testing Program (OSET NTP) in Rotorua, New Zealand in mid-2000 is seen as a positive move to improve treatment performance (ON-SITE NewZ, 2009.)

Within Australia about 12% of the population relies on on-site wastewater systems. The high percentage of urbanisation generally means that reticulated sewerage can be provided for most developing urban areas. However, there are an increasing number of situations where there will continue to be a reliance on on-site systems. Such situations include rapidly developing area on the fringes of the large cities, rural residential developments, small rural communities and isolated residences (Geary and Gardner, 1996).

The US Environmental Protection Agency (USEPA) acknowledged for the first time in 1997 that on-site wastewater treatment systems were and would continue to be a permanent component of their wastewater infrastructure. The Agency recognised that on-site systems are an effective option for protecting public health and water quality, but only if they were properly designed, installed and managed. In the opinion of the writer, this key statement also applies to New Zealand and Australia. Otis (2005) goes one step further by concluding that we have the technologies to meet most treatment requirements, necessary to protect water quality. As a result of the writer's long involvement in this field he concurs with this claim.

Decentralised wastewater management (DWM) can be defined as the collection, treatment and disposal or reuse of wastewater from individual homes, clusters of homes, and isolated communities, at or near the source of generation. There are more than 60 million people in the USA served by DWM and on-site systems (Crites and Tchobanoglous, 1998). It should noted that the term DWM as used in Australia refers to individual on-site systems in addition to clusters of houses numbering from a few to several hundred (Tchobanoglous and Leverenz, 2008).

Typical situations where DWM and on-site systems should be considered or selected include:

1. Where the residential density is sparse and it is remote from a sewerage system
2. Where the operation of existing systems must be improved and the community cannot afford a conventional centralised system
3. Where water supplies are limited

   Where, due to environmental constraints, larger discharges of effluent should be avoided.

Typical treatment options for DWM and on-site systems, covered in this thesis, include:

1. Improved septic tanks
2. Up flow pebble filters
3. Constructed reed/gravel beds and open surface wetlands
4. Composting toilets
5. Greywater treatment options
6. Lime/seawater treatment
7. Options for handling fluctuating wastewater flows.

Typical effluent disposal options for DWM and on-site systems, covered in this thesis include:

1. Evapotranspiration (ET) trenches and beds
2. ET/infiltration trenches and beds
3. Sand mounds
4. Deep shafts
5. Vetiver grass plots
6. Kikuyu grass areas.

The technical literature has been reviewed for the following purposes to:

1. Identify earlier research work

2. Identify more recent research work
3. Be able to compare and confirm my research findings with that of others
4. Source aspects worthy of reflecting and for further research and development

This thesis is not complete without some discussion on the important need for DWM and on-site system operation and maintenance, collectively referred to as management, and public education. There is a wealth of information on how on-site treatment and effluent disposal systems can be managed appropriately and on the consequences of not doing so. The writer has noted, over the past 15 to 20 years that on-site systems are functioning better. Federal Government, State Government, Regional Councils and Local Councils in Australasia have some training and conducted public education programs. Australian Federal Government programs on water conservation and the Queensland *WATERWISE* program have enhanced on-site treatment and disposal public awareness and education.

Bylaws are also an effective way to increase awareness of onsite treatment and disposal issues. For example, the former Whangarei City Council in New Zealand introduced a “Septic Tank Bylaw”. This made ratepayers very aware of the need to install septic tanks of sufficient capacity, to desludge septic tanks on a periodic basis, and to install appropriate effluent systems. It has well proven that solids from septic tanks can carry over into the disposal trenches and beds. The cost of replacing disposal systems is high. The writer assisted with the drafting of this “Septic Tank Bylaw” and reported (Simpson, 1974) that there was a need to instigate on-site training programs.

The former Caboolture Shire Council introduced Household Sewage Treatment Plant (HSTP) quarterly performance monitoring regulations, which made residents more aware of on-site management.

In summary, the standard of management of on-site systems is improving in New Zealand and Australia.

1.2 Aims and Objectives

From the early 1970s the writer perceived the need for more in-depth research within the field of water quality control which included on-site wastewater treatment and effluent disposal. The writer became involved in Local, Regional and State government programs, with the assistance of New Zealand and Australian government funding. The research programs consisted of work arising from the writer’s own consulting projects as well as joint projects with other researchers. These projects were in different forms and subject matters.

The overall goal of this thesis was to integrate this diverse and fragmented research and project work, over a 36 year period, into this Engineering Doctorate, presented as Portfolios 1 to 8.

The primary aim of this thesis is to document applied research projects the writer has undertaken since 1974, which focus on on-site wastewater treatment and effluent disposal in New Zealand and Australia. These projects are briefly described in part 1.3 Structure of Thesis.

A secondary aim is to incorporate past environmental engineering and scientific experience in the planning, design, monitoring, performance evaluation and the management of these projects. Another secondary aim is to subject each research project and case study to some reflective thinking and assess opportunities for future extended research and development.

The main aim of each Portfolio has been achieved by objectives, which are listed in the introduction of each portfolio.

1.3 Structure of Thesis

This thesis is organised in chapters and portfolios. Chapters include the following:

Chapter 1 outlines the writers applied research experience and original work and innovative projects undertaken, related to on-site wastewater treatment and effluent disposal. It also includes a list of technical papers, research projects and case study projects.

The background, aims, objectives, and thesis structure are covered in Chapter 2. Conclusions drawn from each Portfolio are given in Chapter 3. The portfolios that follow on from these Chapters described a range of research projects, surveys and case studies.

Wastewater treatment in coastal and lakeside areas and some smaller institutions are often faced with the problem of treating seasonal and fluctuating flows. The unique problems of fluctuating loads and viable treatment options are outlined in Portfolio 1. Many of the specific methods on treatment are further covered in the portfolios that constitute the bulk of this Engineering Doctorate dissertation.

Portfolio 2 introduces the mechanisms of effluent infiltration and the determination of soil permeability, and it includes important aspects associated with the investigation, design and management of effluent trenches, beds and shafts.

Portfolio 3 examines the design rationalisation of reed/pebble beds, nitrification and de-nitrification, the performance of smaller scale wetlands and treating greywater by reed/gravel beds.

The development of a Clivus type domestic compost convertor in Kerikeri, New Zealand is reported in Portfolio 4. This also includes a case study of a VIP toilet and options for greywater treatment and disposal.

An Australian wide postal survey of on-site effluent disposal systems is reported in Portfolio 5. This portfolio also includes the performance of upflow pebble filters and it offers options for septic tanks and optional effluent disposal systems, including a case study in Russell Island, Moreton Bay.

Portfolio 6 examines the design and use of evapotranspiration effluent disposal trenches and the selection of appropriate vegetation in Part A. The potential of nutrient uptake of vetiver grass and kikuyu grass is determined in Part B.

Portfolio 7 reports on a North Island of New Zealand survey of deep effluent disposal shafts. It also covers case studies of sand mounds in NSW and Caboolture, Queensland. The method of treating domestic wastewater and process waters by using seawater and solar salts in conjunction with burnt lime is covered in Portfolio 8. This Portfolio also covers experimental jar tests and smaller scale pilot plants trials, to achieve a tertiary standard of effluent and well stabilised sludge.

The typical structure of each Portfolio is:

1. Introduction – description of the topic to be covered in each Portfolio.
2. List of technical reports and publications, appropriate to each Portfolio.
3. Description of applied research project and case study, methodology, results and discussion that the writer has undertaken and at times with associated researchers.
4. Reflective remarks – to include a literature review of independent work by others, research confirmation of his findings, the need for further research and in some cases recommendations.
5. Conclusions.
6. References – for each Chapter and Portfolio.

The thesis structure flow diagram in Figure 1 shows how the Chapters and Portfolios are interlinked.

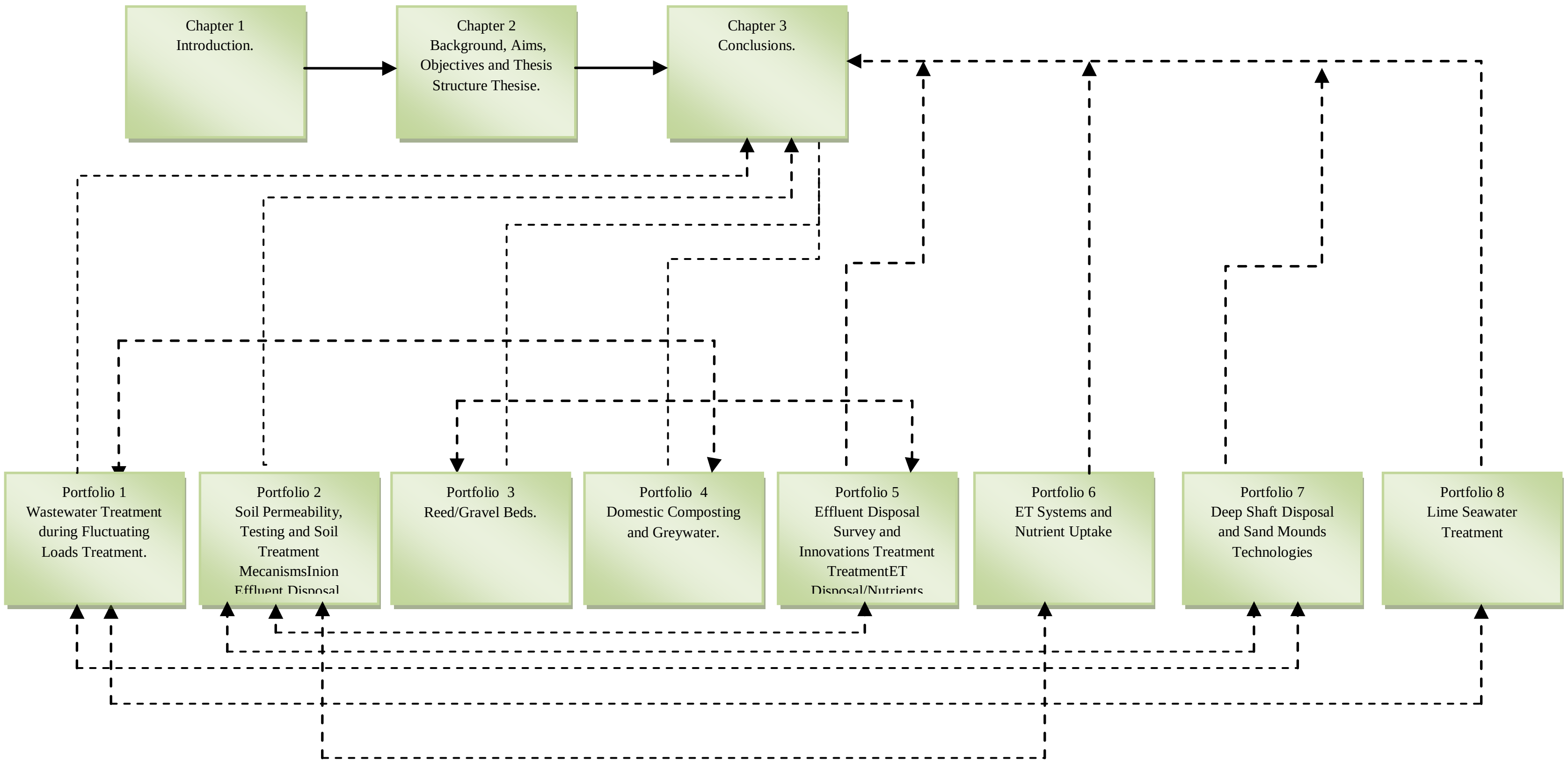

Figure 1: Structure Flow Diagram

Chapter 2 Background to Thesis

2.1 Types of Technological Contributions

Some technologies for on-site wastewater treatment and disposal have been written up in more than one portfolio. For example, reed/gravel beds are a viable alternative for handling variable loads, as written up in Portfolio 1 and as covered in detail in Portfolio 3.

Another example is deep shaft disposal, which has been covered in detail in Portfolio 7 and to a lesser degree as an alternative for handling fluctuating flows, in Portfolio 1.

Types of Applied Research and Innovative Work

The writer has been engaged in a range of applied research, model designs and innovative work in Portfolios 1 to 8 as reported in this thesis. A brief description of each experimental type and some examples are given.

Conventional Applied Research

This involves the formulation of a hypothesis, a comprehensive experimental program, an analysis of the results, and making conclusions and recommendations as reported by Kumar (2011). An example is the lime/seawater/seawater substitutes study in Portfolio 8. The work of Simpson (1986) has been partially verified in the joint work with the Queensland University of Technology (Shanableh et al 1995) and recommendations for futher research were been made.

Cases Studies

This involves reporting on the design and performance of case studies, as reported by Eisenhardt (1989). An example is the on-site evaluation of a large parcel of land at the Whangarei Heads, New Zealand. This utilised soil permeability test methods covered in Portfolio 2 and evapotranspiration/infiltration system design reported in Portfolio 6 Part (A).

Another case study was sand mound performance evaluation in Caboolture, reported in Portfolio 7. A further case study was on the Tiaro Motel, Queensland wastewater system which involved on-going nitrification and de-nitrification in the effluent disposal system, reported in Portfolio 2.

Development Project

The example is the experimental compost convertor in Kerikeri, New Zealand as reported in Portfolio 4. Some experimental work was undertaken. The development project was designed on best available information at the time. The results and performance were analysed and minor changes made. The system was commissioned in 1976 and it is still operational today.

Development of an Innovative and Specific Purpose System

This type of work has been undertaken to suit specific site, soil and climatic conditions and constraints, similar to a case study of some operations of the international consultancy Arup, as reported by Salter (2002). For example, evapotranspiration/ infiltration effluent trenching in situations where the upper soils have a limited permeability but the climate enhances effluent and moist losses. This is written up in Portfolio 6 Part (A).

Postal and Site Surveys

A postal survey was conducted to gather performance data which has been accompanied by a site survey of effluent disposal systems. A study of deep shaft disposal systems in the North Island of New Zealand was conducted using this method. This has been reported in Portfolio 7.

A postal survey of effluent disposal systems in Australia was undertaken and it has been reported in Portfolio 3.

Development of a Simple Innovative Treatment System

To improve the quality standard of greywater a simple and robust treatment system has been developed. This is a wood chip/sand filter for achieving a secondary standard of treatment. An additional woodchip filter is to be developed to achieve de-nitrification in greywater. This is reported in Portfolio 4.

Comparative Study of Alternative Assimilation Methods

A comparative study of three alternative nutrient assimilation methods and models in Stanmore, South East Queensland has questioned current technology. This has been reported in Portfolio 6 Part (B).

Area Specific Alternative Design

An example of an area specific alternative design being more appropriate to site, soil and climatic conditions was undertaken for Queensland and Northern New South Wales in Portfolio 7. This project assessed data, compiled design parameters, and reviewed climatic and soil factors. This was reported in Portfolio 7.

Design Methodology

A methodology for the design of topsoil depths for spray irrigation systems using secondary quality effluent was developed. The procedure is covered in Part (A) of Portfolio 6.

2.2 Timeline of Technical Contributions

This thesis is the consolidation of technical experience and practical research in the specific field of wastewater treatment and effluent disposal gained by the writer over a period of 30+ years. The full range of projects and relevant research that underpin this thesis are summarised in the timeline shown in Table 1.

Table 1: Chronological Timeline Listing of Simpson Applied Research Projects

Research Projects	Period	Location and Presentation
Deep Shaft Effluent Disposal	1976/77	North Island of NZ - Survey of existing systems and report. Portfolio 7.
Ventilated Improved Privy (VIP)	1976/77	Simpson Family Home in Northland, NZ. Portfolio 4.
Upflow Pebble Filters	1977	Design of a system for church camp, South Auckland, NZ. Portfolio 5.
Greywater systems	NZ - 1977/78 SE Qld - 2001/08	Design of associated greywater systems for composting toilets, NZ and Qld. Alternative treatment and disposal systems. Portfolio 4.
Clivus Type Domestic Composting Toilet	1976/78	Experimental system Kerikeri, NZ Reported to *On-site Alternative Sewerage Seminar*, University of Auckland, 1981. Portfolio 4.
Soil Infiltration Studies	1977/78	Experimental work on infiltration methods for trenches, beds and deep shafts, Northland, NZ Portfolio 2.
Evapotranspiration Studies and Selected Plant Species	1977/78	Northland, NZ. Assisted with compilation of list of suitable plant and shrub species for evapotranspiration. Reported to *On-site Alternative Sewerage Seminar*, University of Auckland, 1981 Portfolio 6A.
Alternative or Optional Septic Tanks	1978/79	Optional configurations, North Island, NZ. Portfolio 5.
Bark Filters	1982/83	Design and limited performance of system for light commercial and butchery wastewater, North Shore, Auckland, NZ. Portfolio 4.

Lime/Seawater Wastewater Treatment	1985/87	Applied Research Report for Dip Sc, Auckland Technical Institute. Portfolio 8.
Small Open Surface Wetlands	1993/95	Artificial Wetland Research and Advisory Committee, Qld. Pilot plant wetland system at Wamuran, Qld. Portfolio 3.
Greywater Treatment by Wetlands	1993/95	As above - Wamuran, Qld. Portfolio 3.
Reed/Pebble Beds Wetland	1993/95	As above – Wamuran, Qld
Reed/Pebble Bed – Nitrification - denitrification	1993/2009	SE Queensland. Portfolio 3.
Alternative Effluent Disposal Systems	NZ -1976/78 SE Qld - 2007/08	NZ and Qld. Portfolio 5.
Australian Postal Survey of Effluent Disposal	2001	Refereed paper presented at *ON-SITE 01* National Conference, Armidale, NSW. Portfolio 5.
Raised Effluent Mounds	NZ – 1976/77 Caboolture – 2009/2010	Bay of Islands, NZ and Case Study in Caboolture, Qld. Portfolio 7.
Kikuyu Grass – Nutrient Uptake	2003 to present	Design, based on first principles and soil science, of nutrient assimilation areas for subdivisions in SEQ Water catchments**.** Portfolio 6B.
Vetiver Grass Overland Flow system – Pollutant and Nutrient Uptake	2007 to present	Watson Park Convention Centre and School, Dakabin, Pine Rivers. Vetiver grass rafts and overland flow plot, design and limited monitoring. Portfolio 6B.

2.3 Summary of Technical Contributions and Innovations

The writer's original contributions and innovative work, project uniqueness, and the benefits and values of each project, are listed in Table 2.

Table 2: Original and Innovative Work by Simpson

Description and Location	Period	Uniqueness	Benefits and Values
Development of a Clivus type compost converter, Kerikeri, NZ	1975/76	First modified Clivus system in Australasia. First installed in December 1976 and still operating without problems. 6.0 m high exhaust flue, in black PVC, draws gases up.	No need for an exhaust fan. About 40% savings in household water. Composts household food wastes, as well as blackwater. Reusable compost end product. No problems with flies and odours.
Deep shaft effluent disposal, Whangarei, NZ	1977	In-ground biological treatment takes place within the deep shafts. When the upper soils are relatively impermeable, deeper shafts allow access to more permeable stratum.	Considerable land area saving. Alternative dosing and resting, of two deep shafts, promotes system sustainability. De-nitrification potentially takes place.
Development of a soil conditioner from waste products, Whangarei, NZ	1977	Utilisation of lime kiln dust (waste product) with sewage sludge (waste product). NZ and USA patents obtained and product widely marketed within agricultural and horticultural sectors.	Low cost soil conditioner produced. Well stabilised end product. Economic method. No water quality or other environmental problems.
Narrow trenching for greywater disposal	1977/79	Shallow trench over the depth of the effective soil mantle. Narrow trench width to minimise site disturbance	Reduces trenching costs. Minimises site and existing vegetation disturbance. Reduces risk of slope instability, on steeper sites.
Hydraulic offloading station, Springs Flat, Whangarei, NZ	1979/80	First of this concept in Australasia. A suburban wastewater treatment and effluent disposal system.	Reduces the wastewater treatment load on a main central wastewater treatment plant. Buffers peak hydraulic and organic loads on main central plant.
Pre-treatment wood chip filter, Auckland, NZ	1982	Simple, economic and natural treatment system.	Simple gravity system. Effective pre-treatment system if wastewater contains toxic materials and grease.

Effluent treatment by upflow pebble filter, Church Camp, Beachlands, Auckland, NZ	1983	Unique upflow action filter. The first of this size in NZ.	Capable of handling increasing and fluctuating loads. Readily expanded. Simple action and economic system.
Lime /seawater substitutes, Auckland, NZ	1985/86	Further developed from basic jar testing by NZ engineer, reported in 1970. Pilot plant testing and scaled up for a population of 100.	Economic method of treatment, since using waste products. Results in a well disinfected effluent. Produces a reusable effluent and a well stabilised sludge.
Leachate treatment by recirculation, Auckland, NZ	1988/90	Simple method of leachate treatment. First large scale leachate recirculation system in the Pacific rim.	More economic process than other options (lagoons, chemical and biological). No extra land required. Effectively reduces BOD_5, COD, suspended solids, sulphates, fatty acids, heavy metals and raises pH.
Open surface wetland polishing of oxidation lagoon effluents, Cherbourg, Queensland	1994	Facilitates as an effective natural filter for algae laden oxidation lagoon effluent.	Simple gravity system. Bird and wildlife habitat. Produces effluent suitable for pasture irrigation. Options treatment methods known to be expensive and costly.
Agricultural drainage water treatment by off stream wetlands, Burdekin, North Queensland	1994/95	Off tail drain treatment method. First of this type in Queensland.	Treatment method, by the reduction of nutrients, turbidity, sediments and pesticides, within sodic soils.
Use of water lettuce for phosphorus removal in domestic wetlands, Wamuran, Queensland	1993/95	Unknown potential for reducing phosphorus in effluent, until trailed.	Very high phosphorus reduction. Readily composted with about 80% volume reduction.
Treatment of gold mine leachate using anoxic lime beds and Typha spp. wetland cells, Horne Island Torres Strait	1994/95	Leachate has pH as low as 3.0, high sulphates and heavy metal contents.	Treated water enters waters of the Great Barrier Reef Marine Park.

Multi- purpose wetland concept – Innisfail, North Queensland	2003	Multi-purpose wetland concept.	Effluent disinfection. Eco – tourism potential. Bird and wildlife habitat. Effluent polishing and nutrient reduction. Passive recreational park. Educational and research facility. Landscape appeal. Low operating costs. Effluent reuse potential.
Overland flow of effluent on vetiver grass plot, Dakabin, Queensland	2006/07	Understood to be second such system in Australia. Simple but effective process for effluent polishing.	Reductions in N and P in oxidation lagoons. Enables reuse of effluent by the irrigation of pasture and trees.
Ammonia reduction by aeration and large scale reed/gravel beds, Lorong Halus, Singapore	2006	A solution to the challenge of treating high nitrogen and Ammonia N leachate.	Final effluent disposed into large open surface wetlands, as a passive recreational park and bird/wildlife habitat. Alternative leachate treatment is very expensive.
Integrated Effluent treatment and management, St Michaels College, Farm, Church and Community, Caboolture	2007	Understood to be the first and only integrated domestic and agricultural waste and reuse system in Australasia.	Obtained Moreton Bay Regional Council and Qld EPA approval. Some savings in fertilizers for dairy cow and beef cattle grazing.
Biological nutrient removal activated sludge plant, followed by surface flow and reed/gravel wetlands and overland flow, Chicken processing, Byron Bay, NSW	2002	Dual wetland treatment and overland flow effluent polishing. Design flow 300m^3 /day.	Protection of SEPP 14 natural wetlands in Byron Bay, NSW. De-nitrification by reed/gravel beds. Enables 60% effluent reuse for washing down.
Russell Island shopping centre, Queensland	2010	Specifically designed deep trenching to suit site constraints.	2.0 m deep trenching containing a wood chip zone for nitrification. Other effluent disposal options not suitable (spray irrigation and shallow trenching).

2.4 Projects Completed During 1975-1980

Following some 13 years experience in civil and municipal engineering, the writer undertook post graduate studies in Public Health Engineering, at the School of Engineering, University of Auckland. This was a relatively new course which was based on the Diploma in Public Health Engineering (DipPHE) course at the University on Newcastle upon Tyne, UK.

This followed the commencement of the global environmental movement in the early 1970s. The need to focus on wastewater treatment, effluent disposal and water pollution control was recognised. There was considerable potential for applied research in the development of treatment and effluent disposal technologies. During post graduate Diploma in Public Health Engineering studies, at the University of Auckland in 1974, the writer assisted with a national survey of septic tank systems in New Zealand, initiated by Ian Gunn the course supervisor, and undertook a dissertation *"Wastewater Disposal in Smaller Coastal Communities".* The dissertation was essentially a comprehensive literature review of the unique problem of widely fluctuating wastewater loads from coastal and lakeside communities, camping grounds and resort areas in Portfolio 1. Some further innovations for handling fluctuating loads have been recorded in Portfolio 1.

Conventional biological treatment systems were often not capable of handling fluctuating hydraulic and organic loads. The review covered individual households, groups of houses, camping grounds, resort and small communities. The review also looked at improvements to septic tanks, alternative effluent disposal systems (Portfolio 5), upflow pebble filters (Portfolio 5), stabilisation lagoons, rotating biological discs (Portfolio 1) and emerging technologies at the time.

A survey of existing deep shafts for effluent disposal was undertaken over the upper North Island of New Zealand (Portfolio 7).

The writer reviewed the lime/seawater treatment technique (Mawson, 1970) in the mid-1970 and considered that it had much potential for coastal areas, in a range of climatic zones (Portfolio 8).

A need arose in 1976 to reuse the waste products of kiln lime dust and stabilised wastewater sludge in Whangarei, New Zealand. The writer was commissioned to undertake a feasibility study for *NATUMIX* to examine the environmental aspects of mixing sewage sludge with kiln dust for developing a general purpose fertilizer and soil conditioner. As a result of this study, a soil conditioner was developed for use on dairy farms and orchards in New Zealand. This product gained a USA patent. It had the ability to renovate very poor and acidic soils to the extent they had healthy worm populations and they became very productive, in terms of butter fat and fruit yield.

An experimental *Clivus* Type domestic compost converter was designed, constructed and monitored in Kerikeri, Northland, New Zealand (Portfolio 4). This unit accepts blackwater and kitchen wastes. This system has operated without major problems since being commissioned in December 1976. The design of many associated greywater treatment and disposal systems throughout New Zealand consequently took place (Portfolio 4).

The opportunity arose to install a ventilated pit privy, as an interim measure, during the construction of the Simpson family beach home at Teal Bay in Northland, New Zealand in 1976. The performance of this system was observed during variable loads over about two years (Portfolio 4).

Due to failed effluent trenches in Northland, associated mainly with lower permeability soils, experimental work on evapotranspiration (ET) systems was undertaken in conjunction with Ian W Gunn, Senior Lecturer in Civil Engineering, University of Auckland and Alan Fielding, Environmental Horticultural Consultant and Landscape Architect, during 1976 – 1979. This included assisting with the compilation of selected plants, shrubs and trees for planting within ET systems in Northland, Auckland and Bay of Plenty areas (Portfolio 6).

The writer undertook the process and hydraulic design of a hydraulic off – loading station in 1979/80, a new concept in the form of a fringe suburban wastewater treatment plant, which was to relieve the hydraulic and organic loading on the main central treatment plant (45,000 EP) in Whangarei, New Zealand. This concept was useful for handling fluctuating loads so it has been included in Portfolio 1.

2.5 Projects Completed During 1980-1990

Following correspondence with Keith Mawson, in August 1983 and August 1984, who undertook the initial lime/seawater experimental work in Wellington New Zealand**,** the writer undertook a research project "Chemical Wastewater Treatment" for a Diploma in Science at the Auckland Technical Institute, over the period 1985-1986. This project covered the use of lime and solar salt brines with seawater and magnesium salts, for treating municipal wastewater and a range of process waters. It involved a jar testing program and small scale batch and continuous pilot plant trials (Portfolio 8).

As part of a Masters of Science (Environmental Science and Geography) at the University of Auckland, a methodology was developed for the environmental assessment of wastewater management, based on a case study of a gold mine in Waihi, New Zealand. The Masters Degree thesis "Leachate Recirculation as a Refuse Landfill Management Option" focused on water quality control, leachate treatment, landfill gas control, extensive monitoring and the application of environmental assessment methodologies. The Masters degree included study projects in sand mining, changes in coastal geomorphology, foreshore biological surveys, landfill gas recovery and gas utilisation options. The writer was part of a small team that was responsible for pioneering and developing landfill gas control, gas collection and flaring off. The gas technology work involved regular risk assessments. His involvement in leachate management was ahead of any such work within the Pacific basin.

2.6 Projects Completed During 1990-date

The writer moved to Queensland in 1990 to work for the Department of Primary Industries (Water Resources) to review the *"Guidelines for Planning and Design of Sewerage Schemes"* for Local Government. After the completion of the "Sewerage Guidelines" it was decided there was a need to develop alternative wastewater treatment technologies.

The writer set up an Artificial Wetlands Research and Advisory Committee to develop this technology within the range of climatic conditions in Queensland. Ten pilot scale wetlands were established at my initiation, to treat primary municipal effluent. These pilot wetlands were located in different climatic areas and they extended from Douglas Shire in North Queensland, to Blackall in Western Queensland to Goondiwindi in SW Queensland. Cooperative research was undertaken with Queensland University of Technology (QUT), Griffith University and University of Queensland (UQ). *"Guidelines for Using Free Water Surface Constructed Wetlands to Treat Municipal Sewage*" were published in 2000, as the joint effort of the Qld Department of Primary Industries (DNR), universities, Environmental Protection Agency (EPA), consultants and local authorities (Portfolio 3).

The writer was appointed as Principal Investigator of a joint Qld Department of Primary Industries (DPI) / Land and Water Resources Research and Development Corporation (LWRRDC) research project *"Nutrient Control in Irrigation Drainage Systems using Artificial Wetlands"* which was undertaken in the tail-waters of cane fields of the Burdekin, commencing in 1994. Sugar cane production liberates fertilizer, insecticides and pesticides, which impact on water quality. The writer produced two milestone reports on the progress and findings of the experimental wetlands.

The writer undertook joint research with the Queensland University of Technology (QUT) on the lime/seawater and solar salt brine treatment of municipal effluent, as sponsored by the Queensland Foundation of Local Government Engineering (Portfolio 8).

Over the past decade wetland technology has extended into the enhancement of stormwater runoff. The writer assisted in the development of wetland technology options to handle the widely variable runoff flows by the use of bypass channels, ephemeral zones, deep zone and macrophyte filters, in association with Associate Professor Margaret Greenway, Griffith University and others.

The writer has a particular interest in the use of reed/gravel beds or sub-surface wetlands for treating domestic effluent and achieving nitrification and de-nitrification, in Queensland conditions. These high rate units have a definite application as decentralised and on-site systems (Portfolio 3).

Raised sand mounds for effluent treatment have not been used on a wide spread basis, as a conventional system in Queensland. The writer is interested in confirming their performance, against AS/NZS 1547:2000 prescriptions, in sub - tropical conditions (Portfolio 7). The monitoring of a sand mound has been undertaken in the Caboolture area, as part of this engineering doctorate.

2.7 Grant and Publications Record

The following is a list of grants, research papers and technical papers by the writer and associated authors:

(1973), Awarded NZ Department of Health bursary for Postgraduate Diploma in Public Health Engineering Studies, School of Engineering, University of Auckland

Simpson, J S (1974), "Fluctuating Loads in Sewage Treatment in Smaller Coastal Communities" Postgraduate Diploma in Public Health Engineering Dissertation, University of Auckland

John S Simpson and Associates (1977), "Feasibility Report on Environmental Aspects of Mixing Digested Sewage Sludge with Kiln Dust" *Natumix Fertilizers Ltd*, Hamilton, NZ

Simpson, J S (1976), Report on a Survey of Deep Shaft Effluent Disposal Systems in the North Island, New Zealand, *Fenwick Drilling Contractors*

Simpson, J S (1980), Process and Hydraulic Design of the Springs Flat Hydraulic Off-loading Station, Whangarei City Council.

Simpson, J S (1981), "Clivus Composting Systems in Northland, NZ" and Evapotranspiration – Infiltration Disposal Systems in Northland, NZ"

Case studies presented to *Alternative Sewerage Seminar, School of Engineering, University of Auckla*nd, (1981)

Simpson, J S (1984), "Experimental Work with Polyaluminium Chloride and Magnesium Salts for Waste Treatment" *NZ Water Supply and Disposal* Association, Annual Conference

Simpson, J S (1985-1986), "Chemical Treatment of Wastewater" Diploma in Science Research Project, Auckland Technical Institute, NZ.

Simpson, J S (1986), "The Evaluation of Sites and Alternatives for On-site Effluent Disposal" *New Zealand Journal of Environmental Health*

Brokenshire, C, Rogers, D A and Simpson, J S (1989), "New Zealand's First Landfill Gas Control System" and "Chemistry and the Assessment of Landfill Gas Problems – the Necal/ARC Experience" presented to the *Inaugural Annual Conference of the Waste Management Institute of NZ*, Wellington, November

"Gas Recovery and the Determination of Utilisation Options – Greenmount Refuse Landfill, Auckland", Research Paper University of Auckland, (1988)

"Changes in Coastal Geomorphology in Shoal Bay, Auckland" Research Paper, University of Auckland, (1988)

"Environmental Impact Assessment Methodology for Waste Management" Research Paper University of Auckland, (1989-1990)

Simpson, J S (1988-1990), "Leachate Recirculation as a Landfill Management Option*"* M Sc (Hons 2) (Env and Geo) Thesis, University of Auckland

(1989-1990), Awarded $7,400 grant by NZ Ministry for the Environment for joint research with DSIR on waste stabilisation, rainfall infiltration and gas yield using test cells in refuse, Auckland.

(1990), Awarded $10,000 grant by NZ Ministry of Energy for joint research with DSIR on landfill gas yield and utilisation potential, Auckland.

Loos, E T and Simpson, J S (1991), "The Review of the Queensland Sewerage Guidelines" *Local Government Engineers Association of Queensland, 42nd Annual Conference*, Bundaberg, August

Simpson, J S and Beavers, P (1992), "Use of Wetlands for Small Communities – Strategies for Research and Development" presented at *International Specialist Conference on Wetland Systems in Water Pollution Control*, University of NSW, Sydney, Nov/Dec

Simpson, J S (1992), *"*The Use of Wetlands for Sewage Treatment and Disposal" presented *at Gold coast – Albert Sewage Disposal Alliance Seminar*, May

Simpson, J S and Anderson, R (1992), "Public Participation in Effluent Re-use and Wetlands – A Queensland Experience*" AWWA Regional Conference*, Toowoomba, October

Simpson, J S and Gibson, H E (1992), "Wetlands and their Application for Sewage Treatment in Queensland, *43rd Annual Conference*, *Local Government Engineers Association of Queensland,* Ipswich, October

Burgess, M D and Simpson, J S (1993), "The Potential for Low Technology, High Performance Wastewater Treatment Plant for Small Communities*" AWWA*, *15th Federal Convention*, April

Simpson, J S (1993), "Integrating Freshwater Aquaculture with Wastewater Treatment using Wetlands" *Freshwater Aquaculture Association AGM,* University of Queensland, April

Burgess, M D and Simpson, J S (1993), "The Potential for a Low Technology, High Performance Wastewater Treatment Plant for Smaller Coastal Communities" *First*

Annual Conference, The Institute of Municipal Engineering Australia, Queensland Branch, Goondiwindi, Queensland

Simpson, J S (1993), "Innovative and Appropriate Engineering for Wastewater Treatment – A Case Study" *Water Industry Workshop,* Charters Towers, August

Simpson, J S (1993), "Progress on the Wetlands for Sewage Treatment Research Project" *First Annual Conference of Institute of Municipal Engineering Australia, Queensland Division,* Goondiwindi, September

Simpson, J S and Gibson, H E (1993), "Solving Environmental Problems by Using Artificial Wetlands"

(1992), Awarded funding for visits to wetland projects and discussions with researchers in Albury, Melbourne and Adelaide, sponsored by Land and Water Resources and Development Corporation, Canberra, December. Report "Wetlands for Water Quality Control and Sewage Treatment" undertaken in January 1993.

Simpson, J S (1994), "Appropriate Wastewater Treatment – Case Studies Using Wetlands" *Conference on Technology Transfer in Remote Communities,* Murdoch University, April

Simpson, J S (1994), "Current Directions in Artificial Wetland Research in Queensland" presented to *Workshop on Practical Aspects of Constructed Wetlands,* Wallsend, NSW, June

Principal Investigator (1994-1995), "Nutrient Control in Irrigation Drainage using Artificial Wetlands" *Joint LWRRDC/Qld DPI Research Project, Burdekin, Nth Qld:* Milestone Report No 1 – 31 July 1994, Milestone Report No 2 – 31 July 1995

Simpson, J S (1995), "Current Sewage Treatment Practices in Queensland" and "Results of Artificial Wetland Trials" presented to *Effluent Re-use and Alternative Treatment Seminar,* Townsville, May

Simpson, J S and Woolley, A (1995), "The Application of Artificial Wetlands for Treating Municipal Effluent in Tropical and Sub-tropical Climates" *National Conference on Wetlands for Water Quality Control,* Townsville, September

Simpson, J S and Gibson, H (1995), Poster Paper – The Burdekin Experimental Artificial Wetland for Agricultural Runoff Treatment – Engineering Aspects, presented at the Burdekin Agricultural College, Ayr and the *National Conference on Wetlands for Water Quality Control,* Townsville, September

(1995-1996), Awarded $8,000 Fellowship by Queensland Foundation for Local Government Engineering for research project "Enhancing the Quality of Municipal Wastewater Effluent by using Salt Waste Brine and Lime", jointly with QUT. Research report completed.

Greenway, M and Simpson, J S (1996), "Artificial Wetlands for Wastewater Treatment, Water Re-use and Wildlife in Queensland, Australia" presented to *Second International Symposium on Wastewater Reclamation and Re-use, IAWQ*, Crete, October – published in *Wat. Sci. Tech, 33: 221-229.*

Simpson, J S and Woolley, A (1996), "Wetlands for Effluent in the Tropics and Sub-tropics" *Engineering Edge*, February

Simpson, J S et al (1997), Willawong Solid Waste Landfill Remediation Preliminary Wetland Process Design" *Stormwater Industry Association Conference*, Brisbane

Simpson, J S (1997), "Artificial Wetlands as an Effluent Treatment Option for Small Communities" *Institute of Municipal Engineering Australia, Queensland Division, Northern Region – 100th Conference*, Winton, May

Simpson, J S (1999), "Water Quality Management in Local Government Activities" presented to *Institute of Municipal Engineering Australia - South West Branch of Queensland Division*, Esk, May

Simpson, J S and Anderson, R (2001), "Constructed Wetland Planning, Design and Management" presented at *Wetlands Seminar, Maroochy Shire Council*, August

Simpson, J S, Anderson, R and Herring, A (2004), "Generic Management Plan for Waterways within Pine Rivers Shire Council" presented at *AWA Regional Conference,* Fraser Island, November

Simpson, J S and Anderson, R (2007), "Landscaping, Educational and Community Aspects of Stormwater Wetland Systems in South East Queensland" presented to *Stormwater Industry Association of Qld Inc, Regional Conference on Stormwater Management.*

Chapter 3 Thesis Conclusions

This Chapter covers the writer's engineering philosophises, broad conclusions regarding on-site wastewater and effluent disposal technology based on the writer's project and research experience, and broad conclusions of the eight Portfolios. Recommendations have been made in some instances on the need for further research and development.

3.1 *Engineering Philosophies*

This thesis is for the award of an Engineering Doctorate rather than a PhD so a little personal history and philosophising is considered to be appropriate. The writer did well at secondary school in biological sciences, physics, geography, geometry and technical drawing. The writer perused a five year cadetship in civil and structural drafting and design since it was linked into the provision of infrastructures and essential services. Having an inquiring mind he often questioned the derivation of formulae and design approaches. This was not the accepted thing to do in the 1960s. On reflection, there was value in doing so since there were very few engineering standards and codes of practice at the time.

This Engineering Doctorate is a compilation of many years of practical experience and research in civil engineering, environmental engineering and environmental science which has laid the foundation and provided the stimulus to undertake this thesis. It has also been a classic opportunity to reflect on the writer's past working life and its results and achievements.

The writer's philosophies pertaining to wastewater treatment and effluent disposal include the following:

1. To focus on natural systems which have required a particular ecological understanding
2. To employ innovative techniques to develop treatment and disposal methods that have been more economical than conventional unit processes, the writer's key focus. This has been the writer's opportunity to develop ideas "outside the square" to suit extra ordinary circumstances.
3. To develop treatment processes which are more easily operated when compared with conventional processes
4. To provide an acceptable level of service (degree of treatment and method of disposal) by the governing authority and the general public
5. To provide technology that is appropriate to the financial and operational resources available

In the writer's opinion, simple, economical and viable systems have often been overlooked and replaced by alternatives which have been less economical and required more design and operational input.

This thesis has been based partly on personal work experience with not necessarily much engineering and scientific data. The writer's earlier work in New Zealand yielded useful results but these data have been retained by local and regional authorities, most of which have long been amalgamated. Further data has become unavailable as a result of the writer's move to Australia in 1990. Some of the projects were undertaken at a time when there was less emphasis on monitoring and the recording of results, when compared with more recent practices.

3.2 *On-site or Decentralised Wastewater Technology Conclusions*

The following key factors of on-site technology are briefly discussed and some conclusions made. Some but not all of these key factors are discussed in the portfolios.

State of the Art of On-Site Technologies

The USEPA concluded that on-site systems are an effective option for protecting public health and water quality, but only if they are properly designed, installed and managed (Otis, 2005). The New Zealand Ministry of the Environment aims to improve the management and environmental performance of domestic on-site wastewater systems to reduce risks to public health and the environment (M for the Env, 2007).

It has been concluded by Tchobanoglous and Leverenz (2008) that on-site systems will continue to be used to protect public health and the environment for a large segment of the population.

On-site technologies in New Zealand are similar to that in Australia (Robyn Floyd, Auckland Regional Council, pers. comm. April 2009). In a broad sense this may be correct but in the writer's experience New Zealand tends to include pressure compensating drip irrigation, textile filters and deep shaft disposal in the North Island. Droplet type spray irrigation of effluent is popular in Queensland but this technology is not favoured in New Zealand. In the writer's experience, this is due to the less favourable climatic conditions and the public health risk associated with aerosols.

Tchobanoglous and Leverenz (2008) have predicated that with effective performance standards, application criteria and management strategies on-site systems will achieve equal status with centralised wastewater treatment. The writer has concluded that this prediction is worthwhile but there needs to be more effort enforcing performance standards and public education.

Need for Improved Operation and Maintenance

Although most of the treatment units used in on-site systems in the past required only periodic maintenance, unfortunately they rarely received any. On-site systems must overcome the poor performance stigma of the past (Tchobanoglous and Leverenz,

2008) hence, there is a definite need for the improved management of on-site systems.

Ongoing Need for More Public Education

A common reason for the loss of performance of on-site systems is the lack of knowledge by the home owners. Based on the writer's 35 years experience in on-site technology there is a need for more guidance how water can be conserved, how wastewater treatment and effluent disposal systems function, how they are operated and maintained and how greywater and effluent can be reused.

Sustainability of Effluent Disposal Systems

If aerobic conditions are attained within effluent disposal areas this will enhance the sustainability and effective life of the systems. This important aspect is discussed in more detail in Portfolio 2. The writer has concluded that this is a key factor to the effective performance of effluent disposal systems.

We must pay more attention to the significance of the sodium adsorption ratio (SAR) and the sodicity of soils.

We must appreciate that micro-organisms have their limits in terms of the rate of stabilisation (Dr Pam Pittaway, Microbiologist and Research Scientist, National Centre for Engineering in Agriculture, USQ).

A challenge is put out by Geary and Gardner (1998) that although principles of sustainability are well understood we must implement these principles at economic prices, given increasing urban development and ensure that on-site effluent treatment systems do not contribute to environmental degradation.

The writer advocates that to develop sustainable on-site treatment systems, we must become more innovative in the way we collect wastewater and treat our effluent.

Need for Innovation

Parker (1988) reported that in the USA a relatively modest investment into innovative technology research lead to a major payoff in cost savings and technological advancement. This certainly has been the experience of the writer.

There is an on-going need to apply an innovative approach to on-site wastewater technology to develop alternatives that are technical feasible, reduce costs, be acceptable by the end users and to be readily operated and maintained (Simpson, 1993).

The main requirements for innovation are robustness and to be readily operated and maintained (Kevin Poole, pers. comm., July 2011, Principal Consultant, Water Research Centre, Swindon, UK)

3.3 *Broad Overview of Portfolio Outcomes*

The general presentation style of the conclusions for each Portfolio involves the degree to which the aims and objectives have been addressed, specific technical outcomes, innovative work undertaken, advances in technology over more recent years, and the potential for more research and development.

Portfolio 1 discusses the problems and challenges of managing fluctuating wastewater flows and strengths in on-site situations, small communities, resorts and seasonal holiday facilities. It is a descriptive account of various viable alternatives for treatment some of which have been the writer's interest since 1974.

Portfolio 2 gives an account of the writer's experience and research work on determining soil permeability (K) and the influence effluent has on soil physical and chemical properties. The writer supports the concept of constant head permeability testing.

Portfolio 3 is an account of experimental work on reed/gravel beds in Australia and overseas. Reed/gravel beds are a viable wastewater treatment alternative for on-site situations and small communities. The writer has been interested in ecological engineering and natural wastewater treatment for many years hence, his interest in wetland technology. This portfolio examines the mechanisms of nitrification and the need for de-nitrification to achieve Total N removal. It has been concluded that reed/gravel beds are capable for Total N removal but there is a need for more research and development in the area of de-nitrification. It discusses a large leachate treatment wetland system designed by the writer in Singapore, which is now operational, and could be applied to a small community.

Portfolio 4 covers domestic wastewater composting, VIP toilets and greywater treatment. This is aligned to the writer's interest in natural wastewater treatment systems. The first experimental *Clivus* type composting plant in New Zealand has been a successful project since it was commissioned in December 1976. This is still functioning in a trouble free manner today. An experimental Simpson family VIP system set up in the mid-1970s was successful in that it catered for a full range of usages. There are several suitable alternatives for treating greywater which include pebble filters, wood chip/bark filters and reed/gravel beds. The safe reuse of greywater has put more emphasis on the need for a higher standard of treatment than has been customary in the past.

A postal survey of Australian domestic effluent disposal revealed a broad range of systems in operation in Portfolio 5 Part A. It is possible that some systems operating well in particular soils and climate types could be used within New Zealand and Australia. Innovative effluent treatment/disposal systems presented in Portfolio 5 Part B included anaerobic upflow pebble filters, aerobic upflow pebble filters, simple tube settlers, narrow trenches for steep sites, and wood chip/gravel trenches.

Portfolio 6 Part A covers early evapotranspiration (ET) development works in New Zealand. The natural mechanisms of evaporation and transpiration from vegetation can be effectively harnessed to dispose effluent. These natural mechanisms can effectively enhance the disposal of effluent in adsorption trenches and improve their sustainability. These systems are more suited to locations with a low rainfall and high ET. Nutrients, sourced from effluent, can be effectively assimilated by soil and specific vegetation (Portfolio 6 Part B).

The writer's technical survey of deep shaft disposal in the North Island of New Zealand was a most worthwhile (Portfolio 7 Part A). It concluded that this was a most viable disposal option for flat to moderate surfaces and situations with deep groundwater levels. The definite advantages are the economical use of land area and the further degradation of effluent. This simple and innovative treatment and disposal method could be adopted in other parts of New Zealand and in Australia. There is a need for more research on the degree of treatment that takes place and whether de-nitrification occurs. Sand mounds are an effective method of refining and disposing effluent (Portfolio 7 Part B).

Portfolio 8 has been complied in a manner which is traditional for Masters Degrees and PhDs. The writer was privileged to extend the experimental work by a New Zealand civil engineer to further develop the lime/seawater/magnesium salts technique of treating a range of wastewaters and process waters. This was a most economical process as it utilises burnt lime, a waste product from cement manufacturing, and seawater with its magnesium content or alternatively magnesium based waste products. The process yielded effluents with low organic matter, suspended solids, phosphorus and harmful bacteria.

3.4 *Chapter References*

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OSRAS, 2001, On-site Sewage Risk Assessment System, NSW Department of Local Government, April

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Portfolio 1 - Wastewater Treatment During Fluctuating Load Conditions

1 Introduction, Objectives and Technology Problems

For many years in communities not connected to centralised sewerage systems, the management of fluctuating wastewater loadings has been a problem. This includes seasonal wastewater loads in tourist and holiday resorts. The implementation of more stringent water quality legislation in New Zealand and Australia has generated the need to suitably treat wastewater loadings in variable population situations.

In this Portfolio, the term “fluctuating” is synonymous with several expressions; including:

1. Fluctuating – to vary irregularly, to rise and fall
2. Intermittent – stop for a time, ceasing at intervals
3. Variable – not constant, unsteady
4. Transient – fleeting, quickly passing away

The aim of this Portfolio is to identify the problems involved in the treatment of fluctuating wastewater loads and develop viable alternatives for the management of these loads. The objectives are to:

1. Draw on a literature review undertaken by the writer during post-graduate engineering studies (Simpson, 1974)
2. Document the writer’s design and operational experience in handling fluctuating loads in New Zealand from the mid 1970s until 1990
3. Document the writer’s fluctuating load experience in Queensland since 1990
4. Undertake an updated literature review to identify more recent developments in the treatment of fluctuating loads
5. Reflect on past and more recent findings and provide a ranking table on the suitability of technology alternatives for small communities
6. Where appropriate, make recommendations for further research and development
7. Draw conclusions on how or whether the new findings have added to the experience base on fluctuating load treatment and disposal.

Portfolio 1 provides an introductory description of these treatment alternatives. Several of the treatment technologies are covered in more detail in the other Portfolios of this dissertation.

Publications by Simpson Appropriate to Portfolio 1

1. Simpson, J S 1974, Wastewater Disposal in Smaller Coastal Communities, Dip PHE Dissertation, School of Engineering, University of Auckland, New Zealand.
2. Simpson, J S 1983 Report on Sewage Collection, Treatment and Disposal Alternatives, Waiheke Island, New Zealand (for Bartley Riddell Taylor Ltd, Auckland), June
3. Simpson, J S 1984, Report prepared for Corporate Membership of The Institute of Water Pollution Control (United Kingdom).
4. Simpson, J S 1984, Experimental work with Polyaluminium Chloride and Magnesium Salts in Wastewater Treatment, *New Zealand Water Supply and Disposal Association Annual Conference* , Palmerston North
5. Simpson, J S 1986, Chemical Treatment of Wastewater, Dip Sci Research Report, Auckland Technical Institute

1.1 The Problem of Fluctuating Wastewater Loads

The holiday patterns in New Zealand pose problems concerning the treatment of wastewater and disposal of effluent from smaller coastal and lake side communities. During the four to six week period of the traditional Christmas/New Year holiday, much of the population in New Zealand moves to the coastal and lake side resort and camping areas Gunn, 1970). This comment is supported by Asbey-Palmer (1973) who states that “the approach of Christmas heralds the large scale migration of New Zealanders from the cities to the beaches, swelling the permanent population of coastal communities by factors of five to ten and more”. Driven by the suitability of the weather, the other main holiday periods in New Zealand are during Easter, Labour weekend (October) and some school holidays (formerly in May and August).

During the months when people visit coastal and lake side areas, higher than normal loading is imposed on sewerage systems whereas during the remainder of the year these areas have a limited occupancy causing lower loading (Simpson, 1974). This effect is particularly evident in the camping grounds and resorts which may increase their population by six to twelve or more times the permanent population. As a result, the facilities are underutilised for a large part of the year and are expected to accommodate a population of several hundred to several thousand people, for the short holiday periods (Gunn, 1970).

The specific problem associated with lakeside settlements in New Zealand is the past and current discharge of effluent, either directly or indirectly, into a lake or nearby water courses. In these cases effluent disposal systems have not been operating effectively, particularly in the Lake Taupo and Lake Rotorua areas where there are free draining pumice soils. In these instances, rapid soil infiltration does not permit sufficient breakdown of effluent and uptake of the nutrients. Phosphates and nitrates stimulate the growth of plants, micro–organisms and planktonic algae (Simpson, 1974).

In more temperate coastal areas in Australia there is a similar pattern of seasonal occupancy. Sewerage systems have a small base loading and periodic peak periods during holiday periods. In northern Australia with the exception of the wet season in Northern Queensland, there is not the same problem due to year round higher temperatures, which favours a more regular occupancy.

Research undertaken in the USA (McRae, 1970) clearly showed that recreational pursuits associated with water were on the increase, hence the demand for accommodation was also increasing. In the writer's experience there has been a similar trend in northern New Zealand. Planners and engineers must continue to realise that sea coast and lakeside settlements and resorts have a distinct and different character from urban areas. Past failures to recognise this has lead to the creation of numerous sub-standard areas with poor services and unsatisfactory sewerage systems.

The problems of managing fluctuating flows are more apparent in smaller populations. Often similar problems are encountered in larger treatment plants but the design and operation of small scale systems can be seriously impacted. The prediction of the hydraulic and organic loadings on small plants is often uncertain. These factors, coupled with sub-standard operation and maintenance, often account for the poor performance of small treatment plants (Barnes and Wilson, 1976). A review of conventional methods of wastewater collection, treatment and disposal in coastal and other holiday areas was undertaken and reported by Gunn (1970). In coastal and lake-side areas, high environmental standards are imposed. In the writer's observations, this has been the experience in New Zealand generally since the 1980s (through the Natural Resources Act) and since the mid-1990s in Queensland (through the EPA Act 1994, EP (Water) Regulations and the Water Act 2000).

The characteristic ribbon development in New Zealand made the provision of communal water supplies and sewerage uneconomical. The general demand for holiday accommodation and facilities in North Queensland coastal areas and islands poses problems for the provision of communal sewerage. This is partially due to the constraints of disposal of treated effluent within recreational waters and estuaries. There has been a tendency to establish permanent residences in coastal areas and commute to larger centres for employment. A proliferation of septic tanks and trench systems has promoted public health risks and water contamination problems. Residential housing redevelopment has been allowed in low lying areas, with poorer quality soils, drainage and flooding problems. Some islands within Moreton Bay, Queensland have been allowed to be subdivided into small lots sizes, drainage problems and sub-standard soils.

2 Research by Simpson on Fluctuating Loads

This Section provides insight into how the writer designed and managed fluctuating wastewater load projects in New Zealand and Queensland. The word "research" in the context of this thesis includes postal and field surveys of treatment and disposal technologies, case studies where a methodology has been developed and innovative design work. The writer rates the development of innovative work technologies as a form of applied research. The unit processes considered or used to achieve the treatment of fluctuating loads are given.

2.1 New Zealand Fluctuating Load Experience

During the period 1974-1984, the writer was involved in several New Zealand projects that faced the problem of treating fluctuating wastewater loads.

A major project was the planning of handling load variations at seven smaller coastal and harbour side communities and beach resorts in the former Whangarei County Council, Northland, New Zealand (Simpson, 1974). The location, type of community, permanent population, estimated holiday population and the fluctuating load factors are given in Table 1.1. This includes the township of Waipu Cove which is discussed further in Section 4.5.

Table 1.1: Coastal and Harbour Communities, Whangarei, NZ – Wastewater Load Fluctuations and Populations

Project Location and Type	Permanent Population (1974)	Estimated Holiday Population (1974)	Fluctuating Load Factor
Oakura – coastal township	200	1,700	8.5 times
Tamaterau – inner harbour township	600	1.400	2.3 times
Ngungaru – Estuarine township	500	1,500	3.0 times
McLeods Bay – harbour township	150	550	3.7 times
Taurikura Bay – harbour township	150	650	4.3 times
Waipu Cove – coastal township	200	1,500	7.5 times
Langs Beach – coastal township	150	950	6.3 times

It can be seen from Table 1.1 that all communities were within the wastewater load variation of 2.3 to 8.5, which required some treatment planning to cope with this degree of fluctuation. A typical method of wastewater treatment and effluent disposal was as follows:

1. Septic tanks or Imhoff tanks for primary treatment. The septic tanks and Imhoff tanks provided for the capture and cold digestion of solids
2. Secondary stabilisation lagoons. The secondary lagoons provided secondary treatment and buffer storage for widely varying flows
3. Advanced secondary treatment in a maturation lagoon (disinfection and effluent polishing). A maturation pond also provided for buffer storage of effluent
4. Treated effluent disposal by spray irrigation, within a sandy vegetated area. The spray disposal area was divided into sectors which catered for varying flows.

The Waipu Cove project in Table 1.1 has been used as a model community treatment system. The writer's study (Simpson, 1974) focused on the holiday and resort seaside, and lake-side areas in New Zealand. This study included the smaller coastal and harbour side communities in Table 1.1.

2.2 Queensland Fluctuating Load Experience

During the period 1997 to the present day, the writer has been involved in several fluctuating load projects in Queensland. These are listed in Table 1.2.

Table 1.2: Projects Subjected to Fluctuating Wastewater Loads – South East Queensland

Project/Location	**Peak Flow (L/d) or EP**	**Wastewater Treatment System**	**Degree of Flow Fluctuation**
Sunray Strawberries, Wamuran, Qld[1]	4,000 L/d	Activated sludge package plant	31 times
Strawberry Farm, Ningi, Qld[1]	8,580 L/d	Activated sludge package plant	29 times
Hotel, Service Station and Shops, Murphy's Creek, Gatton	4,580 L/d	Primary tanks and sand filter beds	5.2 times
Outdoor Education Centre, Stanmore, Qld	2,320 L/d	Primary tank and sand filter bed	40 times
Kiosk and Caravan Park, Moore, Qld	1,840 L/d	Activated sludge package plant	9.2 times

Holiday Cabins, Mt Mee, Qld	1,380 L/d	Primary tank and sand filter beds	12 times
Tourist Centre, Allora, Qld	35,926 L/d	Primary tanks and sand filter beds	18 times
Caravan Park, Laidley, Qld[2]	6,000 L/d	Primary tanks and sand filter beds	11 times
Outdoor Education Facility, Imbil, Qld	20,240 L/d (ultimate design)	Primary tanks and sand filter beds (to cater for 3 stage development)	50 times (Ultimate design)
Watson Park Convention Centre, Dakabin, Qld[3]	School – 200 EP.Motel units - 40 EP. Pre-school – 50 EP. Periodic camps – 200 EP.	Septic tanks, oxidation lagoons, vetiver grass overland flow plot and pasture/tree irrigation	Normal load (490 EP) plus annual convention of about 4,500 EP. Extra load factor 9.2 times.

Notes:

1. The activated sludge plants all had a base loading, to keep the microorganisms active. If the base load was not available the Site-based Management Plan gave guidance on how the plant could be seeded and re-started. In the case of the strawberry farms, it would be just prior to the picking season
2. Primary tanks and sand filter beds systems provided storage for varying flows
3. Effluent storage is increased just prior to the September annual convention.

The writer has had experience with the design of several accommodation facilities for strawberry pickers in the Caboolture area, which are faced with a limited picking season from May to November, with a peak during that period. During the off-season, the treatment plant catered for a base load of only the manager and family and maintenance day workers.

The Watson Park Convention Centre, Dakabin wastewater treatment system caters for a school, child care centre, camping ground and a large annual convention attended by some 4,500 people each September. The secondary lagoon was pumped out and the effluent spray irrigated prior to the influx of the annual convention.

A recent project (2009) is a proposed caravan park and camping ground near Laidley, Queensland. During low occupancy periods the base load will be provided by the caretaker and his family and say one caravan.

2.3 Review of Wastewater Treatment Alternatives for Handling Fluctuating Loads

2.3.1 Biofiltration

The writer reviewed methods for handling fluctuating loads for biological filtration in Simpson (1974) and Simpson (1983) and found that:

1. Increased efficiency can be achieved by re-circulating the filtered effluent. The degree of re-circulating depends on the input. During low flows it is important that the biomass receives sufficient nutrients and a balancing tank and nutrient dosing system can facilitate this function
2. A tank prior to a conventional bio-filtration unit containing randomly packed plastic media acts as a roughing filter. This concept was being trailed at the Auckland Regional Council, New Zealand at the time the writer was working there. It was dealing with a three-fold increase in organic flow. It was understood to be capable of treating four to five times the load of conventional rock media
3. Randomly packed plastic media can be used as partial replacement of conventional rock filter media (Pullen, 1977). The roughing filter can be kept activated during low flows and then participate during high organic flows
4. Alternating double filtration is a viable system for handling variable loads as it offers additional flexibility (Pullen, 1977 and Simpson, 1974). The writer reviewed the process in Ruidoso, Lincoln County, New Mexico (Simpson 1974). This town had a resident population of 3,000 that increased to 20,000 during the holiday season

The writer noted the experimental work on biofilter recirculation to reduce BOD_5 that was undertaken at the Whenuapai Airforce Base, Auckland. Ratios of filtered effluent to incoming flow achieved the figures in Table 1.3 (Jeff Burns, Plant Operator, Ministry of Works and Development, pers. comm, 1983). The ratio of 2.9:1 was accepted for operational purposes.

Table 1.3: Biofilter Recirculation Ratios and BOD Removal – Whenuapai Airforce Base.

Filtered Effluent: Incoming Flow	BOD_5 Removal (%)
1:1	70
2:1	93
3:1	95 +

2.3.2 Stabilisation Lagoons

Stabilisation lagoons or oxidation ponds contain an upper aerobic zone and a lower anaerobic zone and are consequently called facultative lagoons. Algae photosynthetically produce oxygen to allow an oxidation process to take place.

During the daylight hours dissolved oxygen and pH rise to a peak, followed by a drop during the night.

Facultative or stabilisation lagoons are suitable for smaller coastal communities for the following reasons (Simpson, 1974):

1. Relatively inexpensive to construct
2. Require minimal maintenance
3. Low operating costs
4. Some benefit from protection of lagoon operators from enteric disease infection
5. High quality effluents can be produced (refer to algal build up below)
6. Capable of handling fluctuating loads
7. Capable of remaining near dormant during minimal flows.

Waste stabilisation lagoons were a viable option in coastal areas of the North Island of New Zealand. The writer was responsible for supervising the design of the first stabilisation lagoon for the former Whangarei County Council at Marsden Bay in 1973/74. Refer to Plate 1.1 for a photograph of the Ruakaka lagoon system. This photograph was taken in 1975.

The site was located within sand dunes adjacent to the ocean and over an area with a shallow groundwater level. As a result, it was important that the lagoon did not leak. To protect the integrity of the rubber liner all tree roots were removed, a suitable thickness was selected which allowed for the impacts weathering over time, and particular attention was made to the jointing operation. The single lagoon catered for a variable wastewater flow generated by the construction of an adjacent oil fired power station and residential development. The development was undertaken in stages, during the power station construction and commissioning. Some houses were then sold while others were relocated.

Stabilisation lagoons can be designed in stages, or as separate cells, to cater for fluctuating loads and progressive growth. In the writer's experience, surface or submerged aerators can be installed in some cells to enhance treatment during peak periods. However, lagoons produce effluents high in algal matter and require some form of polishing treatment, to reduce the BOD_5, suspended solids (as algal matter) and nutrient build up.

Plate 1.1: Ruakaka, New Zealand Oxidation Lagoon

2.3.3 Aerated Lagoons

The writer considers that the aerated lagoon concept has much potential for handling variable loads, reinforced by Eckenfelder (1970), by using the following approaches:

1. Using variable speed floating or submerged aerators
2. Aerator units may be readily added, repositioned or removed to cope with changing conditions
3. Dissolved oxygen (DO) monitoring at least weekly and ensuring the DO does not go below zero
4. The first lagoon could be mechanically aerated and the second lagoon can function as a facultative unit

The writer noted that a dual lagoon system was being designed for treating camping ground/caravan park wastewater in Northland, New Zealand. During the off-season the aerator is housed and the small lagoon functions as a facultative lagoon. During the holiday season the floating aerator is installed to cope with the shock and high loads. The camp manager predicts when campers and tourists are due.

Aerated lagoons are very good for higher BOD_5 reductions and nitrification (Eckenfelder, 1970).

The writer undertook a feasibility study of wastewater collection, treatment and effluent disposal options (Simpson, 1984) for Waiheke Island, near Auckland in New

Zealand. Since the Island communities were subjected to fluctuating loads, one option included an aerated lagoon followed by a stabilisation lagoon.

The writer undertook the preliminary design of a series of aerated ponds for the organic reduction and nitrification of landfill leachate in Singapore in 2006 (Waste Solutions Australia, 2006).The design flows allowed for flow and organic variations and this was facilitated by a holding or balancing tank. The aerated ponds were followed by a settling unit, in this case, to avoid the carry-over of solids into a system of reed/gravel beds.

2.3.4 Septic Tanks

Septic tanks are typically double chambered tanks which allow settleable solids to settle and form a sludge layer on the bottom of the tank. Oils, grease and other light materials float to the top and form a scum. Settled wastewater flows from the clear zone between the scum and sludge. The rate of degradation of the sludge is very slow.

In the writer's experience, a major problem is the carry-over of solids which clogs effluent trenches to the extent they need replacing. Over more recent years effluent filters have been developed to avoid solids carry over. Septic tanks are discussed further in Portfolio 5.

Septic tanks cater well for residences that are occupied for limited periods. Owners must ensure that the crust is broken after a reasonable period of absence. This anaerobic system does not require a constant flow of wastewater, which makes it suitable for variable loads. Also higher flows could result in some solids carry over. Effluent filters are discussed later in this portfolio.

2.3.5 Imhoff Tanks

Imhoff tanks have been described as being a more sophisticated version of a septic tank. They are two storey tanks with sedimentation being undertaken in the upper storey and digestion underneath. The tanks are fitted with gas vents and floating material and scum must be removed periodically. Fixed sprays are fitted, using treated effluent, to control floating matter.

They have been used in New Zealand and Queensland extensively, as a primary treatment process. A survey in New Zealand in 1971 found that Imhoff tanks were handling load fluctuations varying from 2 to 7 fold (Simpson, 1974). In the mid-1970s Imhoff tanks in New Zealand were being replaced by stabilisation lagoon systems since no separate sludge disposal system was required.

2.3.6 Extended Aeration Package Plants

Extended aeration is a variation of the activated sludge process, which is largely an aerobic treatment system. During aeration the influent is mixed with a large mass of previously grown organisms. The solids are flocculated and removed from the liquid by settling.

The writer (Simpson, 1983) was responsible for the design, construction and operation of several extended aeration package plants in northern New Zealand for *G Buchan Water Treatment Co*, in Auckland, over the period 1982/83. As it has some facility for catering with flow variations, the extended aeration process has been successfully used in New Zealand and some of the Pacific Islands.

Staff under the writer’s supervision monitored an extended aeration package plant with a gravity sand filter serving holiday motels in 1974-1975 in the former Whangarei County Council. Over a 15 month trial period the wastewater load fluctuated to up to 8 times the base loading and the average effluent BOD_5 was 16 mg/L. Considering the licence limit for BOD_5 was 20 mg/L under fluctuating load conditions, this is considered to be a very acceptable performance. Treated effluent was recycled as toilet flushing water in the motels, which was an adventurous step at the time.

The writer was commissioned to modify and extend an extended aeration treatment plant in the Bay of Islands in 1976 servicing a 100 lot residential subdivision. At the time the treatment was commissioned only five houses were connected. After a brief settling down period the system was operating satisfactorily under this very low loading. The air supply had been reduced and the activated sludge returned to build up the mixed liquor suspended solids. It was felt that part of the successful performance, under a very low loading, was that it was operated by a certified wastewater treatment plant operator.

In the writer’s experience fluctuating loads were handled by using the following operational practices:

1. Installing adjustable baffles and weirs
2. Reducing or increasing aeration capacities, using multiple compartments
3. Using a special time clock, with 15 minute intervals, to control aeration times per day. This proved to be more successful than throttling or increasing the air supply
4. Increasing or decreasing final settling capacities
5. Including a coarse sand filter, following secondary settling, to be used during higher loads
6. Including a pre- settlement tank to settle out gross solids and some settleable BOD, prior to aeration
7. Seeding aeration chambers, with nutrients or activated sludge, just prior to an expected load shock

2.3.7 Sand Filters

The filters may be designed to provide free access or may be buried in the ground. Some filters employ effluent recirculation and others can operate as alternating units. The writer was instrumental in gaining approval for the use of *ECOSAFE* sand filters in the former Caboolture Shire Council. There are now several hundred *ECOSAFE* sand filters in South East Queensland, servicing individual houses, motels, schools camping grounds, school camps, taverns and smaller shopping centres; many of which I have engineered.

This sand filter is capable of handling fluctuating loads since the permeability of the sand is sufficiently high to allow the downward velocity of the wastewater to be constant, regardless of the rate of application to the surface area. This occurrence allows the organic material in the wastewater to make constant contact time with the microbial mass attached to the sand particles (Brian Hawthorne, *ECOSAFE*, Caboolture, pers. comm. 2009).

In the writer's experience, sand filters have been successful as smaller systems following extended aeration treatment.

During peak periods, partially treated effluents have been suitably polished in alternating sand filters, for shorter periods, without any appreciable decline in the final effluent quality. In some cases, where very stringent effluent quality standards have not been imposed, the writer has designed a "medium rate" filter (a compromise between a slow and rapid sand filter which is capable of meeting the effluent discharge standards but is more economical than a slow sand filter system) with success. Particular attention should be been given to the following aspects:

1. Sand particle size
2. Splash plate design
3. Pumping dose rates
4. Problem-free dosing systems
5. Under drain ventilation and regular filter maintenance

Design application rates were varied according to the filter media available and the standard of pre-treatment. The writer used a particle size range of 0.4 to 0.8 mm and an intermittent dosing rate resulting in no more than 50mm ponding over the filter surface.

When septic tanks and Imhoff tanks have been used for primary settling and cold digestion, the writer has experienced odour and filter clogging problems. A coarse grade sand has been used and at times the effluent has been recirculated to minimise the odours.

2.3.8 Granulated Bark Filters

Granulated bark filters are concrete basins containing bark which is dosed on a continuous or intermittent basis with primary effluent. In 1982 the writer was directly involved in designing the wastewater treatment for a small shopping centre at Kumea, a village just north of Auckland where butchery and hair dresser wastewater was treated by an extended aeration package plant. There were definite problems with the fats and grease (butchery) and toxic materials (hairdresser, at low dilution) being discharged into the plant. Loads fluctuated since the butchery was washed down twice daily. The grease traps were extended and a flushing system installed at the hair dresser, to increase dilution.

A trial gravity granulated bark filter was installed as a pre-treatment system. After several months of operation there were no odour or treatment problems. It was concluded that there was considerable merit in using aged granulated bark as a filter media for handling widely variable loads or withstanding shock organic loads for individual houses and smaller treatment systems, functioning as both aerobic and anaerobic systems.

2.3.9 Upflow Pebble or Banks Filter

This system, often known as a Banks (after the developer) pebble filter, is a means of clarifying secondary effluent by upflow through a shallow bed of pea gravel. Clarification is achieved mainly by straining or filtration, but it is apparent that flocculation and adsorption play their part.

The writer reported (Sewerage Guidelines, 1992) that at an upflow rate of 25 to 30 $m^3/m^2/d$, upflow pebble filters were operating satisfactorily in New Zealand and Fiji. Due to system robustness, simplicity and reliability this treatment method was included in the *Queensland Guidelines for Planning and Design of Sewerage Schemes, 1992.* The writer was the Project Engineer for these guidelines during 1991 to 1992.

The writer concluded that the upflow (or reverse flow) filter was suitable as a tertiary treatment system for handling widely fluctuating loads (Simpson, 1983). One project involved the use of an upflow pebble filter, following extended aeration treatment. Wastewater from a township, which catered for commercial fishing, holiday camps and permanent residences, was treated by extended aeration, followed by an upflow pebble filter. The treatment system complied with stringent discharge requirements of 10mg/L BOD_5 and 10 mg/L suspended solids into a harbour in Northland, New Zealand. Peak summer loadings were seven times the winter or base loadings. There was a need to back wash the filter about once a week. This was simply undertaken by lowering the water level about 200mm, below the pebble bed, and flushing excess solids by hosing with recycled effluent.

There is merit in fitting an upflow pebble filter, suspended in a wire mesh cage, around the outlet of a sedimentation tank. This retrofitting of the upflow filter avoided the cost of installing a separate chamber.

It is the writer's opinion, there is considerable merit in applying this method as a tertiary process or for treating fluctuating loads in the future. It has tended to be a trend that during my work career a simple and viable system is soon bypassed by alternative new technology.

2.3.10 Rotating Biological Contactors (RBC) or Bio-Discs

The RBC process comprises a semi-submerged series of discs, mounted on a rotating horizontal shaft, on which a fixed biological film of biomass develops. The discs are immersed to about 40 % of their diameter and they are alternately in contact with the wastewater then the air. The disc units are followed by a humus tank, which catches the biomass that has sloughed off.

Refer to Figure 1.1 showing a typical cross section and a long section of a RBC system (Antonie 1970).

The writer has followed the development of biological discs since 1974 (Simpson, 1974) for the following reasons:

1. Detention times are relatively short due to faster contact
2. In overload conditions the biomass on the discs is not lost, hence one reason for the ability to handle overloads
3. Very low head losses
4. Considered to be economically competitive with optional systems
5. Low operating costs
6. Using low density, high buoyancy materials for the discs results in low power requirements
7. The rotating discs could be installed as units. Additional disc units could be commissioned to cope with expected peak periods
8. Discs can be made from non-biodegradable, light, and non-corrosive materials
9. There was an absence of odours, flies and noise nuisances.

The writer undertook a university study tour, during Dip PHE studies, of an RBC plant at an Army Camp in Ngaruawhia, New Zealand in 1974. The plant consisted of an Imhoff tank, RBC or Hartman disc unit, followed by a humus tank. The design flow was about 136 m^3 per day and it was found that the plant coped with flows around 455m^3 per day, which included infiltration water, without any noticeable effect on plant performance. (Andrew and Gielen, 1969).

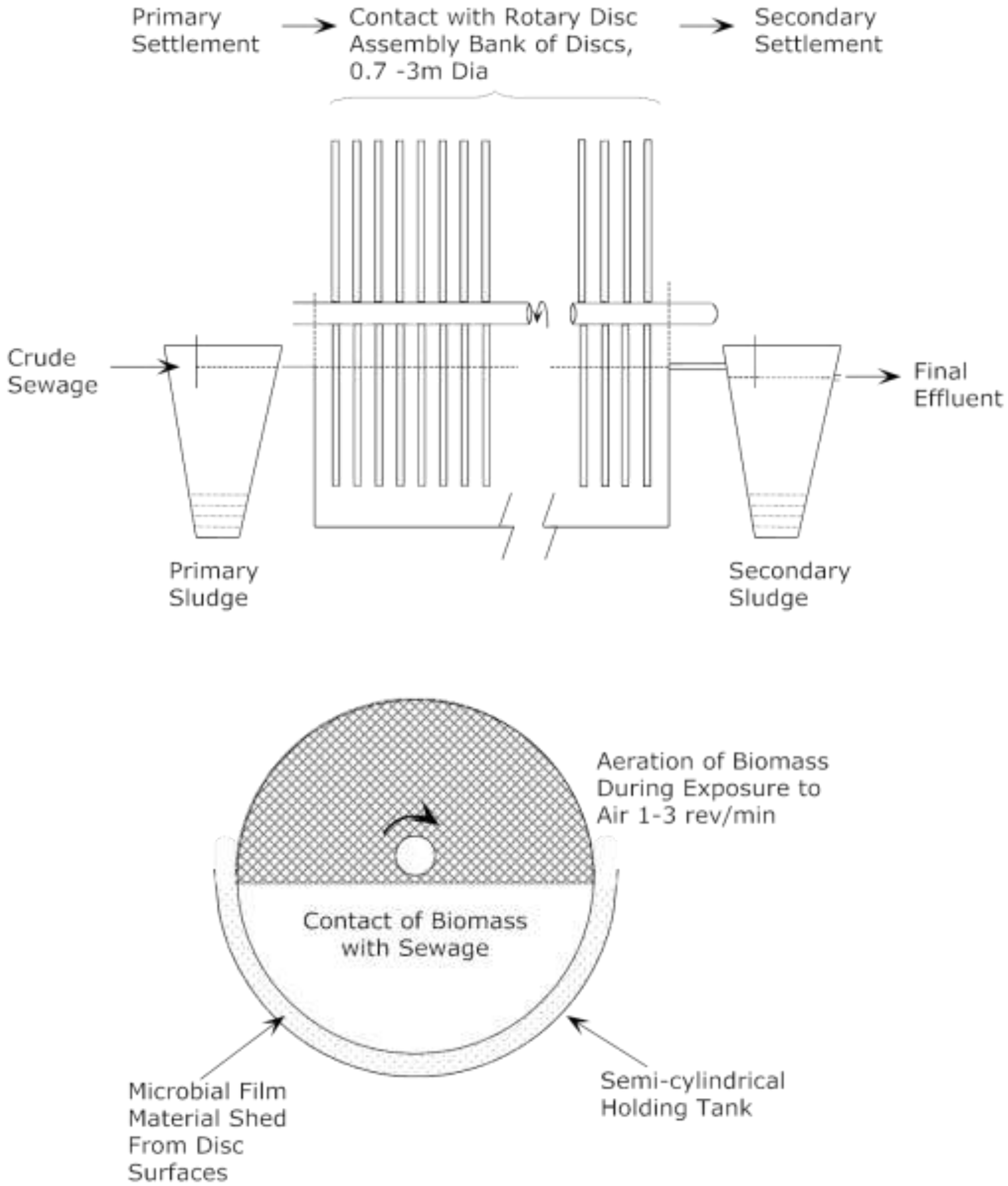

Figure 1.1: Typical RBC System (NTS)

The population of Waiheke Island (near Auckland) can vary some fivefold, from winter to the tourist season in summer. Due to the fluctuating load problem, the writer reported on RBCs as a treatment option (Simpson, 1983). The writer advocates that higher degrees of load variations can be handled by the following:

1. Including a balancing tank, prior to the RBC unit, to help buffer peak flows
2. By recirculation of the effluent, during low flows
3. Including an upflow pebble filter, or simpler, within the settling tank or humus tank.

It is interesting to note that the RBC method of treatment has come back into vogue in Queensland in more recent years. *Stengelin* RBCs, installed by *EPCO*, Brisbane, are in operation in the Torres Strait and some Aboriginal communities in Queensland. They are stable systems and perform well with a low loading of at least 10% of the design loading (Grant Cobbin, EPCO, pers. comm. 2009) An RBC system was installed at an Australian Army Jungle Training Centre in Tully, North Queensland in 1986 and it has a base load at all times. The training centre caters for up to 50 people for 2 to 3 week periods. A carbon source, dried dog food, is dosed into the RBC tank before the full load is imposed.

The writer inspected an RBC wastewater treatment plant at the Warwick Resort on the Coral Coast of Fiji, in June 2010. The plant initially had an oxidation ditch which had become overloaded. To cater for the increased load and the higher standards required for the discharge of effluent to the outer reef, two RBC units were installed and operated in parallel. The design flow was about 20,000 L/d but the plant was subjected to flow variations as a result of the following:

1. Seasonal changes in resort occupancy
2. Changes due to lower occupancy, for example, political uprisings
3. Lower occupancies during cyclonic weather and flooding.

It is understood that the plant produces an effluent which is low in BOD_5, suspended solids, phosphorus and Total N (Murray Thompson, Chief Engineer, Fiji, pers. comm. June 2010). The RBC units cope with load variations, assisted by flow balancing during high inflows, by using the storage within the old oxidation ditch.

Nemeron et al (2009) reports on the ability of RBCs to withstand shock loads. When the peak load exceeds 2.5 times the average daily flow or when a large organic load occurs, the appropriate measures to take are flow equalisation measures or to add disc units.

2.3.11 Biodrum

The Biodrum is a further development from the RBC system. Casey (1997) reports on the use of the Biodrum as a cylindrical mesh drum filled with random packed media. The writer assisted with the design of Biodrum plants for use in housing and light industrial estates in Kuala Lumpur, Malaysia in the early 1980s. The key advantages noted by the writer were:

1. Robust construction when compared with RBCs
2. Media is readily available and it consists of inert type plastic or polystyrene
3. Fluctuating loads are managed by variable depth of immersion, variable speed of rotation and the use of multiple units.

The Biodrum system is still used for treating wastewater. Chiemchaisri and Liamsangoum (2009) report on the use of a multi-stage Biodrum for treating viable organic and nitrogen loads in Thailand. The detention times varied from 3, 6 and 12 hours and average organic removals varied from 90.9 to 96.3% and average nitrogen from 81 to 91.5%. The optimum detention time was 6 hours.

2.3.12 Deep Shaft Disposal

Deep shaft disposal is reported as an alternative technology in Portfolio 6 and it is covered in detail in Portfolio 7. This system consists of <6.0 m deep shafts, with a diameter of 600 to 1,000 mm, which are backfilled with rocks. These rocks develop a biomass, hence the shafts act as a trickling filter unit.

In the writer's experience, groups of deep shafts have been used for effluent disposal for individual homes, motel complexes, school camps, church camps, camping grounds and other resort facilities in Northland, New Zealand. Several of these projects were in the Paihia/Waitangi/Russell areas in the Bay of Islands. Upper low permeable clays prevented the use of shallow trenches and beds. A more permeable weathered rock zone was usually found at about 2.5 to 4.0 m depth.

For variable wastewater loadings the writer has considered the following aspects during the design process:

1. For individual houses, a group of three deep shafts have the capability to be used infrequently, at low loads and during peak loads
2. For larger projects, a network of deep shafts with control valves where the number of shafts in operation depended on the weather and effluent loadings
3. Each deep shaft has the built-in capacity for effluent storage under high load situations or prolonged adverse weather situations
4. Periods of zero loading are not detrimental to the performance of the shaft.

Having had experience with fluctuating loads the writer reported on deep shaft disposal as an alternative effluent disposal system for the seasonal and holiday communities of Waiheke Island in New Zealand (Simpson, 1983).

2.3.13 Constructed Wetlands

Constructed wetlands include open water surface systems which have a depth of water of typically 500 to 600 mm, shallow fringe areas, deep zones and are planted with emergent and floating plants. Another type of wetland is the reed/gravel beds which are discussed in Portfolio 3.

In 1992 the writer was responsible for setting up the Artificial Wetlands Co-ordination and Advisory Committee in Queensland. This consisted of a multi-disciplinary group of specialists to study the performance of ten pilot wetlands in a range of Queensland climatic regions, including sub-tropical, tropical dry, tropical wet, arid west. This program gave me the opportunity to be involved in associated scientific studies with various Queensland universities.

A pilot scale open surface wetland treating primary effluent from the Ingham Township in North Queensland was accidently subjected to wide load variations. Due to a trickling filter breakdown, the daily loads of primary wastewater were directed to the pilot wetlands for some weeks. Due to the writer's ongoing interest in treating variable loads this situation was noted. The monitoring results are no longer available but the writer noted (at the time) that BOD_5 and suspended solids results showed no marked spikes due to the gross overload situation. The writer maintains that it was very evident that open surface wetlands have the ability to buffer high loadings, and can thus handle fluctuating organic and hydraulic loads.

The writer recently designed a wastewater treatment system for holiday cabins near Woodford, Queensland. There are no laundry facilities, so kitchen and shower water and blackwater is directed to a dual septic tank, reed/gravel bed and ET bed. The occupancy is expected to be variable. The reed/gravel bed will cater for load variations and the low occupancy periods will facilitate a resting period for the ET effluent disposal bed.

2.3.14 Domestic Wastewater Compost Converter

Dry composting toilets are devices which receive faeces, urine, paper and kitchen organic wastes into an aerated chamber. This waste matter is decomposed by bacteria and other microorganisms. Greywater must be treated and disposed of separately. The most well known dry composting toilet is the Swedish developed *Clivus Multrum*.

The compost converter is discussed in some detail in Portfolio 4. In the writer's experience this low technology option is particularly good for handling load extremes for the following reasons:

1. They can function well for longer periods with minimal loadings
2. Peak loads, for shorter periods are not a problem, if they have some prior base load
3. They can tolerate the loading scenarios normally expected in urban, semi-rural and holiday homes.

The major benefits are the nil energy requirements and that the system operation is not demanding.

The writer was interested to learn of experiences with larger composting systems in Queensland. The writer consequently sought an update on the experience with larger *Clivus Multrum* systems in Northern Queensland, and contacted *ENVIROMAT AUSTRALIA* in 2010. The outcome is summarised as follows:

1. The units compensate for humid climates if there is good air flow
2. A good C: N ratio produces an inert final waste product
3. Vinegar flies were a problem as the screens were originally not effective in preventing the entry of these small insects
4. A Clivus unit at Lake Tinaroo, Northern Queensland suffered for a period since there was virtually no base load during the week followed by a very high weekend usage. Composting toilets are known to cope with such loads so the writer cannot offer no reason for the problem
5. The Clivus system is considered by *ENVIROMAT AUSTRALIA* to be by far the best wastewater system for tropical Queensland.

During 2008 the writer was involved in the planning of a large group of holiday cabins at Lake Moogerah, near Boonah, Queensland. The treatment system included individual Clivus composting units, greywater tanks and effluent irrigation. The occupancy is expected to generate fluctuating loads. The writer has used and observed dry composting toilets in caravan parks, camping grounds and national parks in New Zealand, Australia and Canada. The lack of odours in each case was an indication that these systems were operating in a balanced manner.

2.3.15 Common Effluent Drainage/Extended Aeration Treatment

An extension of the Midge Point Township, by 103 residential lots, in the Whitsundays was proposed in 2007. In 2008 the writer was commissioned to review and update the wastewater treatment and effluent disposal system. This project was of particular interest to me due to the challenge of catering for widely fluctuating loads. Being on the interface between fresh and saline waters, a high degree of treatment to Class A+ is required.

The initial common effluent drainage (CED) component consisted of 2,500 L septic tanks on each lot to capture solids for macerating. Effluent was to be pumped in a pressure sewer line to the treatment plant. The treatment plant loading would vary from up to 5% to the maximum design loading for 104 lots fully built on. A primary effluent holding or buffering tank would accept the variable flows. The effluent would be subjected to extended aeration, membrane filtration and UV disinfection. Several aeration tanks, settling units and disinfection units would be commissioned, to cater for increased or fluctuating loads. Operational stability was important to facilitate nitrification and de-nitrification and phosphorus removal. It is understood that this treatment system has been approved by the Queensland EPA.

2.3.16 Tube Settlers

The writer adopted simple 100 mm diameter PVC tube settlers within septic tanks in the late 1970s whilst working as a Public Health Engineering Consultant in the Whangarei and Bay of Islands County areas. They were very effective in retaining solids, which could otherwise overflow and block up trench systems, or the solids content be revealed in effluent monitoring. It was found that the tube settlers had to be well fixed or concreted in and have a minimum diameter of 100 mm. In the case of humus tanks, following biofilters or RBCs, Pullen (1977) claimed that the installation of inclined tube settlers had merit for avoiding solids carry over.

2.3.17 Plate Clarifiers

The writer had experience with 60 degree inclined plate settlers over the period 1982-1983 in Auckland. These were integrated into *Jet Aeration* package plants, as secondary clarification units. The clarified solids were returned to the aeration chamber by a sloping floor. These modular units were periodically cleaned by lowering the liquid level and hosing. It was shown that plate clarifier units could be installed into existing primary and secondary clarifiers to help withstand shock loads. The capacity of a clarifier could be increased two to four times.

2.3.18 Lamella Plate Clarifier

The Lamella plate clarifier has a proven application in the treatment of a range of industrial wastewaters (Grimes and Nyer 1978). The writer used this method for the conceptual and preliminary design of food processing and industrial wastewaters in the Auckland area in the 1980s. At the time the writer also rated the Lamella plate separator as having potential for handling fluctuating loads and peak loads with high BOD_5 and suspended solids.

A ski resort at Dinner Plains in the Victorian Alps was being subjected to large seasonal population fluctuations. The primary effluent was being discharged to a series of oxidations and polishing lagoons. The risk of algal blooms during summer months was a threat. The Lamella plate clarifier was intended to further reduce the overall pollutant load and even out the impact of flow variations. East Gippsland Water undertook a three phase investigation, reported by McKean (2010), and the results are summarised in Table 1.4**.**

Table 1.4: Removal Efficiencies – Three Phase Trial, Dinner Plains, Victoria

Treatment Method	BOD_5 removal (%)	Suspended solids removal (%)	TN removal (%)	TP removal (%)
Primary Sedimentation Tank	10	43	<1	21
Lamella Clarifier	30	57	1	6
Lamella Clarifier/Coagulant	31	59	15	47

The writer's comments on the results in Table 1.4 are as follows:

1. The Lamella clarifier phase produced good removal efficiencies in terms of BOD_5 and suspended solids
2. There is virtually no improvement in terms of BOD_5 and suspended solids removal when using the coagulant
3. The adding of the coagulant Polyaluminium chlorohydrate (PAC 43) gave some TN removal and nearly 50% TP removal
4. The 21% removal of TP in the primary sedimentation tank is questionable.

It is understood that the trials are continuing. It is envisaged that that optimisation of the coagulant dosing regime will improve the efficiency of the Lamella clarifier. The writer agrees, based on his chemical wastewater treatment research work, that further work on dose optimisation, mixing and settling rates will improve the treatment efficiency.

It was concluded by Mc Kean (2010) that the improved nutrient reduction, using the coagulant in phase 3, would reduce the likelihood of algal blooms occurring during the summer.

This technology has been included in this portfolio since it is normally used for industrial wastewater treatment. Its application for domestic wastewater treatment is considered by the writer as a novel idea, with merit for handling variable loads. The writer supports the use of polyaluminium chlorate (PAC) to enhance nutrient and other contaminant reduction, based on his experimental work with PAC on a range of waters and wastewaters (Simpson, 1984).

2.3.19 Brush Filters

Proprietary upflow brush filters were investigated at the Oakey Wastewater Treatment Plant in Queensland. The investigation was undertaken by the Department of Natural Resources during the writer's employment in the early 1990s. Brush filters and 60 degree tube settlers were selected as alternatives worthy of trialling. The brush filter tender was accepted due to the lower capital cost and the anticipated superior performance in terms of handling variable loads (CopaClean, undated).

The writer inspected the brush filter installation during the trial and was impressed by the following aspects:

1. Simple system able to be readily installed in existing structures or tanks
2. Readily maintained by lowering the effluent level and hosing down
3. Suitable for both radial flow and pyramidal tanks
4. Operate as upward flow and horizontal flow systems
5. Manufactured from high quality industrial plastics and stainless steel

6. Have no moving parts or energy consumption.

These units were fitted into overloaded secondary filters and humus tanks, just below the surface.

Performance results for BOD_5 and suspended solids (SS) for No 1 secondary sedimentation tank are given in Table 1.5 (CopaClean, undated).

Table 1.5: Oakey No 1 Sedimentation Tank Performances - Brush Filters

Date	After Installation BOD_5 (mg/L)	After Installation SS (mg/L)
January 1994	8	25
February 1994	13	25
March 1994	16	15
April 1994	16	11
May 1994	13	13

Notes:

1. Brush filter installation took place in late 1993
2. October 1993 before installation results BOD5 was 31 mg/L and suspended solids 57 mg/L
3. Licence suspended solids (SS) is 20 mg/L
4. Licence BOD5 is 15 mg/L

The following performance changes can be seen from Table 1.5:

1. The October 1993 (pre-installation) performances were well over the licence requirements for BOD_5 and suspended solids
2. Suspended solids levels came within the licence requirement over the last 3 months
3. BOD_5 levels were just over or within the licence requirement
4. The May 1994 performances were under the license requirements for both effluent parameters.

It is claimed that *CopaClean* clarifiers (the type used at Oakey) will typically reduce suspended solids by 30-70 % with an associated BOD_5 reduction of 1-30% (technical data supplied by *Aquatec-Maxcon Pty Ltd*, Ipswich, May 2009).

The writer visited the Water Research Centre in Swindon, UK during July 2011 and discussed the brush filter systems. The technology was popular for many years but it has fallen out of favour recently, due to the need to lower water levels and hose down the brushes and access to the tanks was sometimes difficult (pers. comm. Kevin Poole, Principal Consultant, Water Research Centre, Swindon).

2.3.20 Lime/Seawater and Solar Salt Brine Treatment

Mixing wastewater with lime and with seawater produces a secondary standard of effluent and a well stabilised sludge. The lime raises the pH and magnesium in the seawater producing a rapid acting magnesium hydroxide precipitate.

This innovative process was originally developed by Mawson (1970), Simpson (1986) and Norwegian researchers including Vrale (1978), Ferguson (1984), and Odegarrd (1989).

The writer’s experimental work, research outcome by others, and the advantages of this treatment technique are reported in Portfolio 8. It is interesting to note that as early as the 1970s, Gunn (1970) reported on this method of treatment as having potential for bacterial and nutrient reduction. This was about the time Mawson (1970) reported jar trial results for this type of treatment.

The merits of the lime/seawater process, in terms of treating fluctuating hydraulic and organic loads, are summarised as follows:

1. The chemically assisted sedimentation requires smaller settling tanks and more efficient solids removal is achieved
2. The clarification process is rapid and positive
3. Higher pH levels give better responses to mixing, settling rates and effluent clarity.

The process is useful for transient communities that experience extreme flow variations, such as tourist areas and military camps (Shanableh et al, 1995).

2.3.21 Effluent Filters

Effluent filters are fitted to the outlets of septic tanks. They are very effective in preventing the overflow of solids from septic tanks. They are a more recent innovation in Australia but they have been operating in North America and New Zealand for many years. Typical effluent filters operating in Queensland include the following:

1. *ECOSAFE* – a vertical filter with some horizontal plates and a top screen to prevent solids and scum overtopping the filter. It is a fully closed system so odours are not emitted
2. *TAYLEX* - a filter cartridge, composed of a series of horizontal plates, is inserted into a PVC tee. Reported performance claims include up to 70% BOD_5 and up to 90% suspended solids reduction
3. *WATERTEC – Orenco* filters have been used in Queensland over the past decade.

It is now mandatory to install effluent filters in many South East Queensland Local and Regional Councils. Based on their very good performance in California, where the Orenco filter was developed some 35 years ago, and their performance in New Zealand over the past 20 years, the writer personally sees merit in their use in Australia.

Surveys on septic tank filters have been undertaken by Stafford and Whitehead (2005; 2007), as reported in Portfolio 5. These studies concluded that due to the variable performances more research should be undertaken.

3 Literature Review on Fluctuating Loads

The literature review covers the fluctuating load treatment alternatives in Section 2.3. The references include some prior to 1974, used in the writer's University of Auckland dissertation preparation, and more recent developments up until 2010.

3.1 Biofiltration

Biological or trickling filtration is the process whereby settled wastewater trickles over stones or plastic media covered in biomass. Single pass, recirculation or alternating double filtration are the modes of operation (Metcalfe and Eddy, 1972). Excess biomass sloughs off the media periodically and this material and the effluent passes into a humus tank for secondary settling.

Biological filtration is a very common method of wastewater treatment but must be adapted to cope with fluctuating loads. The ability of trickling filters to respond to shock loadings (hydraulic and organic) has been acknowledged in the literature, for example (Metcalfe and Eddy, 1972). However, the mechanism by which the trickling filter accomplishes this task had not been shown. Research was undertaken by Cook and Herning (1974) to study the effects on how sudden increases in substrate concentrations affect the substrate removal capability of fixed film bacteria. They came to the conclusion that in order to exhibit shock loading attenuation properties, the filter must be oversized to allow it to operate with the bulk of the organic removal in the upper zone. This maintains a viable bacterial population in the lower zone under starvation conditions which provided the shock load attenuation.

Nicoll (1988) reported that recirculation of effluent was a most effective way of increasing operational flexibility. It resulted in better wetting of filter media during the low flows.

3.2 Stabilisation Lagoons

Facultative lagoons offer a cheap, reliable and adaptable means of treating fluctuating loads (Gunn, 1970). The experience of Simpson (1974; 1983) demonstrated that facultative lagoons are particularly suitable for small communities, camping grounds and resorts as they are capable of handling fluctuating loads. Stabilisation lagoons can be designed so they adjust to fluctuating loads with a minimum of difficulty. The fact that they can remain relatively dormant during low flow periods means that the system is particularly suitable for holiday seaside areas (Asbey-Palmer, 1973).

High speed aerators can be used when lagoons are subjected to maximum loads and the aerators can be switched off during the low flow season (Echenfelder, 1970).

3.3 Aerated Lagoons

The aerated lagoon is a suspended growth reactor, within a lined basin with no sludge recycle. Aerated lagoons have a retention time of 2 to 6 days and a certain amount of nitrification is achieved (Qasim, 1994).

Oxygen is transferred to wastewater in a lagoon from mechanical or diffused aeration units and from surface aeration. Mechanical surface aeration units can be installed on a floating raft or on a permanent base. Aerated lagoons maximise the photosynthetic activity of algae and they are also known as high rate lagoons. In the absence of a clarifier the concentration of suspended solids in the effluent is high. Although the aerated lagoon is designed as a complete mixed reactor, a certain amount of settling does occur in different parts of the basin (Oasim, 1994).

3.4 Septic Tanks

Septic tanks will continue to have an application in isolated coastal areas. If they are correctly designed and operated, such systems can have a long and satisfactory life (Gunn, 1970). The writer feels that this statement has more grounding today due to the improvements made to septic tank design and the public appreciation for the need to desludge them on a needs basis.

3.5 Imhoff Tanks

Imhoff tanks can be designed to cope with wastewater from populations ranging from a few houses to approximately 4,000 people. As such, they are particularly applicable to the treatment of wastewater from holiday seaside areas (Asbey-Palmer, 1973). Imhoff tanks are still used occasionally because they are simple to operate and do not required highly skilled supervision (Crites and Tchobanoglous, 1998).

3.6 Extended Aeration

Biological treatment plants subjected to variable loads need to overcome the following problems (Barnes and Wilson, 1976):

1. Under feeding micro-organisms, leading to poor sludge characteristics and poor effluent quality under low loads
2. Able to “start up” quickly when loads increase

3. Be flexible enough to adjust to wide flow variations
4. Be capable of either closing down or functioning on low base loads.

Extended aeration plants are more suitable for installation at establishments where loadings tend to fluctuate moderately. Examples of such establishments are schools, motels, hotels, and camping grounds. There must be a constant base load and the load fluctuations should not be too great (MOW, 1979).

Nicoll (1988) reported that in diffused air plants varying the air supply was effective.

3.7 Sand Filters

Sand filters may be defined as a bed of sand and granular material which is under drained to collect and discharge final effluent (EPA, 1980). They are capable of operating under variable load conditions with no marked changes in effluent quality. The ECOSAFE filter in Queensland is very good at handling variable loads since the sand permeability is sufficiently high to allow constant downward velocities.

3.8 Granulated Bark Filters

Lightsey (1977) reported that bark filters used in a pulp mill effectively solved the functions of detoxification, absorbing shock loads and the substantial removal of heavy metals. Aged bark was considerably more effective. Fluctuating flow can be readily handled by a bark filter, for example, treating grey water which is generated on an irregular basis.

3.9 Upflow Pebble Filters

The efficiency of upflow pebble filters depends on the flow rates, influent quality, gravel size, gravel depth and the backwash frequency**.** Banks (1965) reported that 63% BOD_5 and 93% suspended solids removals. Best results are obtained when a sludge blanket has formed on the pebbles. Upflow pebble filters are capable of treating variable flows due to the combined processes of filtration, flocculation and adsorption. This is aided by simple “backwashing” by lowering the water levels and hosing the gravel.

3.10 Rotating Biological Contactors

RBCs function effectively under intermittent load conditions and will maintain steady state performance as long as low wastewater flow or effluent recycling is maintained (Antonie, 1970). RBCs are capable of handling a wide range of flows and no recycle is required (Benefield and Randall, 1980). The RBC is capable of producing a good quality effluent when treating wastewater from small isolated communities with a highly variable load, which can occur in small plants (Regent, 1980).

An RBC pilot plant responded well to large wastewater flow and strength variations in Pullman, Washington (Clark et al, 1978). Pullman housed about 66% of the students from the Washington State University so extremely low flows occurred during holidays and vacations, with a very rapid increase of flow when the students returned.

It was reported (DPI, 1992) that some 17 RBCs were operational in Australia, including five in coastal locations of central and northern Queensland, for up to 350 EP. Wastewater is pre-treated by a septic tank, Imhoff tank or a lagoon and then directed to an RBC.

During the 1980s there were problems with this treatment method. The discs tended to be too large and with the added weight of the biomass the shafts and driving systems failed.

RBCs are a more compact percolating filter and hence occupy less site area. Surface loading rates of five to six times that of percolating filters are possible. The bio-disc primary tank provides additional dilution for shock loads not available in biological filters (Lumbers, 1983).

3.11 Deep Shafts

Deep bores are a viable method of ongoing effluent treatment and disposal, and are capable of handling variable loads (Portfolio 7).

3.12 Constructed Wetlands

A study on the use of reed/gravel beds for treating wastewater from small communities in Hungary is reported by Todorovics et al (2005). They found that this technology was capable of handling grossly fluctuating loads and still met regulatory standards.

A two year study of four horizontal and vertical flow reed/gravel beds in tourist facilities with highly variable loads in Central Italy was undertaken. It was reported by Masi et al (2007) that the reed/gravel beds where easily maintained, robust, not sensitive to fluctuating flows, operated at relatively low costs and they utilised local material for construction. The details of four wetland projects are provided in Table 1.6**.**

Table 1.6: Reed/Gravel Beds Servicing Holiday Facilities in Italy

Parameter and Degree of Fluctuation	**Abetina (Reale shelter)**	**Baggiolino (Tourist farm)**	**Relais (Certosa hotel)[1]**	**La Cava (Camping ground)**
EP	100	30	140	80
Flow (m^3/d)	2.0-8.0	0.4-7.0	17.0-33.0	0.3-7.0
Degree of Flow Fluctuation	4 times	17.5 times	1.9 times	23 times
HF bed depth (m)	-	0.7	0.7	0.7
VF bed depth (m)	0.9	-	0.9	-

Notes:

1. The Relais Certosa hotel had a hybrid system which demonstrated the best treatment performance.
2. HF – denotes horizontal flow, VF – denotes vertical flow

It can be seen from Table 1.6 that the reed/gravel bed system is capable of handling very pronounced load variations, as shown by the Baggiolino and La Cava facilities.

A study was undertaken in the Cova da Beira region in Portugal on a wastewater treatment for 800 EP consisting of an Imhoff tank, and two parallel subsurface wetlands (Albuquerque et al 2009). The system responded to transient flows with organic matter, suspended solids and nitrogen. A good correlation was observed between mass removal rates and mass loads for COD and nitrogen compounds, which indicates that the beds had a satisfactory response to changes on incoming loads.

3.13 Compost Convertors

Domestic compost convertors have the ability to cope with both zero load and variable loadings (Portfolio 4).

3.14 Brush Filters

Smith and Woods (1992) report that low cost brush clarifiers can consistently achieve effluent suspended solids of less than 15 mg/L. Their installation is suited to peak or fluctuating loads. Brush filters in a humus tank at Coleshill, UK are shown in Plate 1.2. The brushes appeared to have a flocculating effect in addition to their filtration capacity.

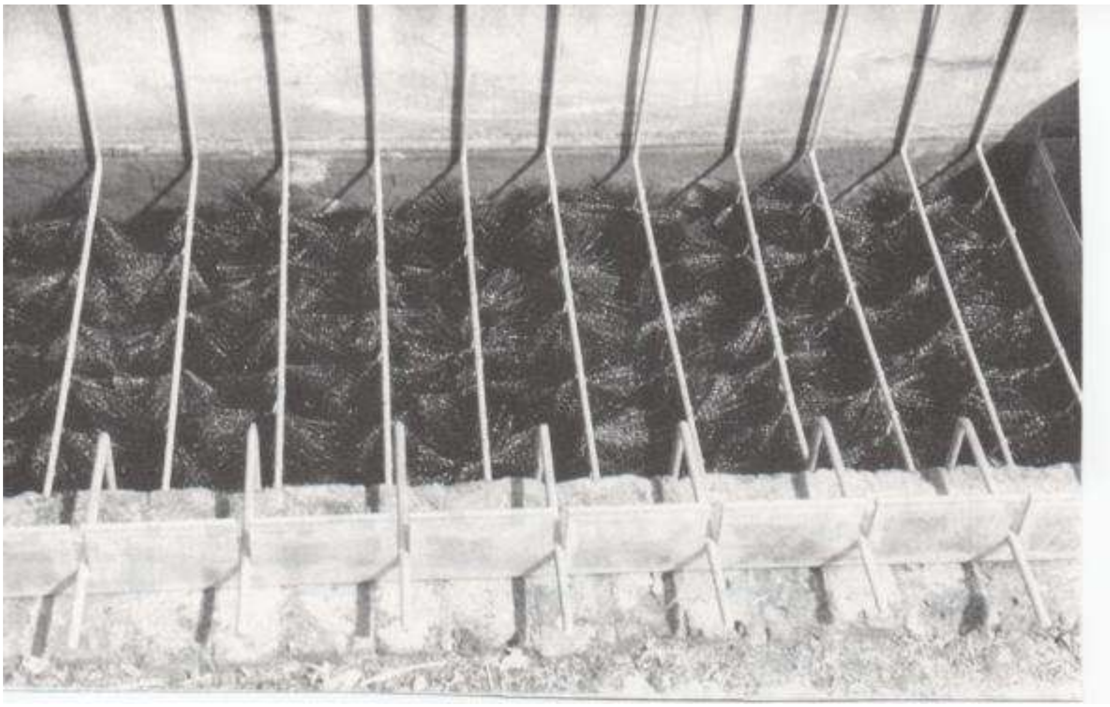

Plate 1.2: Brush Filters in a Humus Tank, Coleshill, UK.

3.15 Study of Small Biological Plants with Load Variations - Germany

The performance of small biological treatment plants with very pronounced load variations were studied in Germany and reported by Kegebein et al (2007). These systems included a trickling filter, a rotating biological contactor, and an activated sludge unit, and a moving bed biofilm system. The operation of the smaller units was ceased for some days. A 20 EP full scale trickling filter with a load varying from 40 to no hostel guests was also studied over a period of 85 days. The following aspects were concluded:

1. From a microbiological point-of-view, it is important to know the impact on the total biomass and its activity during idle periods
2. The loss of biomass significantly affects the treatment capacity
3. A parameter known as specific activity or oxygen uptake rate recovers within a reasonable time
4. Biomass decay does not start immediately after the feed ceases
5. The full scale plant demonstrated that residual substrates provide further feed for the bacteria, particularly when water is recycled through the system
6. A carbon source (sugar or methanol) may be added a week in advance of a system re-start to replace the loss of biomass by natural growth

7. Biomass loss may be compensated with excess sludge from a municipal treatment plant.

3.16 Septic Tank/Upflow Pebble Filter /Sand Filter System – University of Athens

A series of carefully selected unit processes can achieve very good effluent standards under variable load conditions. For example, the National Technical University of Athens experimentally evaluated a system consisting of a septic tank, upflow pebble filter followed by two submerged sand filters operating alternately (Christoulas and Andreadakis, 1989). Features of this system included:

1. The upflow pebble filter did not show any tendency to clog
2. The upflow pebble filter achieved a variable BOD_5 removal efficiency of 25% to 70% and SS average of 40% removal
3. The sand filters achieved an average of 75% BOD_5 removal and a SS average of 50% removal
4. It was felt that clogging of the sand filters may have been prevented by the adoption of two filters operating alternately
5. The sand filters achieved Total N removal of 40% to 45% by the action of adsorption of the NH_4 - N, oxidized to NO_3 - N during the resting period and denitrification about 85% of it during the subsequent flooding cycle.

The following criteria were adopted for the design and operation of the system:

1. The septic tank volume was twice the daily wastewater flow plus adequate space for sludge
2. The depth of filter media for the upflow pebble filter was 500 mm. The bottom 400 mm consisted of 12-18 mm gravel, the top layer consisted of 0.6 mm pebbles and the hydraulic loading of the upflow pebble filter was 2 m^3/m^2/day
3. The upflow pebble filter required backwashing by passing high pressure water downwards after two years of continuous operation
4. The loading rate of the sand filters was 1.5 m^3/m^2/day. The depth was 500 mm and the sand had a 0.6 mm diameter with a uniformity coefficient of 1.7
5. The design nitrification was 50% reduction.

The main advantage of this system was operational flexibility and simplicity. This included desludging the septic tank every two to three years, alternating the sand filters each fortnight, and raking the sand filters and replacing the top 15 mm of sand every year.

The system was capable of achieving BOD_5 of 20 mg/L and SS of 15mg/L under fluctuating load conditions. It was reported by Christoulas and Andreadakis (1989) that the sand filters functioned as anaerobic systems.

4 Reflections on Fluctuating Loads

In the context of this thesis, a reflection is a philosophical overview of a process, research project or a design project. In the case of this portfolio the benefit of hindsight has been utilised on projects undertaken many years ago.

4.1 New Zealand Fluctuating Load Experience

Marsden Point Stabilisation Lagoon System

Reflecting on this project, the writer advocates that the stabilisation lagoon system designed and constructed at Marsden Bay in Northland could have consisted of the following unit processes and layouts:

1. Rather than a single cell to have three lagoon cells, to cater for variable loads and expanded loads. This could also allow one cell to be de-commissioned, if loads reduced
2. Two open surface wetlands, planted with a range of emergent plant species. The maturation lagoon, also known as an algal pond, was originally installed to cater for bacterial reduction. The discharge of high algal content effluent has since become a problem in northern New Zealand and northern Australia. Based on Queensland research, open surface wetlands are very effective in filtering out algae
3. The original layout of the single cell was rectangular. There is scope for lagoon cells and wetlands in particular to be shaped aesthetically and landscaped.

4.2 Model Northland Coastal Community System – Waipu Cove

By looking back, a more viable method of treatment for the Waipu Cove, Northland situation would be as follows:

1. Primary treatment, within a number of oxidation cells, which could be commissioned and de-commissioned according to the prevailing wastewater load
2. Secondary treatment, in multiple open water wetland cells, containing macrophyte filters and deep water zones (to filter out algae)
3. Effluent disposal by spray irrigation in bush land and grassed areas.

4.3 Alternative Technologies Suitability Ranking Table

For the purpose of reflecting on fluctuating load alternatives suitability, a ranking table has been compiled in Table 1.7. This ranking table also shows the following:

1. An aid for comparing the viability of each fluctuating treatment option
2. A link between each option to the relevant chapter or portfolio of this thesis
3. Highlights the writer's research or project involvement on each treatment or disposal alternative.

The lowest suitability ranking is 1 and the highest ranking is 5. The extent of the writer's research and project involvement is given by the grading of (1) to (10), the highest involvement being (10).

Table 1.7: Suitably Ranking of Wastewater Treatment Options for Sea Coast and Lakeside Areas.

Technology Type	Widely scattered homes	30 - 200 persons	Suitability Ranking and Comments
Improved Septic Tanks **(5)**	yes	no	5 – Innovative modifications and configurations to septic tanks can enhance performance. Refer to Portfolio 4.
Imhoff Tanks **(2)**	no	yes	4 - Imhoff tanks are essentially a more efficient form of a septic tank and are more suitable for >20 persons.
Bio-filters **(2)**	no	yes	3 – Offers flexibility for handling variable hydraulic and organic loads. Refer to Portfolio 1.
Stabilisation Lagoons **(4)**	no	yes	4 – Can handle flow variability. They have tended to replace Imhoff tanks in New Zealand and Australia. Problems with high algal content effluents – wetland filters have high potential – refer to Portfolio 1.
RBCs **(2)**	yes	yes	3– Considerable potential but more localised experience needed to instil confidence in this system – refer to Portfolio 1.
Open Surface Wetlands **(9)**	no	yes	4 – Viable option if land is available.[1] Refer to Portfolio 3.
Lime/Seawater **(10)**	no	yes	4 – Very good potential, as demonstrated by full scale plants in Norway. Refer to Portfolio 1 and 8.
Reed/Gravel Beds **(8)**	yes	yes	5 – Viable option and compact with a high treatment rate. Refer to Portfolio 3.

Composting Toilets **(9)**	yes	no	4 – More suitable in semi-rural areas, National Parks, and camping grounds. Refer to Portfolio 4.
Upflow Pebble Filters (anaerobic) **(3)**	yes	yes	4 – Very good potential for primary influents. Refer to Portfolio 5.
Upflow or Banks Filters (aerobic) **(2)**	no	yes	3 – Simple and robust system with very good performance potential. Refer to Portfolio 5.
Aerated Lagoons	no	yes	4 – Offers loading and operational flexibility. Refer to Portfolio 1**.**
Extended Aeration Package Plants **(6)**	yes	yes	3 – Compact system and capable of some flow variability. Refer to Portfolio 1**.**
Sand Filters **(3)**	yes	yes	4 – Offers flow variability and very good performance results.
Bark Filters **(2)**	yes	yes	3 – Probably approach the potential of peat filters, which have been proven in Australia. Refer to Portfolio 1.
Septic Tank/Upflow Pebble Filter and Sand Filter	no	yes	4 – Very good performance potential with minimal operational needs.
Brush Filters	no	yes	3– Worthy of more localised trials. Refer to Portfolio 1.
Tube Settlers and Lamella Plate Separator **(3)**	yes	yes	4 – Simple, compact and effective system. Refer to Portfolio 1.
Deep Shafts **(9)**	yes	yes	5 – Offers flow variability and storage. Refer also to Portfolio 7.

Notes:

1. A large scale experimental open surface wetland in Ingham, for treating primary effluent, underwent widely fluctuating loadings for several weeks due to a breakdown of the trickling filters. The performance of the wetland systems, not designed for a full plant loading, showed no adverse reaction to a large increase in hydraulic and organic loading
2. Ranking applies to domestic wastewater from individual and smaller coastal and lakeside communities
3. Economics are not necessarily considered for this evaluation
4. Marine outfalls have **not** been considered as an option
5. This suitability ranking is intended to be a general guide only. A full study of site characteristics, population and projected flows, treatment options and economics should be undertaken.

4.4 Lime/Seawater Substitutes Treatment

On reflection, seawater substitutes, as bittens and salt waste, can be used in the absence of seawater (Simpson, 1986; Shanableh et al, 1995). This also means that lime/seawater substitute treatment can be applicable in inland areas where the disposal of high saline effluents is not acceptable. This would also enable irrigation reuse.

5 Conclusions on Fluctuating Loads

In the context of this thesis conclusions are the outcomes of a case study, research and a design project. The management of fluctuating wastewater loads is a unique problem since it presents the designer with the tasks of handling both hydraulic and organic variations.

A literature review, undertaken for a Post-graduate Diploma in Public Health Engineering studies (Simpson, 1974), regarding alternative treatment technologies for handling fluctuating loads, has been summarised and updated with more recent developments and the alternatives ranked in terms of suitability. A review of some more recent developments in the treatment of fluctuating loads has been documented.

Suitability Ranking

It can be seen from the suitability ranking table in Table 1.7 that the most viable treatment alternatives for coping with fluctuating wastewater flows are:

1. Improved septic tanks, for up to 20 persons
2. Imhoff tanks for larger flows
3. RBCs (can handle load variations up to about four times, but they can operate as a modular system to cater for larger load variations, based on New Zealand, Fiji and other overseas experiences)
4. Stabilisation lagoons (the writer concurs with Gunn (1970) that lagoons are a cheap and adaptable alternative for handling variable loads, based on New Zealand experience and this remains the case today)
5. Reed/gravel beds, for smaller flows, largely due to the cost of aggregate.
6. Sand filters, which are very adaptable to variable loads, based on Queensland experience
7. Upflow pebble filters (anaerobic)
8. Deep shaft disposal, in situations with deeper ground water levels and away from steeper land slopes, based on New Zealand experience
9. Composting toilets, for lower individual houses and flows to an equivalent of about three houses, based on New Zealand and Australian experience. The writer has seen and used multiple units

functioning in the British Columbia Rockies area, in cases where several buses may call at the same time.

The use of a lamella plate separator has much promise for reducing BOD_5 and suspended solids in domestic effluent. The use of granulated bark filters to reduce heavy metals, toxicity and buffer fluctuating loads, also has much promise.

Brush filters are capable of handling loading fluctuations, particularly for producing effluents with low suspended solids. They also have the advantages of being readily installed, easily maintained and have no moving parts. This technology is worthy of more experimentation on a full scale basis in Australasia.

Queensland Fluctuating Load Experience

For individual dwellings, camps, caravan parks and small communities probably the most viable alternative for handling variable flows is the primary tank with an effluent filter, sand filter and spray irrigation area. Another alternative is a primary tank with an effluent filter and reed/gravel beds.

Arising from the Watson Park Convention Centre experience (Portfolio 6 Part B), another alternative is stabilisations lagoons, vetiver grass overland flow plots and spray irrigation. The above viable alternatives are listed in Table 1.2.

Improved Septic Tanks

The writer has followed the performance of septic tanks since 1974 (Simpson, 1974). It can be seen from Table 1.7 that the writer has ranked improved septic tanks at a high 5.

If well-designed septic tanks with sufficient capacity and with double or multi-chambers and outlet filters are used, they are a very effective option for handling fluctuating loads. In discussions with Newcastle University this high ranking of septic tanks was agreed by Joe Whitehead (pers. comm., February, 2010.

Septic tank/Upflow Pebble Filter/Sand Filter System - University of Athens

The writer advocates that the overall performance of this potentially capable system for treating variable wastewater loads from coastal areas could be enhanced by the following measures:

1. Having a third compartment in the septic tank component
2. Installing inclined tube settlers in the outlet of the second compartment (refer to Portfolio 5)
3. Increasing the depth of the upflow pebble filter
4. Changing the function mode of the alternating sand filters to an aerobic environment by installing air vents.

It is also the writer's view that near full nitrification would be achieved within the treatment train and it is possible that de-nitrification would take place within the anoxic and anaerobic conditions in effluent disposal system.

6 Portfolio References

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Portfolio 2 - Soil Permeability, Testing and Soil Treatment Mechanisms

1 Introduction, Aims and Technology Background

Receiving soil and water environments must be capable of accepting treated effluent on a long-term basis. In order to achieve sustainable on-site wastewater treatment with minimal impacts on the environment and public health, more appropriate means of assessing the long-term performance of on-site dispersal systems are required.

The overall aim of this Portfolio is to review methods of measuring soil permeability and associated effluent disposal mechanisms, ensuring where possible, that they contribute to sustainable systems. To achieve this overall aim, the portfolio structure is as follows:

1. Literature review
2. A brief review of permeability and soil properties that are amenable for effluent disposal
3. Research by Simpson on effluent trenches, beds and spray irrigation, including a case study of the Tiaro Motel, Queensland
4. Reflections and needs for research and development
5. Conclusions.

Saturated hydraulic conductivity or permeability is defined as a measure of the ability of a soil to transmit water (OSSM, 1998). The hydraulic capacity of the soil is also defined as "the volume of water that can be continuously infiltrated into the soil without raising the groundwater level above selected levels" (Geary and Gardner, 1996).

The disposal of effluent into the soil principally depends on soil drainage. The drainage potential is influenced by soil characteristics, climate, contours, surface runoff, evaporation and transpiration **(**Carryer, 1977). These factors and the writer's extensive experience in Northland, New Zealand and Queensland soils, demonstrate that they do influence shallow disposal systems. It is however felt that some consideration to these limitations has been covered by the factors of safety that are built into AS/NZS 1547:2000 (pers. com. Peter Beavers formerly of the AS/NZS 1547:2000 Standards Committee).

The spray irrigation of treated effluent on grassed surfaces has the advantage of potential disposal by evapotranspiration (ET), and by soil infiltration. ET is covered in more detail in Portfolio 6 Part A.

The hydraulic conductivity (Ksat) is influenced by the soil and hydrogeological conditions. Ksat is more limiting in soils of low permeability or in situations with a shallow depth. If the design effluent loading exceeds the hydraulic capacity of the soil, the effluent system will fail. Consequently, one of the most important issues regarding appropriate use of on-site systems is the proper assessment of the site and soil characteristics which play such an important role in the treatment and dispersal of effluent (Carroll, 2005). In the past the hydraulic capacity of soils has been assessed by ad-hoc methods and falling head percolation testing.

The writer had reservations about the earlier focus on falling head testing and, as a consequence, investigated other soil Ksat evaluation methods such as the double ring infiltrometers and constant head laboratory perimeter testing. The falling head percolation test is not to be used as the sole criteria for effluent design. It is simply a means of comparing the percolation capacity of various soil types and structure, and it is only one component of the evaluation process (Gunn, 1994).

More recently soil Ksat values have been calculated from constant head percolation testing, as required by the Australia and New Zealand joint standard, AS/NZS 1547:2000. The falling head test is not recommended in the joint standard due to the lack of a proper physical model, the lack of a mathematical description of the falling head method in an unlined test hole and the resultant uncertainty in converting a measured fall rate into a Ksat value.

Papers by Simpson Appropriate to Portfolio 2

1. Beca Carter Hollings and Fernery 1976, Report on Effluent Disposal and Natural Ground Stability Titirangi Park Estate Residential Subdivision, for Titirangi Park Estates Ltd, November (report as an appendix).
2. Simpson, JS 1978, Candidate's Report for Professional Interview Conducted by The Institution of Public Health Engineers (UK).
3. Simpson, JS 1984, The Evaluation of Sites and Alternatives for On-site Effluent Disposal, *New Zealand Journal of Plumbing and Drainage Inspectors.*
4. Simpson, JS 1986, The Evaluation of Sites and Alternatives for On-site Effluent Disposal, *New Zealand Journal of Environmental Health.*
5. Simpson, JS 2003, Report on On-site Wastewater Treatment and Effluent Disposal – Proposed Tiaro Motel Complex, Tiaro, July.

2 Research by Simpson and Associated Researchers

As mentioned in Section 1 the writer earlier researched alternative K testing using the double ring infiltrometer and constant head laboratory permeameter.

The writer appreciated that previous codes and standards in New Zealand did not adequately deal with effluent site evaluation methods and, as a result, developed the following (Simpson, 1986):

1. Adapted versions of soil permeability testing methods
2. Simple matrix-based methods to evaluate site suitability and soil requirements for effluent disposal

3. An understanding of the capability of soils to treat effluent that is unaccounted for in conventional effluent bed and trench design.

Over the past decade the writer has designed many droplet-type spray irrigation systems in South East Queensland. He has developed a methodology for site evaluation and design and for determining whether the upper soil depth is sufficient for long term surface irrigation.

2.1 Soil Permeability Testing Methods

It has been established, from field and laboratory investigations, that many soil types can be capable of accepting different quality effluents provided the application or infiltration rate does not exceed the soil permeability. This key statement has also been the writer's own experience.

Particle size, particle size distribution, soil particle arrangement, organic matter content, iron oxide content, clay mineral composition, exchangeable sodium percentage and the total salts concentration are among the more important factors affecting soil permeability (Horn, 1971).

Before various infiltration testing systems are discussed, it is appropriate, based largely on the writer's field experience, to briefly discuss the following associated aspects:

1. Trench and bed infiltrative surfaces
2. Soil interface clogging
3. Merits of alternating the dosing of trenches and beds
4. Trench and bed configurations
5. Effluent distribution options.

2.1.1 Falling Head Percolation Field Testing

Earlier work focused on falling head percolation testing. The writer followed a trend in North America during the 1970s to improve the determination of soil K values by experimenting with infiltrometers and permeameters. The current method of soil properties and determination of Ksat values focuses on an assessment method as given in AS/NZS 1547:2000.

Previously, designs in New Zealand, Australia and the USA were reliant on basic falling head testing simple percolation testing to evaluate site suitability. The falling head test is conducted by drilling 100 mm diameter hole, 450 mm deep in the upper soils, filling the hole with a 300mm depth of water and recording the water level falls within a specified time.

A diagram of a falling head test set up is in Figure 2.1. During the 1970s, the writer was reluctant to use the standard falling head type percolation test, particularly as the sole criteria for design effluent disposal systems. This unfortunately was the practice of many local authorities in New Zealand, Australia and the USA.

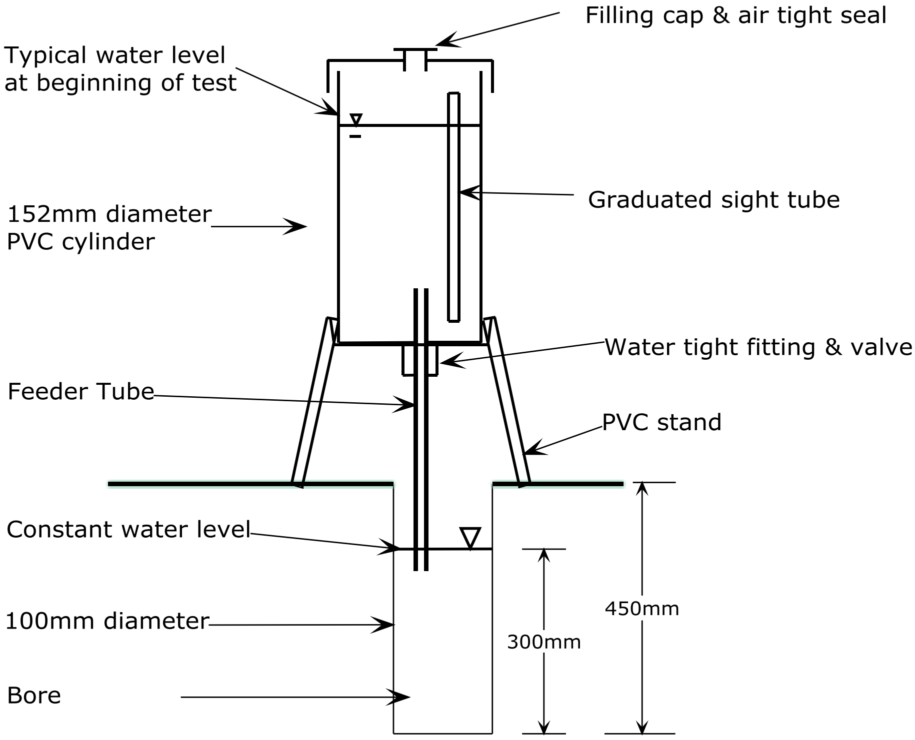

Figure 2.1: Falling Head Percolation Test Setup

Researchers have shown that that the percolation rate on its own can be misleading and does not directly measure any soil characteristics that could be used in the design of sub-surface effluent systems, reported by Dawes (2006), based on Healy and Laak (1974) and Gunn (1989).

As already stated Gunn (1989) rightfully reports that percolation test results must not be used as the sole criteria for designing trenching and beds.

The writer conducted many falling head percolation tests within Northland, New Zealand as it was required in accordance with the New Zealand Standards Institute Code of Recommended Practice for the Disposal of Effluent from Household Septic Tanks (CP 44:1961).

As an example, the writer conducted some falling head percolation testing on a land development project at Whangarei Heads, Northland. Refer to the results in Table 2.1.

Table 2.1: Percolation Test Results and K Values – Whangarei Heads, NZ

Date	**Soil Description**	**Fall in last 10 minutes** (mm)	**K sat value** (m/d)	**Comments by Simpson** (based on experience and AS/NZS 1547:2000)
7/12/76	Topsoil on white clay	22.54	0.194	This is a low figure
7/12/76	Topsoil on yellow crumbly clay	12.49	0.325	This is a low figure
12/1/77	Brown loam with broken shells	65.42	1.56	This is a typical figure for loam
12/1/77	Rich black topsoil on shelly yellow clay	26.46	0.70	This is a typical figure for loam
12/1/77	Grey/ blue/yellow clay	9.80	0.244	This is above the figure expected
7/12/76	Topsoil on white clay	11.52	0.174	This is a low figure

Notes:

1. Recorded falls in last 10 minutes from falling head testing, as a design basis
2. Ksat values calculated from formula from Imperial College, London (refer to the Section 2.1.2)
3. Testing water was not a drinking water standard

Based on the falling head tests and their results, the following aspects from Table 2.1 are to be noted:

1. Falling head tests are no longer used as a standard in New Zealand and Australia, since they have found to be inappropriate on a scientific basis, as already discussed
2. The percolation results for topsoil on white clay and topsoil on yellow clay have impacted and reduced the calculated K values
3. Based on AS/NZS 1547:2000 the K value of 0.7 m/d is representative for topsoil and the K value of 1.56 m/d is representative for a good quality loam.

The writer concludes that the Ksat value, calculated from a falling head percolation test, can be influenced by the available depth and nature of the topsoil and the clay content of the underlying soils.

2.1.2 Constant Head Field Testing

From the writer's extensive field experience in South East Queensland conducting constant head percolation tests, in accordance with AS/NZ 1547:2000, this method has a minor limitation as the test hole is restricted to a depth of 450 mm. Of this 450 mm the depth of soil tested is limited to the water depth of 300 mm.

The test hole depth limit of 450 mm does not consider the potential of deeper soils for accepting effluent, i.e. the soil profile around the 500 to 750 mm depth. The requirement of a 450 mm depth also eliminates the potential for testing shallower soil profiles. As previously mentioned, in Northland, New Zealand a clay pan often exists at 300 to 500 mm, which often eliminates the application of shallower effluent trenches and beds and the applicability of standard constant head testing.

The constant head percolation method of assessing the soil K and effluent loading rates (as prescribed in AS/NZS 1547:2000) for trenches, beds and spray irrigation has merit. Extensive experience is required to match the soil types and mixes with the assessed K and effluent loading rates. Constant head percolation testing, in accordance with AS/NZ 1547:2000, can be difficult to undertake in the case of deeper trenches and disposal bores.

The constant head test superseded the falling head test since it was claimed that it was a more technically sound and representative indicative method. The assessed rate is determined by field observation, the clay content, the soil texture and soil structure, as listed in Tables 4.2A1, 4.2A2, 4.2A 3 and 4.2A4 in AS/NZS 1547: 2000.

2.1.3 Single and Double Ringed Infiltrometer Field Testing

A single ring infiltrometer is used for measuring the saturated soil hydraulic conductivity of a soil layer. A single ring or cylinder infiltrometer is sometimes favoured because the technique is simple, inexpensive and the equipment is hand portable (Hills, undated).

A double ringed infiltrometer is also used in measuring saturated hydraulic conductivity of a soil layer, and it consists of inner and outer rings inserted into the ground. The hydraulic conductivity, or Ksat, can be determined for the soil when the water flow rate in the inner ring is at a steady rate. It works by directing water onto a known surface area due to the perimeter of the inner ring. Infiltration can be measured by either a single or a double ring infiltrometer, with preference usually for the double ring because the outer ring helps reduce the error that may result from lateral flow in the soil (Miller, undated). The writer has experienced the buffering effect of the outer ring, particularly in soils that were dryer initially. Problems with double-ringed infiltrometers, Miller (undated) and McKenzie et al, (2002) with comments based on the writer's own experience, are listed in Table 2.2.

Table 2.2 – Problems and Comments on Double Ringed Infiltrometers

Topic	**Problem**	**Simpson experience and comments**
Time	They are claimed to be very time consuming, requiring frequent attention (Miller, undated).	The writer found that this was not the case. One tends to get into a system of testing steps.
Practicality	Practicality on the instrument is reduced since the rings are extremely heavy to move (Miller, undated).	The writer's double ringed infiltrometer was constructed from galvanized plate and it was not heavy or difficult to move.
Rates variation	Infiltration rates vary with different soils, which have an effect on the results (Miller, undated).	It is true that different soils have different results, hence, the statement does not appear to make sense.
Vertic soils (soils with cracks)	Double ringed infiltrometer not suitable for soils with vertic properties (McKenzie et al, 2002) since they are noted for their shrink/swell properties, which results in cracks when dried.	In the writer's experience, the double ringed infiltrometer was more suitable for sandy and loamy soils and light clays.

It can be seen from Table 2.2 that double ring infiltrometers are not suitable for vertic or cracked soils. Should the need to test vertic soils arise, the use of the Guelph permeameter should be considered. Based on work with vertic clay soils in Sicily, relatively short mean equilibrium times were obtained for the permeameter measurements (11-42 minutes). This suggests that the Guelph permeameter can measure the soils field saturated hydraulic conductivity before wetting causes enough swelling to reduce the permeability (Bagarello et al, 1999).

It is of interest that the University of Newcastle undertook testing in shallow duplex soils in Nimbin, NSW with a degree of success, with a Model 2800 K1 Guelph Constant Head Permeameter (Geary, 1992). It is also interesting to note that hydraulic conductivity was tested in compacted clays in Texas using a range of infiltrometers (Daniel 1989):

1. Open single ringed
2. Open double ringed
3. Closed single ringed
4. Sealed double ringed.

Once again it was found that the most versatile infiltrometer was a double ring type, which, in this case, was sealed.

The writer reviewed the paper on single ring or cylinder infiltrometers by Hills (undated). He appreciated the potential problem of disturbance of the upper soils during placement of the cylinder and consequently selected to work with a double ring infiltrometer. The writer researched the method of determining infiltration rates, using an open double ringed infiltrometer with the assistance of the University of Waikato, New Zealand in the mid-1970s. The double ringed infiltrometer was inserted into the soil to a depth of 50 mm. The rings were manufactured from rolled steel which was galvanized. The internal ring was 230 mm radius and the external ring was 350 mm radius. There was a supporting frame over the rings, which held a graduated feeder bottle with feeder tubes. (Correspondence from Selby, 1975)

Refer to Figure 2.2 for a diagram of the writer's double ring infiltrometer.

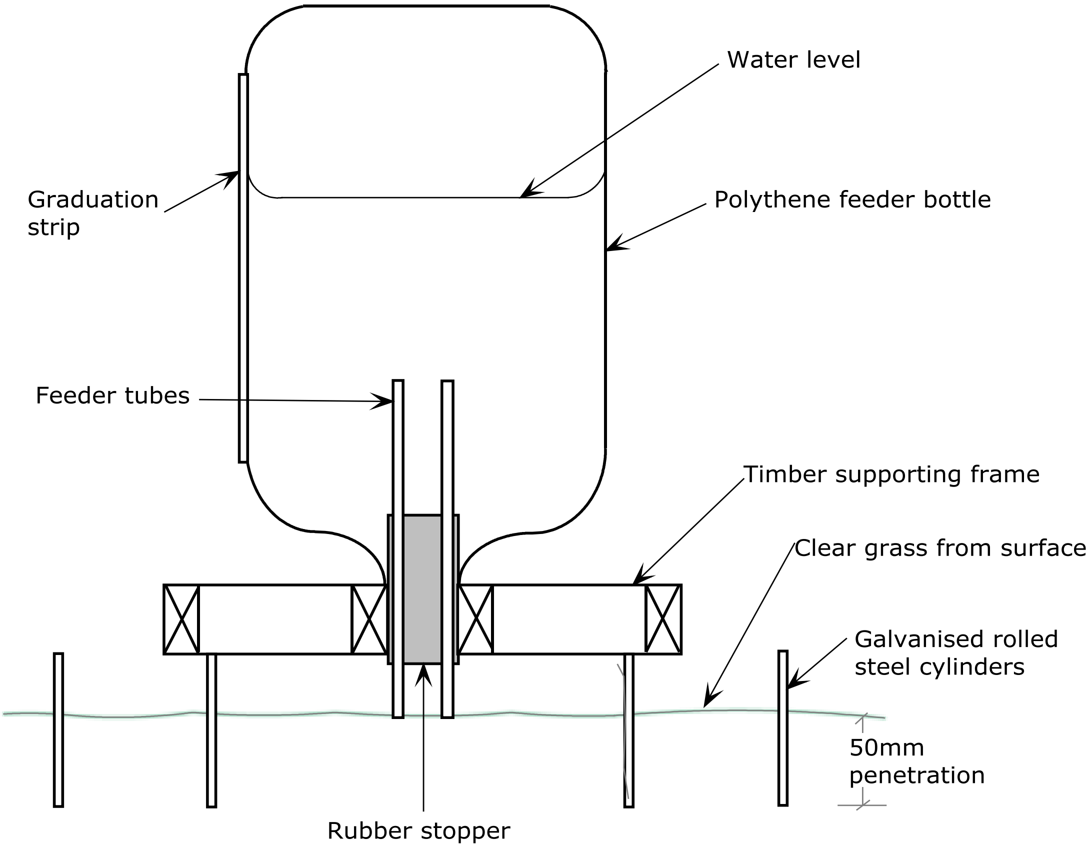

Figure 2.2: Double Ring Infiltrometer

Selby (1975) noted that the only problem experienced by the University of Waikato was placing the rings into the soils. He used a piece of heavy timber across the top of the rings and a large sledge hammer to drive the rings into the soil. The writer overcame this difficulty by using the timber and sledge hammer but also placing the rings in drier soils after pre-soaking the test area.

Hills, (undated) described the rings method as popular because the technique is simple, inexpensive and the equipment is hand portable. However, a lack of experience in the use this method could lead to misleading results.

It was found that some soils (e.g. peat) had a low soil infiltration rate after longer dry periods. By plotting infiltration with time, suitable pre-soakage and testing periods could be determined for various soil types with different field moisture contents.

Hills (undated) claims that some errors can be minimised by careful use and sensible sampling design. This is the approach the writer followed and, based on other methods of determining soil K values, he achieved meaningful results.

With the writer's on-going testing experience, and based on the upper soil characteristics, he found that it was possible to predict infiltration rates for a particular soil under near saturated conditions to a reasonable degree of accuracy. It was the writer's intention to graph the measured against predicted rates for predominant soil types. This is, however, not now possible since the results have been destroyed. In the writer's experience the double ring infiltrometer is a suitable method for testing in sandy and loamy soils, rather than clays.

Ahuju et al (1976) worked with a double ring infiltrometer on silty clay loam in Hawaii. It was found that lateral flow decreased with time during infiltration. Under the moist to wet soil conditions, lateral flow was not appreciable. Using an inner ring of 300 mm diameter, lateral flow was practically eliminated when a buffering or outer ring of 900 mm was used. Lateral flow was also negligible even when a buffer ring of 600 mm diameter was used. This is one confirmation that the writer made the right selection in 1975-1976 of using a double ring infiltrometer.

Double ring infiltrometers have been used in New Zealand to determine saturated hydraulic conductivity (Ksat). It is understood that if irrigation rates are required, the current thinking in New Zealand is to move away from Ksat and look at unsaturated hydraulic conductivities (K-40) (Rob Lieffering pers. comm. Northland Regional Council, New Zealand).

A report on the use of double ringed infiltrometers has been made by Youngs (2006) who found that they provided a simple method of investigating soil structural changes near the surface. Large variations in hydraulic conductivity values were found from experiments with smaller sized rings (around 150 mm diameter), but little variation was found for large sized rings (600-900 mm diameter).

Barnett and Ormiston (2010) have found, in the lower North Island of New Zealand, that constant head testing is not well suited to soil testing for shallow compensating dripper irrigation systems. The double ring infiltrometer provides a more reliable assessment of the surface zone into which treated effluent is applied by shallow systems with surface and subsurface pressure compensating dripper systems.

2.1.4 Deep Effluent System Field Testing

Following some years of field testing in New Zealand and Queensland, it soon became apparent to the writer that it was difficult to conduct tests strictly in accordance with the code, particularly in the complex clay soil types in the Northland

Province of New Zealand. It was often not feasible to locate suitable and more permeable material within the upper horizon. As mentioned earlier, many of the clay upper soils contained characteristic clay pans which inhibited soil infiltration. Deeper test bores were required and it was not practical to conduct the standard falling head percolation tests in the deeper bores. This was largely due to the higher volumes of water that must be taken to the sites.

Projects with site constraints; such as low permeability upper soils, unsuitable terrain and un-favourable climatic conditions, require special attention in terms of investigation and design. Because of the writer's knowledge of deep shaft disposal, as reported in Portfolio 7**,** he was a specialist advisor to *Beca Carter Hollings and Ferner*, New Zealand's largest engineering consultancy, for effluent disposal investigations and reported on a subdivision of 49 lots, varying in area from 2,000 to 10,000 m^2, in Titirangi near Auckland (Beca, 1976).

The Auckland Regional Water Board would not allow conventional shallow trenching and shallow pits, due to the clay nature of the upper soils. Evapotranspiration (ET) systems were also ruled out due to the high rainfall and the difficult terrain. Site drilling established that a sufficiently permeable stratum, of unweathered Manukau Breccia, was available for disposing effluent into deep shafts, on each lot. Percolation testing was undertaken by the following two steps:

1. Each shaft was filled with water from the town supply and this was maintained for four hours
2. A constant head test was run for at least 30 minutes with the water level being maintained for at 300mm below ground surface level.

From extensive field experience in a range of soils, together with an understanding of groundwater hydrology and soil science, the writer has developed systems whereby infiltration rates can be calculated and applied to effluent field design by:

Step 1 - Undertaking percolation tests in accordance with the New Zealand Standards Institute Code of Recommended Practice for the Disposal of Effluent from Household Septic Tanks (CP44:1961).

Step 2 - It was established earlier that the design percolation rates can be applied to a formula (derived from notes taken at the Imperial College, London) and the available soil K value calculated. The permeability formula is:

$$K = \frac{r^2 \Delta h}{h^2 \Delta t} \cdot 2.3 \log({}^{h}/_{r}) \quad \text{(cm/s)} \qquad \textit{Equation (1)}$$

Where: r = test hole radius (cm), Δ h = drop in water level in delta t (cm), h = average depth of the water and Δ t = time (minutes)

A K value in units of m/d is calculated. To allow for a factor of safety, taking into account the effluent quality, an assumed design K was selected from the calculated

K. For example, using a primary standard of effluent the design K was about 70% of the calculated K.

This approach was unique at the time since it was technically better to apply the K formula to percolation test results, than to apply the percolation test result direct. The writer used this approach on many occasions and at times with a colleague Simon Carryer, who specialised in engineering geology, effluent field design and groundwater studies. It was concluded that the above approach was in the right order when compared with other methods available at the time, such as Winneberger (1974).

A falling head modification of the code of practice proved successful when applied to test holes deeper than about 1.20 m. In many of the Northland soils the deeper soil profiles showed alternating impermeable and more permeable strata. The falling head test fully evaluates the seepage potential of the sub-soils, over the full depth of the hole. By graphing the fall in hydraulic head with time in Figure 2.3 it can be seen when the test results start to level out – the basis for design. The test water was a drinking water standard.

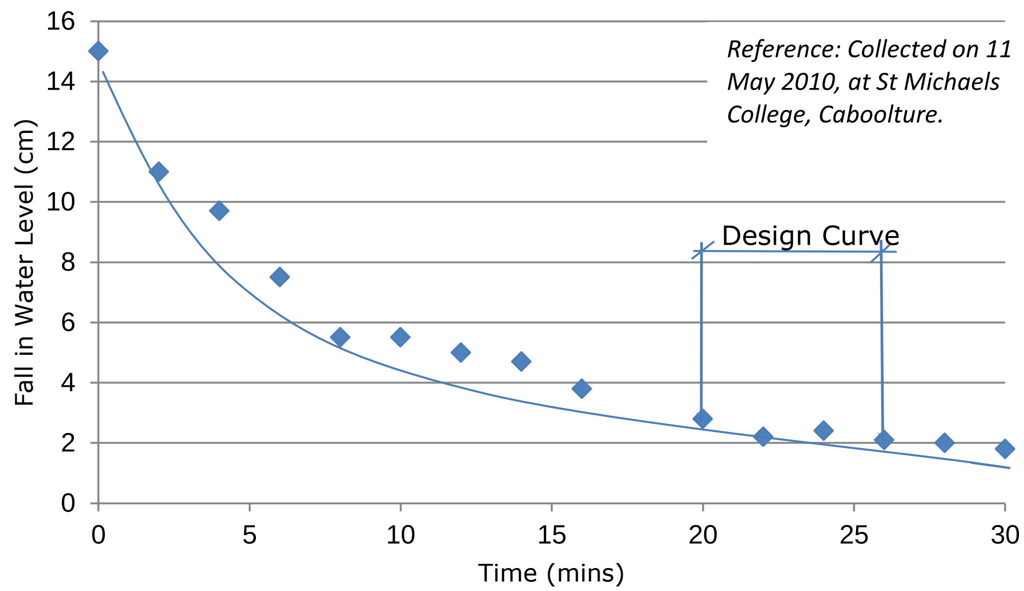

Figure 2.3: Plot of Falling Head Percolation Test Results

It is to be appreciated that effluent fields will at times operate under a hydraulic head, particularly with slower permeable soils. This operating condition can help alleviate soil pore clogging, which is a real benefit, particularly with anaerobic effluents.

2.1.5 Constant Head Laboratory Permeameter Method

The writer undertook comprehensive constant head laboratory testing using town water on a range of soils in Northland, New Zealand over the period 1976-1978. This involved collecting field samples of undisturbed soils and testing them under constant head conditions in a small laboratory permeameter. A diagram of the writer's constant head laboratory permeameter is in Figure 2.4.

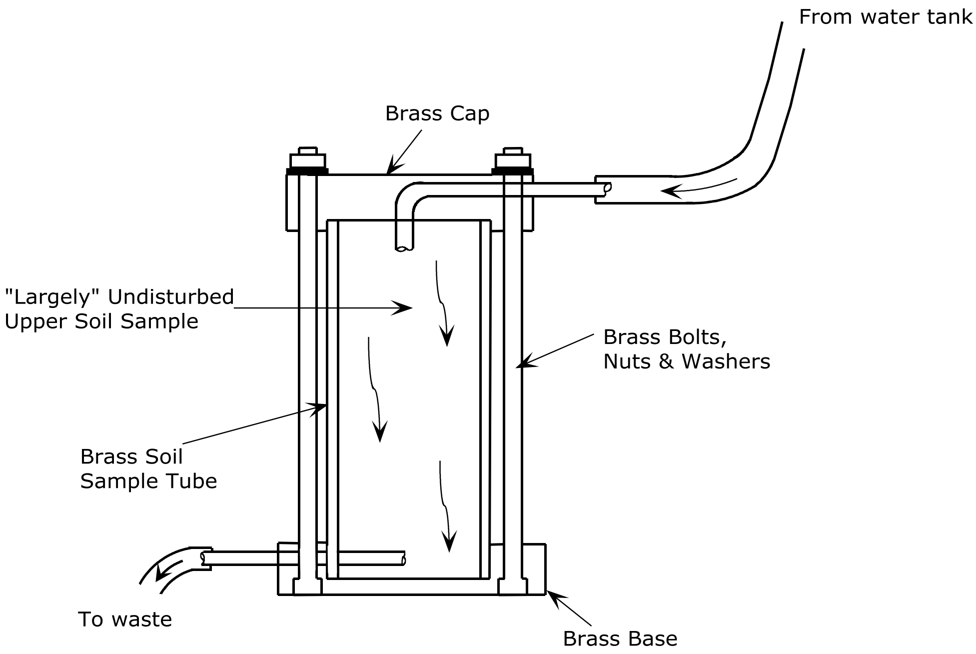

Figure 2.4: Constant Head Laboratory Permeameter

The writer found that this constant head laboratory permeameter was very useful for testing more permeable soils, with a K value of greater than 0.4 m/d. From experience, the following factors are important when using this laboratory method:

1. Representative in-situ soil samples must be taken with care. Greasing the sample tubes made sample collection easier
2. It was noted that results had a definite pattern. Erratic results indicated that air could be trapped within the soil sample or that there was a leak
3. It was important that the head remained constant.

The constant head laboratory testing formula is:

$$K_{20} = \frac{(R_1 - R_2)}{Tt} \times \frac{1.9655L}{60AH_c} \quad \text{(cm/s)} \qquad \textit{Equation (2)}$$

Where: K_{20} = permeability at 20 degrees C, T = temperature correction factor =1.10, Hc = head constant (cm), A = tube area (cm^2), t = time (5 minutes), $R_1 - R_2$ = drop in grad. Cylinder (cm) and L = sample length (cm)

2.2 Interpreting Permeability Test Results

2.2.1 Assessed K Values versus Measured K Values

Since the introduction of AS/NZS 1547:2000 the writer has often compared assessed Ksat values with calculated Ksat values. The assessed K values are listed in AS/NZS 1547:2000. The calculated K values are derived from constant head percolation testing results and then applied in a K formula in AS/NZS 1547:2000. Often they were within a similar range. On occasions the calculated K was lower than the assessed K. Table 2.3 contains some example results that the writer has recorded in the Caboolture area, over the period 2002 to 2009. The test water was from the town supply.

Table 2.3: Assessed Soil K values and Calculated K values for sites near Caboolture, Queensland

Soil Type	**Assessed K** (m/d)	**Calculated Ksat** (m/d)	**Soil Moisture and Comments**
Sandy loam	1.5 - 1.7	0.45	Moist
Sandy loam	1.8 – 2.2	1.23	Slightly moist
Sandy loam	1.8 – 2.2	1.6	Slightly moist
Sandy loam	1.5 – 1.7	0.57	Moist
Sandy loam	0.7	0.31	Dry, drought conditions[1]
Sandy clayey loam	0.2	0.14	Moist
Sandy clayey loam	0.5	0.135	Dry
Clay loam	0.2	0.08	Dry, drought conditions[1]
Clay loam	0.45	0.6	Dry
Clay loam	0.2	0.084	Dry, drought conditions[1]
Clay loam	0.3	0.27	Slightly moist
Sandy loam	0.9	0.48	Moist
Sandy loam	1.25	1.3	Moist
Sandy loam	1.2	0.9	Slightly moist

Sandy loam	0.65	0.4	Moist
Sandy loam	1.5	1.38	Moist
Sandy loam	2.0	2.8	Moist
Sandy loam	1.6	0.7	Slightly moist
Sandy loam	2.8	4.0	Moist
Sandy	2.5	2.8	Moist
Loam	0.75	0.6	Moist
Loam	1.9	2.5	Slightly moist

Notes:

1. During drought conditions the soils repel moisture, which inhibits infiltration, unless the soils are soaked for at least 12 hours. This condition was described by the former Soil Bureau, DSIR, NZ but the correspondence has been lost.

It can be seen from Table 2.3 that soil moisture conditions do impact on the soil K values. A moist soil produces a lower K value. A slightly moist soil generally gives a soil K similar to that assessed. Very dry soils give lower K values than assessed values. The writer advocates that some field experience is required to assess K values, within a range of soil categories and moisture contents, to a reasonable degree of accuracy. The moisture contents are those noted at the time of testing. The original standard requirement of pre-soaking test holes is no longer stipulated.

2.2.2 Constant Head Laboratory Permeameter

After 18 months of testing, the writer concluded that constant head laboratory permeameter was an acceptable method, as it is essentially testing under saturated in-situ conditions. The results tended to be lower when compared with other testing methods, which provided a built-in factor of safety. Most of the results from laboratory testing have, unfortunately, since been destroyed. However, a limited number of example test results are available (Simpson, 1978), as presented in Table 2.4.

Table 2.4: Laboratory Permeameter Test Results (Simpson, 1986)

Date	Soil Description	Fall in last 10 minutes (mm)	Ksat in m/d (Laboratory test)	Comments
7/12/76	Dark topsoil on weathered basalt and clay	3.19	0.044	These are all very low K values for topsoil or loam. The low results could be due to the saturated test conditions. There was possibly sufficient clay contained within the samples to cause some swelling, hence, the impact on the K results. The other factor to consider is that air trapped in the soil sample can decease the K value by a factor of two or three (Healy and Laak, 1974)
7/12/76	Dark topsoil on weathered basalt and clay	3.92	0.150	As above
15/12/76	Topsoil on very sandy brown clay	1.72	0.009	As above
15/12/76	Topsoil on very sandy yellow-brown clay	1.70	0.015	As above

Notes:

1. Falls in the last 10 minutes are from the writer's falling head percolation testing
2. Ksat results are calculated from the writer's laboratory tests
3. Clay contents of all the upper soils were not assessed at the time. Having a working knowledge of the Whangarei Heads area all the uppers soil have above average clay contents, probably at least 20-25%, which would normally reduce the K values.

The above test results are a limited sample of his constant head laboratory testing. In the case of this specific project, the laboratory K testing should be treated with caution. However, as mentioned earlier, work over a number of projects produced acceptable results.

Results of laboratory permeability testing, particularly in South East Queensland soils, are difficult to source. To supplement the writer's work on constant head testing work in a range of soils in Queensland the results, as reported by Dawes and Goonetilleke (2004) are shown in Table 2.5.

Table 2.5: Laboratory Permeameter Test Results

Soil Type	Sample Depth Tested (m)	Ksat Value (m/d)	Comments by Simpson (based on experience and AS/NZS 1547:2000)
Sandy loam	0.2 - 0.35	0.378	This is typically slow. The soil may have been moist and the clay content within the higher range for sandy loam.
Sandy loam	0.25 – 0.4	1.258	This is typical for sandy loam.
Sandy loam	0.1 - 0.22	1.245	This is typical for sandy loam.
Sandy loam	0.25 – 0.41	0.881	This is within the mid-range for sandy loam. The depth indicates it could have more clay content.
Sand	0.2 – 0.37	2.540	This is typical for sand, which can be > 3.0 m/d.
Clay loam	0.3 – 0.44	0.008	This is a very low figure for clay loam. It could have higher clay content and it may have been moist.
Sandy clay	1.1 – 1.25	0.018	This is typical for sandy clay.

The writer has commented on these results based on his experience and AS/NZS 1547:2000. The Ksat values in Table 2.5 generally compare well with values the writer has encountered.

2.2.3 Using Tap Water for Percolation Testing

Using tap water or good quality creek or spring water for percolation testing has been debated for many years. Using tap water for testing purposes creates problems as, in practice, the trenches and beds dispose of a primary standard of effluent. The difference in quality is considerable and this has concerned the writer for many years. To demonstrate this point, refer to Table 2.6 for typical primary effluent constituents.

Table 2.6: Primary Effluent Constituents

Constituent	Primary Effluent Concentration (mg/L)	Comments
Suspended solids (SS)	180	Tap water contains negligible suspended solids. 180 mg/L would eventually contribute to soil clogging.
BOD_5	180	This level of dissolved and settleable BOD would eventually contribute to soil clogging.
Total N	45	This concentration will impact on soil clogging, since it is a mobile state.
Total P	15	This concentration will impact on soil clogging since it will be demobilised.
Chlorides	50	This concentration will not enhance the condition of infiltrative surfaces.
Grease	90	This level will help to clog infiltrative surfaces. The main constraint is the coating of organic matter, thereby inhibiting degradation.
Sodium adsorption ratio	4.1	Safe for soils.
Electrical Conductivity	500 uS/cm	Moderate salinity water and no special salinity control needed.

It would be more scientifically representative to use fresh primary effluent for percolation testing. However, this is not a practical option and it would be a health risk. It is also not practical to assimilate a primary effluent for percolation testing, as sometimes undertaken on the pilot plant testing of industrial wastewaters.

To counteract the problem of using tap water for testing, the writer has applied a factor for increasing the length or area of disposal trenches and beds. For example, applying a factor of 1.25 times the calculated trench length, when using a primary standard of effluent. This has been accepted by many local government engineers and plumbing inspectors in South East Queensland.

2.3 Infiltration System Selection

2.3.1 Trenches versus Beds

The debate over trenches versus beds is worthy of some discussion. Beds tend to be favoured since they require less land area and are more economic to construct.

Trenches provide more side wall area per bottom area than beds. Constructing beds can result in the compaction of the bottoms which inhibits infiltration. Trenches also perform better in more permeable soils, such as loams and sandy loams.

The former Caboolture Shire Council developed wide bed infiltration systems in the mid to late 1990s. There were some earlier failures in situations, and resulting court cases, where the soil permeability was low, or less than 0.2 m/d. The writer has checked the bed systems, within the past ten years, in situations where the soils have consisted of loam and sandy loam and found that they remain largely effective. He developed a simple steel probe which is sunk into the beds for measuring the depth of saturation or effluent level. For assessing the effectiveness of the bed the writer has taken into account the following factors:

1. The depth to the saturation zone/effluent level
2. The soil type and depth
3. The soil moisture, as determined in the fringe of the beds
4. The rainfall prior to the time of the investigations
5. The effluent loading.

The procedure of placing at least two vertical air vents with caps into each trench was a popular idea 30 years ago (USEPA, 1980). This practice was introduced into New Zealand by the writer and Ian Gunn, University of Auckland, during the period 1976-1979. A breather is a vertical pipe consisting of a perforated section within the gravel backfill of a trench or bed. This allows air to enter the trench or bed and enhance aerobic degradation of the effluent, which is also likely to extend the effective operating life. The breather pipe extends from the bottom of the disposal system to about 150 mm above the ground surface. It has a vented cap to prevent the entry of rainwater. The purpose of the breather is to provide a point of inspection as well as to aerate the disposal field. Breathers do not generally cause the release of odours (Winneberger and Klock, 1973).

2.3.2 Shallow versus Deep Systems

The advantages and disadvantages, based largely on the writer's experience, of shallow and deeper trenches and beds are covered in Table 2.7.

Table 2.7: Effluent Disposal Systems – Advantages and Disadvantages

Effluent Disposal System	**Advantages**	**Disadvantages**
Shallow Infiltration Trenches	Less bottom surface area per meter of trenching than beds. Tend to be more sustainable than beds.	Prone to bottom surface clogging by anaerobic effluent.
Evapotranspiration (ET) Trenches - (1)	Landscape amenity. Can be sited on sloping ground, by following the contours.	Reliant on ET disposal only. More suited to aerobic effluents.
ET Beds - (1)	Compact system. Landscape amenity.	Reliant on ET disposal only. Manufactured bottom seal normally applied – cost factor. More suited to aerobic effluents.
ET/Infiltration Trenches - (1)	Makes use of both ET and soil infiltration disposal mechanisms.	No obvious constraints. Surface planting restricted to the growing season.
ET/Infiltration Beds - (1)	Makes use of both ET and soil infiltration disposal mechanisms.	Can cause construction delays in wet weather. Surface planting restricted to the growing season.
Deeper Trenches (600-1500 mm)	Different soil horizons available for infiltration. Benefit of operating under head conditions. Minimal land area required, when compared with shallow trenching.	Some site disturbance during construction.
Deep Shaft (<6.0m) – (2)	Minimal land area required. Minimal site disturbance. Benefit of operating under head conditions.	Not suitable for sloping and potentially unstable ground. Not suitable for location near groundwater table.

Notes:

1. Covered in Portfolio 5
2. Covered in Portfolio 6

Shallow infiltration trenches have the following advantages:

1. The upper soil horizons usually consist of organic and more permeable soils

2. The roots of plants and grasses are able to take up soil moisture and effluent, and the leaves transpire amounts of soil moisture and effluent and enhance the uptake of nutrients.

Deep trench systems have the advantages of increased side wall exposure, the potential for operating under a greater head and are often able to reach more permeable strata when the proximity of groundwater tables do not preclude their use. These advantages were often evident to the writer when investigating deeper systems in the Bay of Islands and other Northland areas in New Zealand.

Dividing the effluent field into two or three individual systems and alternating their operation is, in the experience of the writer, very effective. This allows at least one deep trench to undergo a period of resting so the infiltrative surfaces can be rejuvenated. The same technique applies to shallow trenching.

2.4 Infiltration System Sizing

2.4.1 Influence of Sidewall and Base Infiltration

Both the horizontal base area and the side walls of trenches and beds can act as infiltrative surfaces. Because the side wall is vertical, clogging is not expected to be as severe.

When a gravity fed system is first commissioned, the base is the main infiltrative surface. In the writer’s experience, after a relatively short period, the bottom can become partially sealed to result in some ponding where the side walls act as infiltrative surfaces. This process takes place more rapidly with septic or anaerobic effluents since slimes which can inhibit the infiltration of effluent can develop.

The general trend, in the case of anaerobic or partially aerobic systems, is for the base surface to become partially or fully sealed. In this case the side walls play a major role as the sole infiltrative surface. In the case of deep trenches or shafts, the side walls are the more effective infiltrative surfaces and the tendency for clogging is minimised due to hydraulic head.

It is understood that AS/NZ 1547:2000 is based on bottom surface infiltration which could be somewhat conservative in terms of design. In the writer’s opinion, a reasonable compromise would be to accept the bottom surface plus 50% of the side walls as effective infiltrative surfaces.

It has been the writer’s experience that the frequent cause of failure of infiltration trenches and beds is the use of sub-standard construction techniques. Trenches should be spaced at least 2.0 m apart to facilitate the operation of construction equipment. Trenches, rather than beds, are preferable in soils with higher clay contents so machines can straddle the trenches. This reduces the compaction of the bottom surface.

In the case of side walls, the method of scarifying of the infiltrative surface is often recommended. In the writer's experience, scarifying is not always successful in soils with higher clay content since as it merely smears the surface and makes the problem worse. The preferred method is to liberally apply gypsum which breaks down the surface and improves the infiltrative potential. This method has been well proven within the horticultural and agricultural industries.

Laying the trenches and bed level and along the contours is imperative to achieve optimum effluent distribution. To ensure flows are evenly spread, it is also important that distribution boxes are laid level and preferably on a concrete base (Salvato, undated). The consequence of not doing so has been witnessed by the writer, including one trench being overloaded whilst the other trench was underloaded. It is also important that the trenches and beds are backfilled in the correct sequence. A shade cloth type or similar barrier should be placed over the upper aggregate to avoid the vertical movement of topsoil. It is advisable to well mound the trench or bed top surface to allow for compaction by rainfall and prevent low spots and ponding occurring.

One solution to effluent system recovery if there are two or more trenches, is to rest one trench. This exposes the sides and walls to air and the infiltrative surfaces are broken down by physical and biochemical processes. Another solution is to rejuvenate by the application of oxidising agents. Hydrogen peroxide can restore infiltrative surfaces within days. Hydrogen peroxide oxidises the black insoluble sulfides, thus releasing trace elements from insoluble sulfides. It also oxidises accumulated organic matter.

Another option is the application of biological cultures as additives or enzymes, which in the writer's experience has had degrees of success, over the past 20 to 25 years. It would appear that this method is more successful if follow up dosing is also undertaken.

2.5 Requirements for Spray Irrigation

2.5.1 Adjustments to Spray Irrigation Standard Design

The soil assessment that the writer has adopted involves hand auguring two 100 mm diameter bores for a depth of 500-600 mm within the proposed spray irrigation area. This method assesses the soil type, soil structure, moisture and clay content, as prescribed in AS/NZS 1547:2000. This procedure basically follows AS/NZS 1547:2000 with some adjustments, as presented in Table 2.8.

Table 2.8: Adjustments to AS/NZS 1547:2000 Spray Irrigation Designs

Step	Feature	Adjustments and Comments
1	Select soil category	The soil category is selected from the soil texture, ideally it is sandy loam, loam or clay loam.
2	Select soil structure	In the writer's experience this should be either weak or moderate
3	Soil colour	Soil colour is not mentioned in AS/NZS 1547:2000. Colour is an important soil chemistry factor. For example, a yellow soil indicates it contains clay.
4	Assessed Ksat	AS/NZS 1547:2000 includes assessed Ksat ranges for each soil category.
5	Design Irrigation Rate (DIR)	AS/NZS 1547:2000 includes DIR values. The writer weighs up the site and soil factors for the location before adopting the DIR. For example, the soil texture may have an assessed % clay content, as per Table 4.1D1, which is above the range tabled. In such a case the writer adopts the next lowest DIR, as a factor of safety.
6	Irrigation area	The irrigation area is calculated.
7	Peak factor	The writer (at times) applies a peak factor, ranging from 1.10 to 1.30, to allow for instances when the soils tend to hold moisture or the sites are not exposed to sun and wind.

It is imperative that effluent sprinklers give a good surface coverage. In the writer's experience droplet type spray irrigation is more effective than shallow pressurised irrigation as it benefits from evapotranspiration mechanisms. The writer prefers a riser pipe with about a 6.0 m sprinkler hose, to enable full 360 degree coverage. For an individual home, at least two spray irrigation risers with two sprinklers per riser are required. Pop up type sprinklers can leak and a network of mini-sprinklers is more difficult to maintain.

2.5.2 Additional Soil Depth Requirements for Spray Irrigation

The droplet type spray irrigation of secondary effluent is generally an efficient method of disposal. This is due to the use of a high quality effluent, the direct application on the surface of more permeable upper soils, the presence of soil bacteria which enhances effluent breakdown and nutrient uptake, and the availability of evapotranspiration mechanisms.

Very little attention has been given in the standards and technical literature, to the design of upper soil depths for effluent spray irrigation areas. The writer has developed a methodology for determining the upper soil depth for spray irrigation areas.

The role of the upper soil in spray irrigation areas includes:

1. To provide an organic content to assimilate nutrients
2. To provide a medium for growing grass and selected vegetation
3. To house soil bacteria for the further breakdown of effluent
4. To provide storage for further effluent breakdown.

For several years the former Caboolture Shire Council had a standard area of 200 m^2 and a minimum topsoil depth of 100 mm for all sites for spray irrigation areas. AS/NZS 1547:2000 specifies topsoil depths from 100 to 150 mm for trenches and beds.

The writer has often been faced with the task of assessing whether the existing upper soil is of sufficient depth or whether an overlay of loam or sandy loam is required. For example, site investigations have shown that 130 mm of clay loam exists. When weighing up the soil and site features it was necessary to specify than an overlay of at least 75 mm is required.

The aim is to determine optimum depths of topsoil for different soil and site features. This has been undertaken by the development of two matrices:

1. Soil features
2. Site features.

The matrix rankings are 1 (poor or very limiting) to 5 (very good and requiring no action). It has been assumed that at least two low rankings are required to justify any adjustments or additions to the depth of the upper soil. The soil features matrix is presented as Table 2.9. The site features are presented as Table 2.10.

Table 2.9: Soil Features Matrix to Confirm Upper Soil Depth

Soil Features	Ranking (1)	Comments
Upper soil type:		
Sandy loam	5	Sandy loam and loam are the ideal upper soil types
Loam	4	
Clay loam	3	
Light clay	1	
Upper soil structure:		
Weak	5	Weak and moderate soil structures are acceptable
Moderate	4	
Moderate/strong	2	
Strong	1	

Subsoil type:		
Light clay	4	Light clay and sandy clay loam more acceptable
Sandy clay loam	5	
Sand clay	3	
Silty clay	1	
Cohesive/heavy clay	1	
Existing upper soil depth:		
50-75 mm	1	50-75mm upper soil depth barely acceptable
75-100mm	1	
100-125mm	2	
125-150mm	2	
150-175mm	4	
175mm	5	175mm ideal depth
Upper soil K value:		
0.15-0.25 m/d	1	0.15-0.25 m/d barely acceptable
0.25-0.45 m/d	2	
0.45-0.65 m/d	3	K values from soil testing and K calculation or assessed values as in AS/NZS 1547:2000
0.65-0.85 m/d	3	
0.85-1.15 m/d	4	
1.15- 1.35 m/d	4	
1.35-1.55 m/d	5	
1.55-1.75 m/d	5	
1.75-2.0 m/d	5	
>2.0 m/d	5	

Notes:

1. Denotes rankings by the writer. These are not soil categories, as per AS/NZS 1547:2000

Table 2.10: Site Features Matrix to Confirm Upper Soil Depth

Site Features	Ranking	Comments
Degree of slope: Near level Slight Moderate	 5 4 3	Near level and slight slopes ideal Maximum allowable slope is 6 % (AS/NZS 1547:2000)
Rainfall intensity: Typically light Typically moderate Subjected to heavy	 5 4 3	Can be the influence of micro-climates in some cases
Degree of exposure; Sheltered Partially sheltered Moderate exposure High exposure	 3 3 4 5	For example - elevated sites
Grass cover: Sparse Light Moderate Heavy	 2 2 3 5	Sparse grass associated with poorer upper soils
Selected vegetation type: None Existing trees/shrubs Selected shrubs	 1 4 5	
Need for cut off drain: No need Need	 5 4	

Sprinkler type: Larger droplet spray type Multiple spray heads	 4 5	 Satisfactory effluent distribution Better effluent distribution
Groundwater influence; None Some	 5 2	
Presence of rock: Deep rock Shallow weathered rock Shallow hard rock	 3 2 1	

Assumptions:

1. At least two very low rankings justify any adjustments or increases in the upper soil depth
2. Those effluent loadings do not exceed that prescribed in AS/NZS 1547:2000 for the particular soil type
3. The effluent quality is secondary or advanced secondary.

The roles of the above matrices are best demonstrated by an example given in Table 2.11.

Table 2.11: Site and Soil Matrices Ranking

Soil and Site Feature	Matrix Ranking
Upper soil – loam	4
Upper soil existing depth – 100mm	1
Upper soil structure – moderate	4
Subsoil type – light clay	4
Upper soil K value – 0.6 m/d	3
Ground slope – moderate	3
Rainfall intensity – typically moderate	4
Site exposure – partially exposed	3
Existing vegetation – none	1

Grass cover – moderate	3
Rock – deeper rock	3
Cut off – need	4
Sprinkler – droplet spray	4
Groundwater - none	5

The matrices include two cases with a 1 ranking. The solution is to increase the upper soil loam depth by 25 mm and this eliminates a 1 ranking. This methodology has been applied in several cases by the writer during 2011.

2.6 Comparison of Codes of Practice and Standards

A discussion on infiltration methods and effluent field sizing is not complete without the experience of comparing the former Australian Standard Disposal systems for effluent from domestic premises (AS 1547-1994) with the current AS/NZS 1547:2000. In the early days of designing effluent trenches, beds and spray irrigation areas, the writer noted there were some considerable differences in area estimates when the calculations using both the standards were compared. These changes were discussed with plumbing inspectors since the variation in areal requirements and installation costs increased considerably. The increased costs were important as effluent trenching typically runs at about $2,500 to $4,500 depending on the area, soil type, standard of effluent and the design flow.

It is to be noted that the earlier 1994 standard was based on the falling head percolation and consideration of site characteristics. AS/NZS 1547:2000 was based on assessed K values, given in Tables 4.2A1 and Table 4.2A4, and effluent application rates and conducting constant head percolation tests. Table 2.12 compares typical effluent field sizes based on the writer’s projects in the Caboolture area, using both standards and codes.

Table 2.12: Comparison of Effluent Field Areas from the Caboolture Area

Soil Type	Sizing – AS 1547:1994 (m^2)	Sizing – AS/NZ 1547:2000 (m^2)	Type of Effluent System	Comments and Conclusions
Sandy loam	23	32	Trenching	Current Standard has increase of 39%
	23	32	Bed	Current Standard has increase of 39%

	88	150	Spray Irrigation	Current Standard has 59% increase
Loam	38	42	Trenching	10 % increase
	38	42	Bed	10 % increase
	131	188	Spray Irrigation	46 % increase
Clay loam	44	63	Trenching	43 % increase
	44	63	Bed	43 % increase
	164	210	Spray Irrigation	28 % increase
Sand	23	32	Trenching	39 % increase
	23	32	Bed	39 % increase
	88	150	Spray Irrigation	70 % increase
Light clays	60	125	Trenching	108 % increase
	60	125	Bed	108 % increase
	228	263	Spray Irrigation	15 % increase

The above table is based on an assumed 750 L per day flow, from a 4 bedroom house, using standard water saving devices. Under AS/NZS 1547:2000 for trenches and beds, sandy loam has a Design Loading Rate (DLR) of 20 mm/d, loam is 15 mm/d, clay loam is 10 mm/d, sand is 20 mm/d and light clay is 5 mm/d. Under AS/NZS 1547:2000 for spray irrigation, sandy loam has a Design Irrigation Rate (DIR) of 35 mm/week, loam is 28 mm/week, clay loam is 25 mm/week, sand is 35 mm/week and light clay is 20 mm/week.

The following aspects can be seen from Table 2.12 above:

1. For sand and sandy loam the spray irrigation area must be increased by 59 to 70%
2. For light clays the increase of 108% for trenching and beds highlights the need to consider a higher standard of treatment (secondary) and be faced with a nominal increase in spray irrigation area of 15%.

It would appear that effluent disposal areas under AS 1547-1994 were on the low side, with degrees of risk involved. AS/NZS 1547:2000 is based on research findings and it is more conservative where risk management has been applied. The writer discussed the issue of larger areas, as highlighted in Table 2.12, with South East Queensland plumbing inspectors. The extra cost for trenches and beds was of concern to home owners and developers. A solution was to reduce household water consumption. This has since been addressed by new State and Federal Policies and the Queensland *WATERWISE* program. The installation of full water saving devices in the household plumbing reduces water consumption quite considerably. These devices are listed in Appendix 4.2D in AS/NZS 1547:2000.

The standard of effluent also governs the selection of the K value and distribution rate. During the writer's earlier research on aerobic trenches and beds, in conjunction with evapotranspiration (ET) it was proven (Bernhart, 1973) that aerobic trenches required only about 50% of the area of anaerobic trenches. It is pleasing to note that AS/NZS 1547:2000 has acknowledged this aspect by including a choice of DLR as a maximum rate. This covers double chambered septic tanks with an effluent filter and in situations where site and soil limitations are absent. The mechanisms of ET and aerobic trenches and beds are discussed in more detail in Portfolio 6A.

2.7 Unaccounted Soil Capability to Treat Effluent

During the period of the 1960s to about 1990 the slow infiltration of effluent in slower permeability soils was seen as a treatment and disposal constraint. The relatively long hydraulic retention time of effluent in unsaturated soils provides opportunity for treatment processes such as oxidation, adsorption, die-off and ion exchange (Beal et al, 2005). The writer concurs with Beal et al (2005) since he has held that opinion for at least 30 years, for the following reasons:

1. Oxygen is available for nitrification hence, the value of installing breather pipes in shallow and deep trenches and beds
2. Soil bacteria have more time to breakdown pollutants
3. Natural predation of harmful bacteria takes place with time
4. Ion exchange processes can be enhanced.

The writer maintains that in deep trenches, within anoxic or anaerobic zones, de-nitrification can take place. The value of on-going effluent treatment within sub-surface disposal systems is demonstrated by a case study the writer undertook on the Tiaro Motel, Queensland.

2.8 Case Study Tiaro Motel, Tiaro – Value of Soil Treatment in Effluent Disposal Systems

In the writer's opinion, the treatment value of the effluent disposal process is often not considered, or it is underestimated. Obviously, primary treated effluent would undergo more treatment within the soil environment than a secondary standard effluent that is spray irrigated. The potential for trench side and bottom wall clogging with a matt or biomass layer has been studied by Beal (2007). Whilst the clogging layer will inhibit effluent infiltration, the biomass consists of biological matter and slimes that will further refine the effluent infiltrating into the soil.

The writer has elected to demonstrate the value of further effluent treatment in a shallow trench system by outlining a case study of a project he undertook in Tiaro, Queensland.

The Development Application for a 16 unit motel complex in the township of Tiaro, Queensland was undertaken in July 2003. The motel was connected to the council water supply. The design flow was 6,360 L/d, assuming a full occupancy.

Wastewater was treated in a large primary tank and directed to a sand filter bed system to produce a secondary standard of effluent. The main use of this type of treatment plant is at a household scale. In such cases, the effluent monitoring is normally limited to BOD_5, suspended solids and pH. These parameters do however vary with councils. Nutrient monitoring is introduced when wastewater flows exceed 21 EP which is a requirement of the Queensland EPA. The treatment company had a very limited data base on nutrient treatment capabilities of the primary tank and the sand filter bed system.

Treated effluent is pressure irrigated in a network of narrow shallow trenches, over an area of 1,600 m^2.

The Queensland EPA imposed secondary effluent standards shown in Table 2.13. The nitrogen effluent parameters from a single sampling event are listed in Table 2.14.

Table 2.13: EPA Licence Limits – Tiaro Motel

Parameter	Units	80 % ile	Maximum	Sampling Frequency
BOD_5	mg/L	15	20	Quarterly
Suspended solids	mg/L	25	30	Quarterly
pH			8.5	Quarterly
Faecal Coliforms	org/100 ml	4000		Quarterly
Total N	mg/L	NA	15	Quarterly
Total P	mg/L	NA	10	Quarterly
Residual Chlorine	mg/L	0.3	0.7	Weekly

Table 2.14: Effluent Results – Tiaro Motel

Parameter	Units	Results[1]	Comments
Ammonia - N	mg/L	9.59	High
Nitrate - N	mg/L	67.3	High
Nitrite - N	mg/L	1.47	
Total N	mg/L	82.7	Very high
Town water pH (not effluent)		8.1	Town water range 7.6 to 8.3

Note:

1. One-off sampling and results requested by the writer to assess the reasons for low pH and high Total N, as requested by the Queensland EPA

Routine effluent monitoring results were not available to us since they were being retained by Hervey Bay Water. However, it the BOD_5, suspended solids and faecal coliform results were consistently below the EPA limits. The very low BOD_5 and suspended solids and low levels of faecal coliforms certainly show a higher than secondary standard of treatment was being achieved. On the other hand, pH levels were generally low (around 6) and the Total N in excess of the EPA maximum. It is understood that the low pH was due to the presence of humic acids as a result of the wastewater degradation and sand filtration processes.

The writer engaged the assistance of James Pulsford, Soil Agronomist/Agricultural Scientist, Morayfield and the following aspects were established:

1. The high Total N and low pH was indicative of incomplete nitrification taking place within the treatment plant (primary tank with a sand filter bed system)
2. Further nitrification and de-nitrification would take place in the effluent irrigation field
3. The high Total N benefitted from a plant and grass fertilization point of view

The lower pH and Total N levels were considered not to be detrimental to the receiving environment for the following reasons:

1. The method of irrigation was shallow sub-surface, with no direct contact with the public
2. The denitrification process would take place in the soil environment, with no impacts on the soil and vegetation
3. The Total N will reduce to acceptable levels and the pH will increase
4. Since the groundwater was very deep, groundwater contamination was not an issue in this case
5. The upper catchment of the irrigation area was very small

6. Surface runoff was diverted away from the irrigation area
7. Enhanced nutrient uptake by vegetation took place.

Under the Queensland EPA license conditions, effluent sampling is taken at the outlet of the treatment plant. In this case, there is no opportunity to sample effluent quality following exposure to pumping, sub-surface irrigation and soil contact.

The expected effluent treatment mechanisms, following the treatment plant, are as follows:

1. Some aeration during the pumping phase to the irrigation field and flow within the distribution pipe work
2. Continuing nitrification, within the irrigation trenching
3. De-nitrification within the irrigation trenching and associated soils. The trenches are 200 mm deep with loam into silty clay. The irrigation trenches would contain anoxic zones which would enhance de-nitrification
4. The above treatment mechanisms would also increase the pH levels.

It is to be noted that the writer made a case to the EPA for no effluent chlorination. Since the irrigation is by shallow sub-surface there is no direct human content or risk of pathogenic bacteria exposure. Chlorination is likely to inhibit the functioning of soil microorganisms (natural bacteria included) even at low chlorine residual. This reasoning was accepted by the EPA.

3 Literature Review

This Portfolio focuses on shallow trenches, beds and spray irrigation for treated wastewater disposal. Carryer (1977) reports on factors that can complicate shallow disposal methods which include:

1. Variations in drainage mechanisms from season to season
2. Groundwater level fluctuations
3. Changes in soil moisture, which can result in soil cracking.

An earlier study reported by Healy and Laak (1974) revealed some very interesting findings for the design of effluent systems. Conclusions the writer has drawn from this survey of previous work are:

1. There is little difference in the final or long term acceptance rate between a soil that is flooded continuously leading to anaerobic decomposition and the same soil that is flooded intermittently, allowing aerobic decomposition
2. A hydraulic head of about 300 mm of effluent is required to push the liquid and nutrients into the soil which would lead to efficient operation of an effluent field

3. The fact that a relatively long term acceptance rate (LTAR) develops indicates that a balance is achieved between bacterial growth and decomposition of clogging matter within the active soil interface. Material is accepted only at the rate at which it can be consumed
4. The more permeable the soil, the further the suspended and dissolved materials can move into it. This results in a thicker active zone allowing a higher long term acceptance rate.

3.1 Trench and Bed Configurations

During the 1970s in North America there was a period of applied research activity on on-site effluent disposal undertaken by such persons as Bernhart (1973), Bouma and Converse and Converse (1974), Healy and Laak (1974) and Winneberger (1974). This research activity tended to be the catalyst for the writer’s experimental work.

3.1.1 Microbial Life in Trenches and Beds

The functioning of aerobic or partially aerobic effluent systems is enhanced by aerobic oxidising bacteria, found in zones of aeration in the soils (Bernhart, 1973).

The upper soils also support earthworms, protozoa, rotifers, microscopic nematodes. Protozoa feed mainly on bacteria; rotifers feed on bacteria and protozoa, and the nematodes help maintain an open soil structure.

These organisms have the role of reducing complex organic matter to simpler and more soluble forms. The aerobic environment in turn accelerates the activity of microbial and larger life thereby offsetting soil clogging and helping to maintain the trench-soil interface (Salvato, undated). The action of aerobic or partially aerobic trenches and beds is discussed in more detail in Portfolio 6.

3.1.2 Shallow versus Deep Systems

Shallow trenches can be used over a broader range of soil types than deep systems. In soils with ample clearance from the groundwater deep trenches are suitable. They utilise sidewall infiltration providing for the continuous rejuvenation of the infiltrative surface due to aerobic action with each cycle of fill and drain (Gunn, 1994).

3.2 Spray Irrigation

In the early 1990s the writer organised and attended a study tour by New Zealand engineers and scientists of golf courses being irrigated with treated effluent, within South East Queensland. This experience was applied to the spray irrigation of municipal effluent on pastures for Taupo, New Zealand. It was found that spray irrigation was much more efficient than subsoil irrigation (Water and Wastes, 1996).

Early guidelines for spray irrigating treated effluent were developed in Victoria (Davis, 1991).

The writer acted as an advisor to the original spray irrigation of sugar cane and tree plantation trials with municipal effluent at Hervey Bay in the mid-1990s. These experiences lead into more work on the spray irrigation sugar cane and crops in Queensland, as reported by Gardner and Gibson (2000).

Spray irrigation of effluent gives rise to aerosols that can contain harmful microorganisms. Buffer areas are required which are dependent on such factors as proximity of the public, wind speed and direction, spray height, droplet size and the type of system used. The addition of vegetation barriers within the buffer areas can minimise spray drift (NWQMS, 2000).

There is a dearth of information on droplet type spray irrigation of secondary effluent at a household or house cluster scale. However, the method of droplet type spray irrigation, at a household scale, has been well presented in AS/NZS 1547:2000. This covers upper soil assessment, effluent loading rates, surface water diversion and maximum ground slopes. There have been a number of investigations into the effects of irrigation effluent on soil biochemical properties (Spier, 2002).

The studies have shown that effluent application has a beneficial effect on soil properties and plant growth and this is reflected in the enhanced soil biochemical activities. Monitoring is required over a longer duration than has occurred in several studies examined.

3.3 Construction and Restoration Methods

A frequent cause of the early failure of effluent disposal systems is the use of poor construction techniques. During trench and bed construction, the soils can be compacted, smeared or puddled, thereby impacting on the infiltrative potential. This applies more to soils with high clay content. Undue compaction should be avoided during the backfilling operation (EPA, 1980).

Gunn (1994) reports on construction constraints. These include avoiding construction during wet conditions, distribution pipes must be laid level, and surfaces should be well mounded with topsoil.

Over the years, methods of trench and bed restoration have been sought so that failed systems need not to be replaced (SSWM, 1976). Trench and bed failure may occur occasionally or continuously.

Occasional failure may be the result of heavy and prolonged rainfall periods or periodic overloading. Continuous failures are more difficult to determine. The first step is to determine the age of the system. Short term failure can be caused by poor siting, design or construction. If failure has occurred after many years of useful service it is likely to be due to hydraulic over-loading or poor maintenance (EPA, 1980).

3.4 Effluent Disposal Design Aspects

The physical and chemical attributes of soils which would generally be considered for effluent disposal, would include (Dawes et al, 2000):

1. High soil cation exchange capacity
2. Good permeability to assist the movement and absorption of effluent
3. Stable soil with calcium to magnesium ratio of > 0.5
4. Dominance of exchangeable Ca or exchangeable Mg over exchangeable Na
5. Low exchangeable Na
6. Uniform bright colours indicating good aeration of the soil

The writer concurs with this summary of effluent design attributes.

3.4.1 Soil Clogging

The clogging zone in trenches and beds can develop as a result of (ASCE, 1981):

1. Compaction and smearing of the soil face during construction
2. Puddling as a result of constant soaking of the soil during operation
3. Blocking of the soil pores by excess solids in the effluent
4. A build up of biomass from the growth of micro-organisms
5. Ion exchange causing a deterioration of the soil structure
6. Precipitation of sulfides, during anaerobic action.

Although many of the field studies have been confined to laboratory column testing, the results have been quite meaningful (Bouma, 1975). Clogging in sands and sandy loams is less of a problem since the soil environmental is actually partially aerobic and the purification processes are enhanced (Bernhart, 1973).

Several studies have indicated that biological mechanisms are the primary causes of soil clogging (ASCE, 1981). The writer concurs with this comment since the extent of biological growth, in anaerobic trenches and beds, cannot be under-estimated.

In the case of sub-surface effluent systems, a clogging mat will inevitably form at the soil-liquid interface (Dawes, 2006). Reasons for the clogging of infiltrative surfaces have been well researched and the adverse impacts leading to failure of disposal systems have been well documented (Dawes, 2006).

Even though the formation of clogging mats is considered a major problem, it can also be beneficial (Dawes, 2006). The infiltrative capacity of the clogging mat has been shown to reach a state of equilibrium after a period of time (reported by Dawes, (2006), based on Otis (1984) and Siegrist et al (2000). Effluent will still be able to seep through the mat, but at a slower rate. The formation of the mat can enhance purification by increasing biogeochemical reactions, as well as creating unsaturated conditions beneath it due to the reduced soil K rate below the clogging mat (Dawes, 2006). This situation also permits greater contact between the effluent and the soil particles as effluent flow is only through the smaller pores.

Very interesting work on field and laboratory column testing has recently been undertaken in Queensland. The low permeable biomat zone that develops on the infiltrative surfaces of trenches and beds is a key component of the hydraulic and treatment performance of a soil absorption system (Beal, 2007). A survey of 19 councils in South East Queensland was undertaken by Beal (2007) to obtain data on aspects of soil absorption systems. As a result of Beal's experimental work in soil columns, the following observations and conclusions were made:

1. A relationship between biomat resistance and organic loading rate was observed in all the soils
2. Results showed that whilst initial soil K values are important in the establishment of the biomat zone, the long term acceptance rate (LTAR) is predominantly influenced by the biomat resistance, and to lesser extent the unsaturated soil hydraulic conductivity, and not the soil K values
3. Field results indicated that sidewall flow above the biomat during high trench loadings was a major flow path in the soils
4. Modeling and field studies demonstrated that during extreme trench flows the permeability and extent of the biomat free upper side walls is the ultimate determinant of trench hydraulic failure.

The accumulation of solids, in the writer's experience, is often due to the carry-over of solids from poorly maintained septic tanks, and the associated biological growth resulting in soil clogging. This growth can be chemical and bio-chemical. Totally anaerobic effluent trenches and beds in heavy soils can become clogged in less than a year, due to deposits of anaerobic bacteria and of black ferrous sulphide.

The writer agrees with the reasons for soil clogging. The beneficial side of a clogging mat should not be ignored.

3.4.2 Soil Limitation Ratings – Septic Tank Effluent

Table 2.15 contains the criteria that can be used for rating soils for use as effluent disposal systems (Voss, 1975). It has been included in this thesis since such a rating table is not often developed and since it also covers a range of criteria. This ratings method would have been very useful before the introduction of AS/NZS 1547:2000.

Table 2.15: Soil Limitation Ratings

Item affecting use	**Degree of Soil Limitation**		
	Slight	**Moderate**	**Severe**
Permeability class (1)	Rapid 2 and moderately rapid	Lower end of moderate	Moderately slow 2 and slow
Hydraulic conductivity rate	> 25 mm/hr (2)	25 – 15 mm/hr	< 15 mm/hr
Percolation rate	> 45 mins/15 mm (2)	45 – 60 mins/25 mm	< 60 mins/25 mm
Depth to water table (4)	> 1.8 m	1.2 to 1.8 m	< 1.2 mm
Flooding	None	Rare	Occasional or frequent
Slope	0 to 8 %	8 – 15 %	> 15 %
Depth to hard rock (4)	> 1.8 m	1.8 – 1.2 m	< 1.2 m
Stoniness class (5)	0 and 1	2	3,4 and 5
Rockiness class (5)	0	1	2, 3, 4 and 5

Rating scale:

1 – Limitation rating should be related to Ksat values of soil layers below the distribution pipe
2 – Where pollution is a hazard to water supplies
3 – In arid and semi-arid areas, soils with moderately slow K may have a limitation rating of moderate
4 – Based on assumption that distribution pipe is 600 mm deep
5 – Referred to in US Soil Survey Manual

3.4.3 Risk Zones of Effluent Soils

Risk assessment approaches for effluent systems have emerged over more recent years. As a basis for preliminary work for identifying areas of possible high risk, as a result of poor system performances on the Gold Coast, Carroll et al (2003) developed criteria for effluent soil risk zones. The criteria adopted a qualitative approach. Initially, the soil suitability was evaluated based on drainage characteristics of different soil types on the Gold Coast. Planning criteria were based on allowable lot sizes as specified in the Town Plan. Environmental sensitivity was based on the regulatory setbacks that were current at the time. The soil, planning and environmental criteria and appropriate risks are listed in Table 2.16**.**

Table 2.16: Initial Risk Zone Criteria for Soil-based Effluent Disposal at the Gold Coast, Queensland

Soil Criteria			
Risk	**Criteria**	**Implication**	**Comments by Simpson**
High	Soils with imperfect or poor drainage ability	Soils with poor drainage inhibit effluent disposal, which reduces soil renovation ability. Hydrosol soils although well drained sandy soils, are saturated, making drainage poor	Drainage characteristics may be poor but it is possible that they could be better for bacteria removal, due to the increased retention times in the soils.
Medium	Soils that are moderately well drained. Man made soils and soils which have been altered.	Moderately well drained soils allow slow drainage, which can affect soils renovation ability.	The reported implication by Carroll et al (2003) may be in conflict with the belief of many practitioners, including myself, that longer residence times in the soils may be beneficial in terms of more treatment potential.
Low	Soils that are well drained	Soils with good drainage have increased ability to renovate effluent	Soils with very high drainage characteristics have minimal in-situ treatment potential, particularly in the case of primary effluent.
Planning Criteria			
Risk	**Criteria**	**Implication**	**Comments by Simpson**
High	Less than 0.4 ha	Minimum lot sizes for developments in these residential areas must not less than: residential – 400 m^2 detached dwellings – 600-2000 m^2 hinterland subdivisions – 4000 m^2	The minimum lot size for residential lot of 400 m^2 can be disputed as being too small. In the Moreton Regional Council the minimum lot size is 3000 m^2 where I have been responsible for DAs involving several hundred such lots. The writer advocates that site specific studies are warranted on lot sizes rather than these areas being in accordance with the Town Plan.
Medium	0.4 to 4.0 ha	Lot sizes must not be less than 4000 m^2 minimum and no larger than 4.0 ha	The writer comments on 4000 m^2 lots also apply here.

Low	Greater than 4.0 ha	Rural residential areas with lot sizes greater than 4.0 ha, with maximum lot sizes up to 20 ha.	The writer considers these minimum lot sizes to be conservative.
Environmental Sensitivity Criteria			
Risk	**Criteria**	**Implication**	**Comments by Simpson**
High	Less than 100 m from nearest water source	Greater risk of contamination of surface water resources from surface and sub-surface flow.	This will depend on the effluent quality and site and soil specific factors.
Medium	Between 100 m and 500 m from nearest water source	May impose some risk of contamination from surface and sub-surface flow, more likely surface flow.	This could be a little conservative. A site specific study should be undertaken since it will depend on factors including soil type, slopes, soil Ksat, nutrient assimilation and ground cover.
Low	Greater than 500 m from nearest water source	Minimum risk of contamination of water resources.	This will also depend on site and soil specific factors.

It is understood that the above risk assessment tables have been refined as more data and analytical results became available (Carroll et al, 2003).

3.5 Soil Permeability

3.5.1 Factors Associated With Permeability

A decline in soil infiltration or permeability is usually attributed to one or more of the following (Jackson, 1977):

1. Blockage of soil pores due to a build up of bubbles of air released from solution in the water
2. Blockage of soil pores due to washing of silt and clay particles down from the soil surface
3. Blockage of soil pores due to the accumulation of bacterial slime
4. Change in structure of the soil due to the dispersion of clay as a result of cations that prevent dispersion from exchange sites with progressive leaching.

Since the side wall is normally vertical, clogging may not be such a problem, when compared with the bottom surfaces of trenches and beds. This is due to the following factors (SSWM, 1976):

1. Effluent suspended solids, sometimes sourced from the carry-over from the septic tank, may not be a significant factor in side wall clogging

2. A higher quality or secondary standard of effluent will not be a problem
3. The side walls will be subjected to periodic effluent level rising and falling, thereby alternative wetting and drying occurs whereas the bottom surface is constantly inundated
4. Slimes and other biomass can readily slough of the side walls, thereby helping to ensure the infiltrative surface is maintained.

Salinity

Salinity is a characteristic of soils relating to their content of water – soluble salts. This refers to salts, which are dissolved in the soil water and are free to move down profile by leaching. Salinity is commonly measured as electrical conductivity (EC) of a soil water extract of ratio 1:5. As noted by Beavers (1993), the values of EC and total dissolved solids (TDS) are interrelated, and the ratio of the two values is generally in the range of 0.5 to 0.7. Beavers (1993) further quotes a factor of 0.64 for assessment of treated effluent for irrigation. Salt is commonly added to the soil by the effluent and problems may arise if the added salts accumulate to a concentration that is harmful to the soil structure (Dawes et al, 2000). Therefore good drainage is essential to ensure that salt accumulation on the surface does not reach harmful levels. There is a need for further investigations to evaluate the suitability of various soil types for surface irrigation where the TDS/EC ratio is less than 0.5 (Dawes et al, 2000).

Soil Colour

Soil colour is given very little attention in AS/NZS 1547:2000. Soil colour is assessed on the surface of a freshly broken aggregate of moist soil (McDonald et al, 1990). The common colouring agent of the upper soil is organic matter which is relatively darker than the underlying horizons. The common colouring agents in subsoils are iron oxides, which in well drained soils range from red through to brown to yellow. A red colour usually indicates good drainage. Yellow and grey colours are indicative of reducing conditions as poor drainage and aeration (Gunn et al, 1988). Mottles are spots, blotches or streaks of subdominant colours different from the matrix colour.

Clay Content

The type and amount of clay present influences many soil physical properties such as cracking, swelling, pore space and stability. In gradational profiles, the increasing clay content with depth may result from greater exposure to leaching and intensive weathering in surface horizons than at depth. There is a downward movement of clay during rainfall events, fine particles can be washed downslope adding finer soils to lower slopes and possibly enriching nutrients (Dawes et al, 2000).

Calcium to Magnesium Ratio

The calcium to magnesium ratio in a soil can be employed to indicate cation distribution, particularly in the case when the subsoil is dominated by Mg. It has been found that ratios less than 0.5 are associated with soil dispersion (Emerson, 1977).

Exchangeable Sodium Percentage (ESP)

ESP is the amount of sodium held in exchangeable form on the soils cation exchange complex expressed as a percentage of the total. High levels of soil Na + can adversely affect plant growth and lead to the development of poor soil physical conditions (Rayment et al 1992). At a high ESP (>6), soils tend to lose aggregation and undergo clay dispersion, impermeability, surface crusting and poor aeration, as reported by Dawes et al (2000).

The study of two New Zealand soils shows there is increasing anecdotal evidence that irrigating sodium concentrated effluents increases ESP by at least 31%. This study also shows that irrigating with sodium contaminated effluent may cause a worsening of SAR, structural problems and reductions in soil permeability (Menneer, et al, 2001).

Unsaturated Flow Conditions

Unsaturated flow is necessary to achieve adequate treatment of septic tank effluent. Column studies and field monitoring experience indicate that 1.0m of unsaturated soil below the infiltration system can be adequate for pathogenic purification, provided that the soil is not overloaded (Bouma, 1975).

Many soils have seasonal or permanently high ground water levels which are close to effluent infiltration systems. The addition of effluent can impact on the horizontal flow lines and raise the ground water level. Minimum clearances of the groundwater level and the bottom of effluent trenches or beds are given in AS/NZS1547:2000. A minimum buffer of unsaturated depth is important. This minimum buffer is less in the case of lower permeability upper soils and it is at a maximum in the case of sands. It is also a maximum for primary effluent and less for secondary aerobic effluent.

Dosing and Resting Technique

The research and practical experience of Winneberger (undated) quickly led him to the belief that the principle of alternation of effluent fields would be beneficial. Early experimental work by Bouma et al (1974) was undertaken in Wisconsin using an upper loam overlying a silty clay loam. Soils with similar profiles were known to malfunction in this soil profile. A very acceptable percolation rate is 25 mm in 30 minutes, when dosed once daily by pumping. The benefits of dosing and resting sequences in aerobic disposal systems are discussed in more detail in Portfolio 6.

Effluent Distribution

Gravity flow is the most common type of effluent occurring within the first four to six metres of the distribution box. The most effective method of distribution is by pumping. The system then operates in sequences of dosing and resting which has promotes system longevity. The resting phase enables oxygen to be in contact with the infiltrative surfaces which enhances biological degradation of the slimes and biomass by aerobic bacteria.

In the case of anaerobic trenching, effluent dosing by pumping, siphon or a tipping bucket system, will induce aerobic conditions and result in better effluent distribution (Bernhart, 1973).

Dunedin City Council, New Zealand, favours the installation of the tipping bucket device as an alternative to flood loading by pumping (ON-SITE NewZ). The writer has used tipping buckets for the effluent dosing of alternating sand filter beds and effluent trenches. They can however, cause problems if the buckets corrode or the shaft seizes through the lack of lubrication.

Generally, effluent is distributed from pipes which have either slots or holes. The rationale for the sizes of the slots and holes does not appear to be well documented. The Environmental Protection Agency (Ireland) (2009) recommends 8mm diameter holes at 600 centres, located at 4, 6 and 8, as on a clock face. This appears reasonable but the writer maintains that holes should be located at 12 o'clock for the express purpose of oxygen supply.

3.5.2 Permeability Testing

The saturated hydraulic conductivity (Ksat) was measured in loamy sand, fine sandy loam, silty loam and clay in southern Ontario. The techniques used included air-entry permeameter, constant head Guelph permeameter and falling head permeameter. Statistical comparison of the mean Ks values indicated significant differences between some or all the methods within each site. The measured K values ranged from over an order of magnitude for sand, one to two orders on loams and three orders of magnitude for clay (Lee et al, undated).

It is interesting to note that double ring infiltrometer testing was selected as the technique to investigate permeability responses in sands in the Swan Coastal Plain in Perth using wetting agents. The initial application of the wetting agents usually improved the wettability of the sand. This was thought to be due to the reduction in water surface tension (Gross et al, 2011).

4 Reflections and Recommendations

4.1 Soil Permeability

4.1.1 Factors Associated With Permeability

Thinking back, the writer feels that the benefit of unsaturated flow conditions, particularly in primary effluent systems, is often over looked. Salinity build up and the development of unfavourable SAR conditions is often over looked. This situation can occur in areas with poorly distributed rainfall with long dry periods in between. On reflection, the writer feels that it is advisable to check SAR to prevent potential large scale vegetation impacts or failures, particularly when communal effluent systems are being designed,

As discussed earlier, effluent dosing and resting is most beneficial to system sustainability.

The writer feels that practitioners should be encouraged to compare assessed Ksat values with calculated Ksat values, under different soil conditions, to establish a co-relation, if using AS/NZS 1547:2000. The assessed soil moisture content should also be taken into account when selecting effluent loading rates. This could impact on soil K values and DLRs.

Soil colour is related to soil chemistry and the degree of permeability. In hindsight, there is a need to place more importance to recording and understanding the significance of soil colour.

4.1.2 Soil clogging

Semi-Permeable Barriers in Trenches

The construction of effluent trenches usually involves at least two and normally three layers of soil or aggregate. For many years, a geotextile (BIDUM or similar) has been specified to act as a semi-permeable barrier, to prevent the downward movement of topsoil, sand or fine gravel. With the benefit of hindsight, in the writers opinion, a geotextile could be prone to clogging, particularly with a primary standard effluent. With anaerobic effluent there is the opportunity for binding to occur as the result of solids and slimes. Over the years the writer has used hessian and shade cloth as an option, with a higher porosity, to minimise clogging. Since hessian has a limited life shade cloth is the writers preferred option. The writer feels that similar benefits may apply in cases where anaerobic or septic tank effluent, with the aid of simple aeration pipes and dosing/resting, the development of slimes will be minimised.

4.2 Soil Permeability Testing

4.2.1 Double Ring Infiltrometers

The writer found that the double ring infiltrometer was suited to shallow soil testing where surface or subsurface pressurise compensating dripper systems were used (Barnett and Ormiston 2010). Geary (1992) had a degree of success with using a double ring infiltrometer in shallow duplex soils. On reflection, the writer found that the method was more suited to shallow soil testing, governed by the allowable depth of penetration of the rings.

The saturated hydraulic conductivity (Ksat) was measured in loamy sand, fine sandy loam, silty loam and clay in Southern Ontario. The techniques used include air-entry permeameter, constant head Guelph permeameter, and a falling head permeameter. Statistical comparison of the mean Ksat values indicated significant differences between some, or all of the methods within each site. The measured Ksat values ranged from over an order of magnitude for the sand, one to two orders on the loams and three orders of magnitude for the clay (Lee et al, undated).

4.2.2 Using Tap Water

AS/NZS 1547:2000 has noted the limitation of the standard clean or tap water percolation test (Dawes, 2006).

4.3 Tiaro Motel Case Study

Regarding the Tiaro Motel effluent disposal project, as outlined in Section 2**,** on reflection it would be interesting to undertake investigations to:

1. Conduct additional sampling and analyses of the effluent parameters, particularly pH and nitrogen compounds
2. Confirm whether more complete nitrification and de-nitrification takes place within the shallow pressurised irrigation field
3. Undertake the soil sampling plan mentioned earlier.

To date, this contingency plan has not been put into operation. Monitoring of treated effluent at the end of the irrigation area will confirm whether the expected nitrification and de-nitrification processes have taken place.

4.4 Codes of Practice and Standards

4.4.1 AS/NZS 1547:2000

AS/NZS 1547:2000 has been operational since August 2000. This has allowed ample time to review the merits and problems of this joint standard.

In hindsight, the design tables for effluent disposal (Tables 4.2A1, 4.2A2, 4.3A3 and 4.2A4), the soil texture/clay content of soils (Table 4.1D1) and typical domestic wastewater flow design allowances (Appendix 4.2D) have been most useful.

The standard has the following limitations or problems:

1. Sand mounds have tended to be undersized. The former Caboolture Shire Council found that the footprint area should be increased and they developed a model to rectify this

2. The standard is generally disjointed and difficult to follow in places
3. "Informative" and "normative" sections have not been clearly defined in places
4. Soil structure descriptions are difficult to determine in some cases
5. The soil K formula contained an error. More assistance on how to apply this formula is needed since it is to be used by plumbers and drainers
6. Nutrient assimilation was not covered. This was probably not detailed as it was not an issue at the time the standard was formulated
7. Set back distances for effluent fields were not covered. This was addressed in Queensland by the QPW Code
8. The inclusion of a maximum design loading rate (DLR) for trenches and beds (Table 4.2A1) in situations with no site and soil limitations could have been open to some misinterpretation.

It is understood that some of the listed problems have been addressed in the new AS/NZS 1547: 2012 but this is outside the scope of this thesis.

4.4.2 Risk Assessment Tables

After some consideration the writer advocates that the updated risk assessment tables by Carroll et al (2003) would be most useful as an inclusion in the revised version of AS/NZS 1547:2000, the Queensland Plumbing and Wastewater Code or as a water quality planning tool in future revisions of Town and Regional Plans.

5 Conclusions

5.1 Soil Permeability Testing

The writer considers that his experimental work of percolation testing and laboratory K testing has been of value since it was undertaken when effluent design codes tended to be deficient.

Slower soil Ksat values, within the range of 0.06 to 0.18 m/d, have some potential for treating effluent, due to the retention time offered whereby further stabilisation and the die-off of harmful micro-organisms can take place.

The constant head permeameter is appropriate to a full range of soil types. Soil Ksat values determined by constant head percolation testing compared reasonably well with assessed Ksat values in AS/NZS 1547:2000. Soil moisture levels can also be an important factor.

The single ring infiltrometer has an application for some soils for determining K values, since it is simple, economic and hand portable.

The double ring infiltrometer has an outside ring which helps to reduce error due to lateral flow in the soils. Based on the experience in the lower North Island of New

Zealand (Barnett and Ormiston, 2010) the double ring infiltrometer is suited to the assessment of surface and sub-surface pressure compensating dripper systems.

5.*2* *Infiltration System Selection*

5.2.1 Trench versus Beds

Trenches are preferred to beds for effluent disposal (USEPA, 1980). This has been proven by researchers and confirmed by the writers own experience.

Trenches can also be curved to suit the site contours and to avoid mature trees. Trenches are the only suitable option on slopes < 10% unless the system is terraced.

It has been the writer's experience that trenches receiving anaerobic effluent are also more suitable in terms of system sustainability when compared with beds. When the base surface infiltrative ability fails, effluent seepage is limited to the side walls. This becomes less of a constraint in trenches with more limited base areas. Furthermore, preference is given to trench systems, with higher side wall areas, to minimise the impact of bottom surface clogging.

Installing breather pipes in anaerobic trenches and beds to encourage partially aerobic conditions and longer life offers economic benefits. This also enhances de-nitrification. Breathers should be placed at the centre and end of trenches or in the centre and sides of beds.

Be aware that relatively long retention times of effluent in unsaturated soils provides the opportunity for treatment processes such as oxidation, adsorption, die-off, ion exchange and nitrification/de-nitrification.

Undertaking thorough site and soil assessments is essential. AS/NZS 1547:2000 covers most aspects to be investigated. Local knowledge should also be sought, including rainfall data, flooding, rock presence and type, and the past performance of effluent trenching.

ET systems should be considered in lower permeability soils, in areas with higher exposure to sun and wind, to enhance all disposal mechanisms. Also refer to Portfolio 6. The use of ET/infiltration trenches should be encouraged in locations with long growing seasons, high ET potential and upper soils with some degree of permeability.

To prevent potential larger scale vegetation impacts or failures, the writer feels that it is most advisable to check the SAR, particularly when larger scale or communal effluent systems are being investigated. The risk of failure could apply to grass or vegetation cover for spray irrigation and evapotranspiration systems.

5.2.2 Shallow versus Deep Trenches

Deep trench systems have an application where upper soils are not suitable for shallow trenches, provided the trench inverts are clear of the groundwater table and slope stability is not a risk. Deep trenches provide considerable wet weather storage. Further benefits of deep trenches are reported in Portfolio 5.

5.3 Requirements for Spray Irrigation

There is merit in using the alternative droplet type spray irrigation in sandy loam, loam and clay loam soils. Particular attention must be made to divert surface runoff and to not exceed a downhill slope of 6 degrees. On reflection, the use of secondary effluent and dual disposal mechanisms of ET and soil infiltration renders spray irrigation as probably the most sustainable effluent disposal system. In the case of wet weather, a low retention mound could be constructed below the irrigation area, to contain effluent

5.4 Codes of Practice and Standards

5.4.1 AS/NZS 1547:2000

AS/NZS 1547:2000 to some degree was a step in the right direction for site and soil assessment, effluent disposal design and management. Some comparison of this joint standard is made with AS 1547-1994 and it was concluded that for most soil types larger disposal areas were required. The current review of the joint standards committee is expected to address technical issues and short comings that have not been covered previously.

5.4.2 Reserve Effluent Disposal Areas

Earlier standards and Codes of Practice in Australasia and the USA often did not stipulate the need for reserve effluent disposal areas. In the writer's experience this oversight was not sound practice. Septic tank effluent systems in particular need to have a reserve area for additional trenching or beds. It has been the writers experience that the effective life of an anaerobic trench or bed is within the range of 14 to 20 years, depending on the soil type. Systems with more permeable soils tend to be more effective for longer periods.

It is gratifying to note that the AS/NZS 1547:2000 calls for 100% reserve areas for trenching and beds. The writer feels that land must be allocated for reserve areas. According to AS/NZS 1547:2000 no reserve areas must necessarily be allowed in the case of spray irrigation systems. They have the advantage that they must use a secondary standard of effluent. However, in the writer's opinion it is sound practice to allow for at least one extra sprinkler irrigation circle, in the case of droplet type sprinklers.

Barnett and Ormiston (2010) believe, based on the experience in the lower North Island of New Zealand, that the benefits of having a reserve outweigh the risks of not having a reserve area.

5.5. Effluent Disposal Design and Operation

5.5.1 Soil Properties

A discussion on a range of soil properties and characteristics is considered necessary to help gain an understanding of sustainable effluent disposal.

Exchangeable sodium percentage, sodicity and sodium absorption ratio can impact on soil quality and soil K values. They should be investigated for larger than individual home on-site effluent disposal projects.

Soil clogging inevitability takes place and reduces effluent infiltration but the clogging mat can work as a treatment aid. The treatment ability of biological/chemical mats should not be underestimated.

5.5.2 Tiaro Motel Case Study

The case study of the Tiaro Motel indicates that the soil environment within the disposal field could be useful for undertaking further effluent treatment. There is potential for further nitrification and de-nitrification within the disposal system, but there is a need for soil monitoring to confirm the potential of in- trench treatment.

More designers should realise that a slow permeability soil does not necessarily result in poor effluent treatment levels. The additional effluent treatment that takes place, since residence times can be long, should be appreciated.

5.5.3 Climatic Factors

Climatic factors as evapotranspiration, rainfall, and temperature influence the design and performance of effluent disposal systems. The degrees or exposure to sun and wind and the site elevation are also important.

5.5.4 Sustainability

The sustainability of effluent disposal systems can be enhanced through the following practices:

1. Alternative effluent dosing and resting, to create aerobic conditions and result in more sustainable disposal systems
2. Venting trenches and beds, since aerobic conditions enhance effluent treatment
3. Maintaining unsaturated flow conditions which enhances treatment mechanisms.

If a secondary standard of effluent is available it is very beneficial to use droplet spray irrigation.

5.5.5 Construction and Restoration Methods

The writer feels that there is now a better appreciation of effluent disposal construction methods. Biological cultures continue to be used to restore the infiltrative capacity of the side walls and bases of trenches and beds.

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Portfolio 3 - Reed/Gravel Beds

1 Introduction, Aim and Technology Background

The underlying theme for this Portfolio is that reed/gravel beds are a viable treatment method, they are capable of nitrification and de-nitrification, and they are suitable for treating mixed wastewater and greywater.

The overall aim of this Portfolio is to document the writers research involvement in reed/gravel beds, review the literature for more recent developments, reflect on this work and the work of others and make recommendations for technology improvements.

This Portfolio covers different aspects of predominately horizontal flow reed/gravel beds (vertical flow reed/gravel beds are discussed briefly in Section 3.3), so it has been structured in the following sections:

1. Description of reed/gravel beds and their design development
2. Research by Simpson and associated researchers, including reed/gravel bed design and nitrogen removal
3. Research by others, including work on nitrification and denitrification processes
4. Reflections and recommendations for research and development.
5. Conclusions.

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1.1 Description of Reed/Gravel beds

Reed/gravel beds are otherwise known as sub-surface flow (SSF) constructed wetlands and they can be referred to as the “root zone method” of treatment. Constructed wetlands are engineered systems that have been designed to utilise natural processes involving wetland vegetation, soils and the associated microbial assemblages to assist in treating wastewaters. Reed/gravel beds are specialized wetlands designed such that the flow moves through a gravel matrix, which is planted with emergent type plants or macrophytes. Pebbles may also be used to form the matrix.

Interest in the use of reed/pebble bed treatment systems generally developed in the 1980s in several EU countries. Professor R Kickuth has had reed beds operating in Germany since 1972 (Kickuth, 1984).

Tests of various systems took place in Germany, Denmark, France, Belgium, Austria and the UK (Cooper, 1990). Davies (1988) reported that reed/gravel beds were a well established method of wastewater treatment for individual homes and small towns up to 500 persons in Europe. A typical section of a European horizontal flow reed/gravel bed is in Figure 3.1 **(**Cooper, 1993).

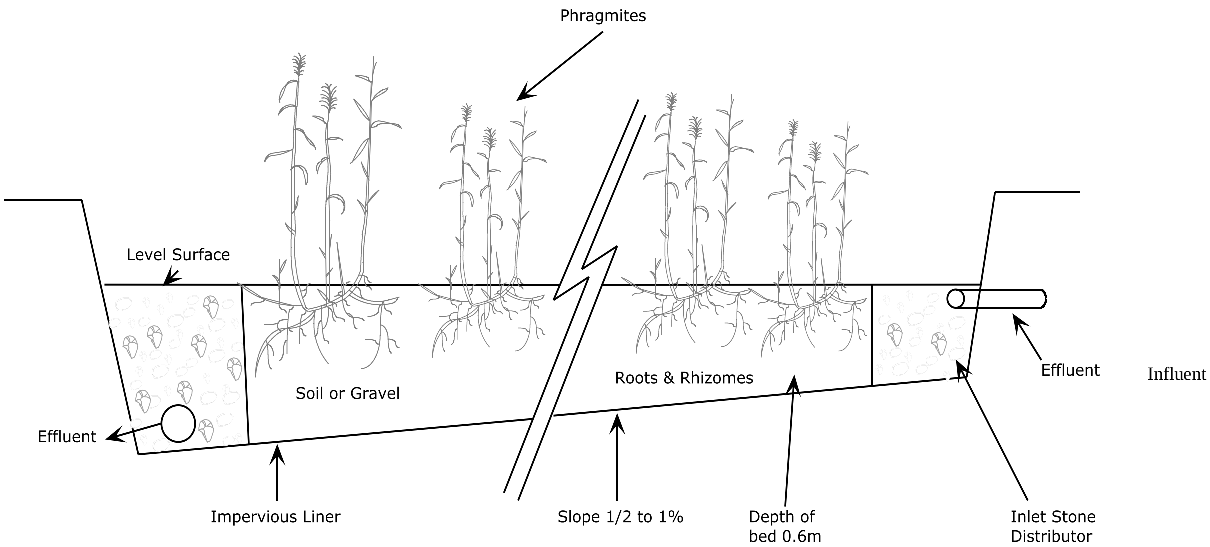

Figure 3.1: Section of European Horizontal Flow Reed/Gravel Bed

As noted in UK design guidelines, a sub-surface wetland provides a highly intense wastewater polishing system (Cooper et al, 1996). It usually occupies less space than an open surface wetland and is able to treat greater quantities of wastewater. The basic features of reed/gravel beds include:

1. A shallow bed, typically 500 to 600 mm deep, with a liner and filled with gravel
2. Wastewater is treated by aerobic and anaerobic or anoxic microbial activity
3. Planting with suitable emergent vegetation, such as *Phragmites australis*, which can transfer oxygen into the gravel bed through its roots and rhizome. Composting of suspended solids in the wastewater may occur in the above ground layer, derived from dead leaves and stems
4. Wastewater flows moving horizontally through the bed. The roots and rhizomes of the macrophytes provide a hydraulic pathway through the rhizosphere along which wastewater can flow
5. Minimal plant harvesting is required
6. Wetland influent is often from a septic tank.

In Australia, early wetland researchers included Weir, Finlayson, Mitchell, Chick, Bowmer, Roser, Bavour**,** Fisher, Breen**,** Davies and Cullen, over the period 1976 to1989 (Fisher, 1990). A key study is a large-scale wetlands pilot investigation set up by the former Hawkesbury Agricultural College and the CSIRO (Fisher, 1990). This project involved five wetland and two control systems, ranging from reed/gravel beds to open water wetlands and mixtures of both. Monitoring took place over the period June 1984 to August 1987. BOD_5 removal of the treated wastewater varied from 59-95 %, suspended solids removal from 74-97% and Total N from 39-69%. The initial retention times were 3 to 4 days and Total N removal was poor. This was attributed to the possibility that the development of nitrifying bacteria had been slow. It is interesting to note that very much improved nitrogen removals were obtained when retention times were increased to 6 to 9 days.

The processes of nitrification and de-nitrification are discussed later in this Portfolio. The removal of P is not discussed in any detail in this Portfolio, with the exception of the Wamuran pilot wetland, where the use of water lettuce was successful. Generally, in the writer's experience, there is an initial P uptake in wetlands which tends to taper off after the first year, unless special media and/or soil additives are used.

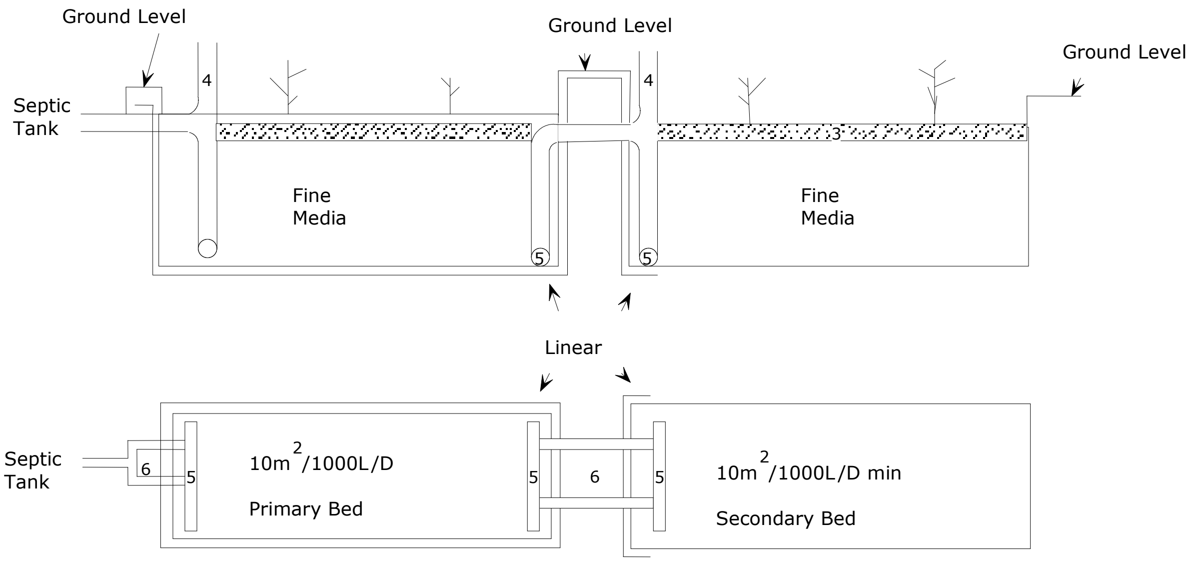

Figure 3.2: Victorian EPA Reed/Gravel Bed Details

A three year study of Australian wetland systems was reported by Davies et al (1991). Thomas Davies, formerly of Monash University, undertook work on reed/gravel beds for a range of effluents. The writer has communicated from time to time with Thomas Davies since 1992, particularly on reed/gravel systems. A domestic scale reed/gravel bed was developed by Davies (1992) to treat septic tank effluent. This was approved by the Victorian EPA under a Certificate of Approval CA 033/92. A detail of the Victorian EPA system is presented as Figure 3.2 (Davies, 1992).

The writer was one of the main organisers of Australia's first *National Conference on Wetlands for Water Quality Control*, in Townsville, in 1995. The writer invited Ian Gunn, Senior Lecturer in Civil Engineering, University of Auckland as a guest speaker. (Ian Gunn was his supervisor during Diploma of Public Health Engineering studies in 1974, a co-researcher on ET effluent disposal systems and a co-supervisor for his Master Science thesis during the period 1998 -1990). He reported that the first engineered utilisation of a wetland in New Zealand for treating effluent began in Paihia, Bay of Islands in 1979. New Zealand adopted the wetland practice of Canada and the USA in the 1980s. The earliest wetland design guidelines were prepared by Kloosterman and Griggs (1989) from Works Consultancy Services for the Auckland Regional Water Board. In Australia, reed/gravel beds were designed based on European experience using the horizontal flow approach. On reflection, this was probably a sound move, based on temperate climatic similarity.

The City of Whangarei (then with a population of about 45,000) developed the first large scale wetland system after trials in New Zealand in 1987 (Venus and Oldcorn, 1992).

In the late 1980s the earliest reed/gravel bed was for Coromandel township (1,200 population), south east of Auckland. It is interesting to note that the Consulting Engineer carried out permeability testing of the gravel bed media prior to construction. The writer can recall that hydraulic conductivity and the potential blocking of media were concerns in reed/gravel beds at the time. Many other reed/gravel beds systems have been installed in New Zealand since 1988.

Advantages of reed/gravel beds are as follows (DNR, 2000 - this section of the Guidelines was compiled by the writer) and Davison (2001):

1. Greater ability to treat high organic loads
2. Better cold weather tolerance
3. Greater treatment area per unit area of land, when compared with surface flow wetlands
4. Mosquitoes and odours are generally not a problem
5. No public safety problem, since there is no body of water
6. Minimal harvesting needs
7. Odour reduction by virtue of the reduction of BOD_5 and some disinfection

Disadvantages of reed/gravel beds are limited to the following:

1. Higher capital costs, associated with the supply of gravel, particularly for larger systems
2. Systems can be prone to blocking, particularly at inlet zones, unless this aspect is recognised in the design
3. Generally limited to smaller loadings. One exception is the preliminary design of a large leachate treatment wetland system in Singapore completed by the writer (described further in Section 2.2.2), where reed/gravel beds were used for de-nitrification.

2 Reed/Gravel Bed Research by Simpson

This section covers the writer's research activities and projects associated with reed/gravel beds, including his involvement in the Queensland Artificial Wetlands Research Program. The writer was awarded funding to undertake a study tour of wetlands covering the Shortland Wetland Centre (NSW), Hunter Valley, Sydney (University of NSW), Griffith (CSIRO), Adelaide University, Melbourne Water and Monash University, Albury (CSIRO), University of Canberra, and the ACT Planning Authority (Simpson, 1993).

The writer will also draw from industry experience in the design of treatment systems, notably at the Tropical North Queensland Institute of TAFE, Cairns and the Lorong Halus Landfill, Singapore. Specific aspects including the selection of plant species, design rationalization and preliminary sizing are also described in more detail, as well as promoting nitrification and denitrification processes within reed/gravel beds.

2.1 Queensland Artificial Wetlands Research Program

An *Artificial Wetlands Research Co-ordination and Advisory Committee* was set up by the Queensland DPI Water Resources in 1992. This was a joint program between the Queensland State Government, local governments and universities with additional funding from the Federal Government under the National Landcare Program. The writer was the Project Engineer and Coordinator with responsibility for the selection of committee members, the design of the pilot wetlands, and co-coordinating the research projects. Ten pilot plant wetlands were constructed extending from Douglas Shire Council in the north, to Blackall in the west, and to Goondiwindi in the south west of Queensland.

It is to be noted that only one pilot plant was a reed/gravel bed. These pilot wetlands covered a full range of Queensland climatic regions, which included sub-tropical, arid, tropical wet and tropical dry (Greenway and Simpson, 1995). The pilot projects offered the ideal opportunity to undertake specific research studies by universities and research organisations. Routine performance monitoring was undertaken by the respective local governments. Summary information on the pilot scale wetlands, taken from Simpson and Woolley (1995), is provided in Table 3.1.

Table 3.1: Summary Information on Queensland Pilot Wetlands

Location	Climate	Construction Date	Treatment Objectives	Wetland Type	Water Reuse/Discharge
Mossman	Wet tropics	July 1995	A, B, C	OSF	Golf course irrigation
Edmonton (Cairns)	Wet tropics	April 1994	A, B, C	OSF	Mangrove estuary discharge
Ingham	Wet tropics	February 1993	A, B, C	OSF	River discharge
Mt St John (Townsville)	Dry tropics	March 1993	A, B, C	OSF	Natural wetland discharge
Mt Bassett (MacKay)	Dry tropics	May 1994	A, B, C	OSF	Mangrove estuary discharge
Yeppoon	Sub-tropical	October 1994	A, B, C	Shallow Melaleuca	Ground soakage
Emu Park	Sub-tropical	January 1994	A, B	OSF	Land disposal
Blackhall	Arid	February 1993	A, C	OSF	Irrigating parks and tree lots
Goondiwindi	Arid	June 1994	A, B	OSF	Pasture irrigation
Wamuran (Caboolture)	Sub-tropical	October 1992	Household on-site	SF + OSF	Ground soakage + plant irrigation

Treatment Objectives:

A – BOD_5 and suspended solids reduction
B – nutrient reduction
C – disinfection

Wetland Types:

OSF – open surface flow
SF – sub-surface flow

The writer took the responsibility of setting up and co-coordinating specific studies which included the following:

1. Suitable wetland plants species, largely by the Queensland Herbarium and also Griffith University
2. Identification of noxious and evasive weeds, by the Queensland Herbarium
3. Uptake of heavy metals, by Griffith University
4. On-site household greywater and blackwater treatment, by the Queensland University of Technology and University of Queensland

5. Wetland plant species selection, nutrient uptake and biomass production, by Griffith University
6. Nutrient uptake, by DPI Soil Scientists and Chemists
7. Performance of wetlands in the wet and dry tropics, by James Cook University
8. Literature review and phosphorus removal in wetlands, ACTFWR, James Cook University.

In his endeavour to develop wetland technology in the many climatic areas of Queensland, the writer undertook an in-depth interest in each research project. He also maintained an interest in the development of this technology in the other states of Australia, the UK, the USA and Europe. He also assisted Dr Margaret Greenway, Griffith University with the co-supervision of undergraduate and Masters Theses on wetlands. Simpson and Woolley (1995) reported on the outcomes and progress on the *Artificial Wetlands Research Co-ordination and Advisory Committee*, based in Brisbane.

2.1.1 Wamuran, Caboolture - Reed/Gravel Bed Pilot System

The Artificial Wetlands Research Program involved a number of open surface systems supported by local authorities. The writer saw the need for the design of a smaller scale reed/gravel bed wetland for treating household wastewater in sub-tropical conditions and based along the lines of the Victorian EPA approved system (Simpson and Gibson, 1993).

A domestic scale system was subsequently constructed at Wamuran, Caboolture as part of the Queensland Artificial Wetlands Research Program. The system initially consisted of three reed/gravel beds for blackwater and two reed/pebble beds for greywater. The layout of this system, sourced from Borthwick (1995), is shown in Figure 3.3.

The bed sizes were 16 m^2 for blackwater and 8 m^2 for greywater and they were originally planted with *Typha* and *Phragmites* emergent species. The strategy was designed to operate separate greywater and blackwater reed/gravel beds as a trial. Should there be any treatment performance problems, it could then operate as a combined system.

It was the writer's philosophy that, by using this modular and cell system, some intermediate monitoring could be undertaken to gauge the different degrees of treatment being achieved under different detention times. An open surface wetland, originally intended as a storage pond, was 15 m^2 in area. It was aware that reed beds should have a surface of sandy soil so the plants could be anchored and the aggregate insulated from the heat of the sun, as adopted in Victoria (Simpson, 1993; Simpson and Gibson, 1993).

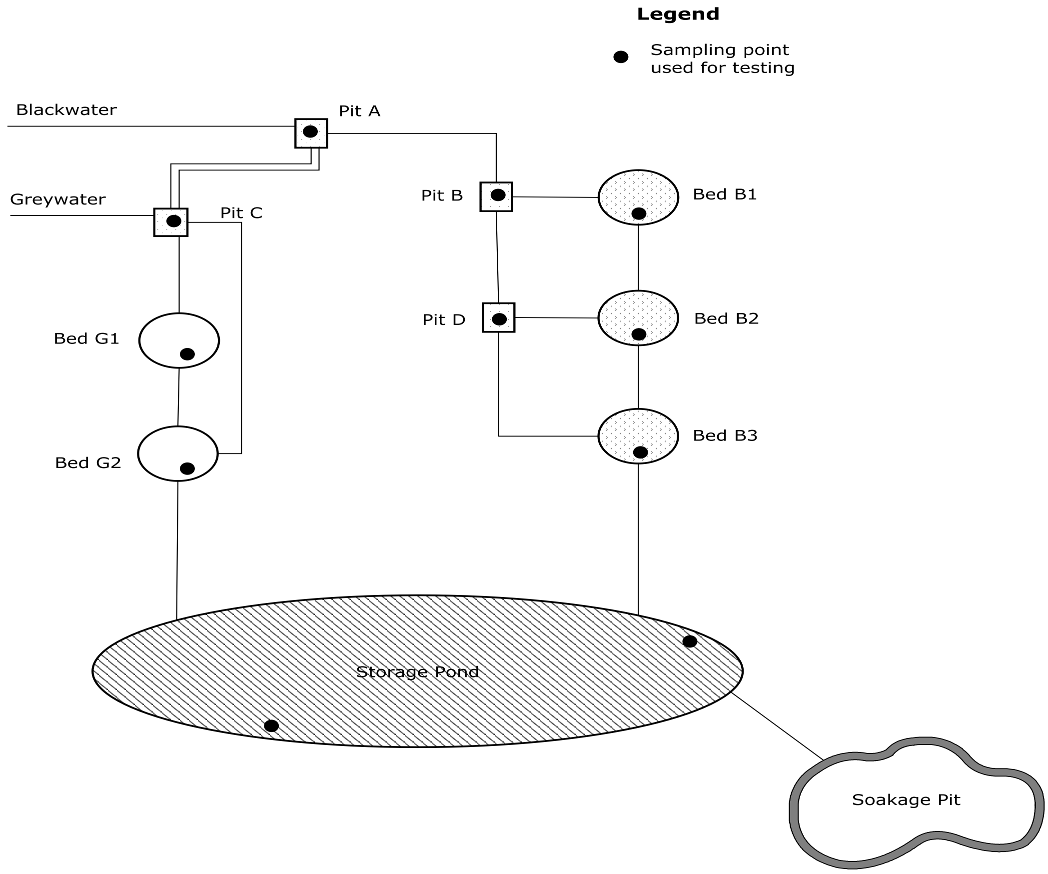

Figure 3.3: Layout of Wamuran Wetland System

Monitoring of the greywater components over several months revealed poor plant growth and limited reductions in nutrients. The nutrient contents of the plant stems and roots were analysed by Griffith University and no clear conclusions were derived. The emergent *Phragmites australis* species in the reed/pebble bed had periodic problems with rust. *Typha australis* was also established in the reed/pebble bed. These species struggled with the greywater feed stock. The *Phragmites spp*. was eventually replaced with Canna lilies which thrived.

The reason for the substandard performance of the reed/pebble beds treating greywater and planted with *Typha* and *Phragmites* was not identified at the time. Based on the response and the difference in appearances of the plants in the blackwater and greywater systems, it was soon apparent that the greywater either contained some inhibiting ingredients or it lacked a nutrient balance. The reed bed planting showed that the blackwater plants were taller, denser and the leaves were darker whereas the greywater plants were stunted and lacked colour.

An investigation of leaf and tissue analyses was undertaken by Griffith University, under the supervision of Dr Margaret Greenway. Unfortunately, no conclusions were drawn as to why the growth of the greywater plants was being hindered. The writer has continued to study this problem and has since concluded the following:

1. Greywater contains an excess of carbon in relation to nitrogen (Dallas, 2005). This imbalance could be a cause of the poor plant responses with greywater treatment
2. Greywater contains soaps, detergents, household cleaning products and surfactants, which could inhibit the treatment of greywater. When mixed with blackwater these chemical constituents would be diluted and be less of a treatment problem
3. Greywater has a less favourable N: P ratio, when compared to mixed wastewater. This is likely to be a major contribution to the treatment problem
4. It is possible that the high pH of greywater and sodium content, when compared with mixed wastewater, could have been a problem
5. The writer appreciates the role of emergent plant stems for transferring oxygen from the atmosphere to the roots. The roots and rhizomes of emergent plants on greywater reed /gravel beds would receive less oxygen, when compared with open surface wetlands, resulting in less plant response and treatment potential.

Marshall (1996) also reported the problem of plant development using *Phragmites australis*, when treating greywater in NSW. On the basis that two researchers have reported similar problems, the writer would suggest that emergent plants, other than *Phragmites spp*, be used. He has experienced good treatment and good responses from *Typha spp*. when treating greywater in Victoria and Queensland. Residential reed/gravel beds treating greywater have shown good responses from *Typha spp*. in Victoria and South Australia (Marshall, 1996).

A further investigation of the Wamuran pilot plant greywater and blackwater influent and effluent has since been done. According to Thomas Davies (pers. com August 2009) the problem is due to the unfavourable Total N to Total P ratio in the greywater, compared with blackwater. The writer has also discussed the problems of treating greywater in horizontal flow reed/gravel beds with Professor Ashantha Goonetilleke, Queensland University of Technology (pers.com 2009). We both concur that treatment is inhibited by the lack of a nutrient balance and/or organic matter. An analysis of the Wamuran water quality has been undertaken to confirm the above greywater treatment inhibitions. Unfortunately, Total N measurements were not taken prior to the mixing of greywater with the blackwater. A summary of the reed/gravel bed components of the Wamuran pilot plant is provided in Table 3.2.

Table 3.2: Blackwater and Greywater Compositions and Ratios

Blackwater	Greywater	Comments on Greywater
BOD_5 median – 175 mg/L	BOD_5 median – 210 mg/L	The blackwater BOD_5 compares with the range of 140-200 mg/L (Metcalfe and Eddy, 1991) The greywater BOD_5 was higher than usual. The house was on tank water which would have increased the concentration due to lower water usage. Mean BOD_5 of 160 mg/L based on Jeffersen and Solley (1994). The greywater BOD_5 median of 210 mg/L does not represent a lack of organic matter.
NH_4 - N median – 68 mg/L	NH_4 – N median – 2.2 mg/L	The $NH_{4\ \_N}$ median for the blackwater is just above the range of 20-60 mg/L (Metcalfe and Eddy, 1991). The NH_4 median for the greywater is low, indicating a possible constraint in terms of treatability. Mean NH_4 –N of 5.3 mg/L (based on Jeppersen and Solley, 1994)
Reactive P – 37 mg/L	Reactive P – 4.95 mg/L	The Reactive P level for blackwater is higher than Total P reported range of 10-30 mg/L (Metcalfe and Eddy, 1991)[1] Mean Total P of 8 mg/L (based on Jeffersen and Solley, 1994).
COD median – 595 mg/L	COD median – 381 mg/L	These values are normal concentrations (Metcalf and Eddy, 1972).
BOD_5 to COD – 1 to 3.4	BOD_5 to COD – 1 to 1.8	COD values are greater than BOD_5 values (Sawyer and McCarty, 1967**)** so these ratios are normal. Usual untreated wastewater ratios are 0.3 - 0.8 (Crites and Tchobanoglous, 1998)

Note:

1. Reference as reported by Borthwick (1995)

Based on some flow records, the theoretical HRTs are about 15 days for blackwater and 2 days for greywater.

The N to P ratio of the combined greywater and blackwater was 7.1 to 1.0. This is an acceptable ratio for wastewater, as evidenced by the very good combined blackwater and greywater treatment performance results, reported by Borthwick (1995). Since no Total N testing was undertaken, a comment on the suitability of the N: P ratio cannot be made. However, it is the writer's feeling that there was an imbalance of Total N and Total P in the greywater that contributed to the substandard growth of the emergent plants in the earlier greywater cells. The important factor is that the writer made the correct move by combining the greywater with the blackwater, as originally planned.

In more general treatment performance terms, the monitoring up to June 1993 showed that the pilot project had much promise in terms of BOD_5, suspended solids, COD, and Total N reductions (Simpson and Gibson, 1993). To enhance nutrient uptake, the writer opted to experiment with the use of water lettuce (*Pistia stratiotes*) in the open surface cell. The combined wetland system, with the water lettuce produced excellent results. The water lettuce was responsible for very high and consistent phosphorus reductions. A summary of the combined treatment system results is given in Table 3.3.

Table 3.3: Combined Wamuran Pilot Wetland Mean - Performance Results

Parameter	Units	Bed 3 outlet[1]	Pond outlet	Comments
BOD_5	mg/L	75	30	BOD_5 close to a secondary degree of effluent quality
SS	mg/L	50	26[2]	SS reduction represents a secondary quality of effluent
Ammonia - N	mg/L	55	12	Significant Ammonia – N reduction. Indicates further nitrification and de-nitrification takes place in the pond. Aeration, by roots of water lettuce, considered to enhance treatment (Borthwick, 1995)
Nitrate	mg/L	2.7	2.2	
Ortho P	mg/L	16.5	7	P levels were consistent over time. Pistia stratiotes or water lettuce was very successful. Harvesting water lettuce not a problem in smaller ponds or wetlands.

Notes:

1. Bed 3 is the last reed/gravel bed
2. SS result when water lettuce established in the pond

The writer was involved in the initiation of the project, design, construction supervision, and co-ordinating the effluent monitoring.

Based on the research work on the household scale reed/gravel bed system at Wamuran, the following design and operational suggestions have been made based on Borthwick (1995) and the writer's input:

1. The design and operation must be failsafe – use a liner, careful gravel and surface cover placement and avoid possible blockages
2. Ensure good flow distribution by placing inlets well away from outlets
3. Ensure adequate retention time, at least greater than 3 days

4. Design the beds so that under very low or no flow conditions, the beds will retain some wastewater to provide a source of moisture and nutrients
5. Mix the greywater water with the blackwater
6. For optimum ammonia - N reduction, select macrophytes which have high oxygen transfer capacity and deep root systems, typically *Typha spp.*
7. Operate the system to encourage root penetration through the entire water column and maximum contact between root zones and wastewater.

Based largely on the writers experience with the Wamuran pilot scale reed/gravel bed system, the useful life of a reed/gravel bed can be seen to be limited by substrate clogging. Hence, it is most advisable to ensure the following**:**

1. Effective primary treatment, with provision to prevent the carryover of suspended and settleable solids. For example an effluent filter at the outlet of the septic tank
2. Including an inlet zone containing larger rocks (40-75 mm), with a higher void space
3. Installing a high void plastic media (modules of plastic media can be removed for high pressure spray cleaning) experiences
4. Installing a separate solids collection chamber, or modified PVC grease trap, a practice developed by the former Caboolture Shire Council.

2.1.2 Climatic Influences on Wetlands

The Queensland Artificial Wetlands Research Project also provided the opportunity to investigate the effect of climate on wetland vegetation in open surface wetlands and reed/gravel beds. The link between climate and the functioning of wetlands in Queensland has been reported by Simpson and Beavers (1992) and Simpson and Woolley (1995).

Queensland experiences sub-tropical, tropical, tropical wet, tropical dry and temperate arid climatic conditions. There is a very wide diversity of rainfall and temperatures across the state. Temperatures below zero are recorded in the south east highlands and open plains in the south west. Prolonged periods of high temperatures are recorded during the summer. Evaporation is high in western Queensland and along the Queensland coast. Estimates indicate that the wetland evapotranspiration losses are in the order of 20 to 40% of the average daily inflow during drier summer months in Northern Queensland (Edmonton, Ingham and Mt Bassett) and in the order of 8 to 16% at other times.

This indicates that macrophyte transpiration losses are high, particularly in Northern Queensland. This can influence the wetland hydraulic design. It is understood that water budgets were started or undertaken at the pilot wetlands in Mackay, Blackall, Edmonton, Ingham and Mt St John. These Northern Queensland climatic conditions are conducive to high plant growth rates and hence offer much potential for use in constructed wetlands (Greenway and Simpson, 1995).

Climate has a definite effect on the selection of wetland plants. The range of appropriate species is more diverse in North Queensland and along southern coast and far less diverse in the arid west and south west Queensland (Simpson and Gibson, 1995). Climate also has an influence on the propagation or planting of wetland systems. It has been shown in Queensland that warmer days and nights greatly enhance the establishment of wetland plants. This certainly enhances wetland metabolism, as shown by the relatively short retention times to attain a desired standard of effluent treatment (Simpson and Woolley, 1995).

Up until 1995, the limited experience of the Queensland Artificial Wetlands Research Program had certainly indicated that climatic factors greatly influenced wetland design and management (Simpson and Woolley, 1996). Higher metabolism is associated with higher temperatures. In contrast, higher temperatures can reduce dissolved oxygen levels (Krebs, 2009; Odum and Barrett, 2005). These aspects were noted during the Queensland Artificial Wetlands Program, particularly in Northern Queensland.

Based on a review of overseas literature and a consideration of South East Queensland conditions by Simmonds and Bristow Pty Ltd, evapotranspiration rates from wetlands can be as high as $100m^3$/ha/day, but they generally average at approximately 50 m^3/ha/day (Simpson, 1992). Such rates can obviously have an influence on wetland sustainability and water balances.

2.2 Simpson Industry Projects Involving Reed/Gravel Beds

As part of his consultancy practice, the writer has been involved in the design of numerous reed/gravel bed systems. A small selection of relevant projects is discussed in this Section. Experience gained from this work, coupled with his involvement in the Artificial Wetlands Research Program, has led to a sound knowledge of plant species selection which is also covered in this Section. Furthermore, there are specific design aspects associated with reed/gravel beds that have been developed over time working on various projects, and these include:

1. The need to incorporate design features to enhance nitrification and denitrification processes to facilitate nitrogen removal
2. A compilation of reed/gravel bed parameters so the design of these systems can be rationalized
3. Identification of rule of thumb sizing of reed/gravel beds.

2.2.1 Horticultural Irrigation Water Treatment, TNQ Institute of TAFE, Cairns

The writer designed an irrigation water/runoff water treatment system for the Horticultural Section of the Tropical North Queensland (TNQ) Institute of Technical and Further Education (TAFE), Cairns (Simpson, 2008). Under the Queensland EPP (Water) this untreated water was classified as a trade waste since it contained fertilizers, insecticides, pesticides, algicides, fungicides and biocides. Such irrigation water could not be discharged without treatment to the adjoining tidal wetlands or the council stormwater system.

The treatment system is the first of its type in Australia. The design took into account several considerations including economics, simplicity of operation and maintenance, effectiveness of pre-treatment, site aesthetics and the provision of a student training facility.

The "first flush" runoff was directed to a reed/gravel bed and an activated carbon filter, before discharge to the council stormwater system. The "first flush" amount was determined by Queensland Project Services. Higher flows, typically during higher rainfall events, were diverted to an adjacent canal by a Fox Environmental © system.

A design report (Simpson, 2008) was completed and the design flow for the reed/gravel bed and the activated filter was 7,000 L/day. Based on a detention time of 3.5 days, the reed/gravel bed surface area was 130 m^2. A detail of the reed/gravel bed and activated carbon filter is in Figure 3.4a and 3.4b.

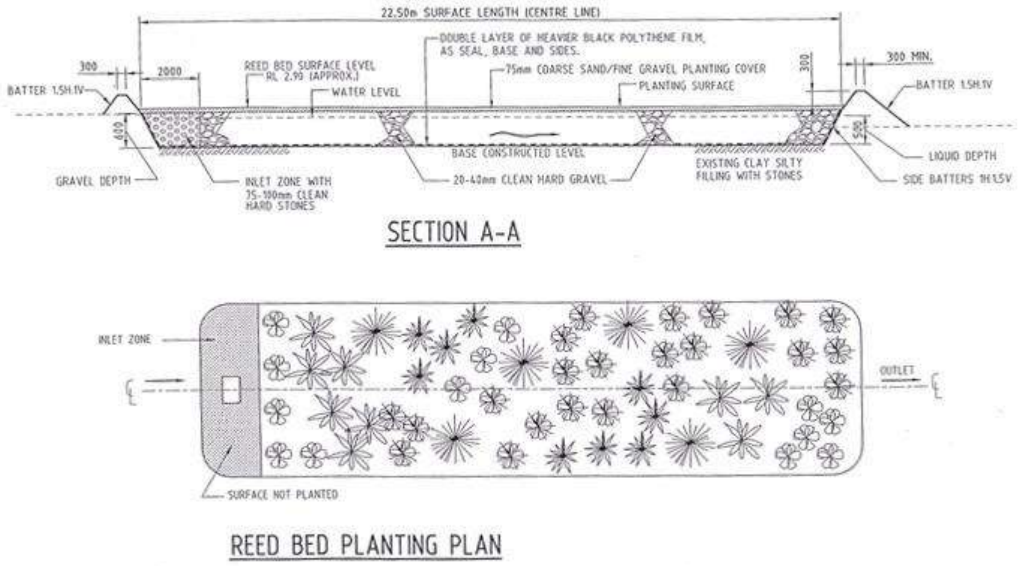

Figure 3.4a: Cairns TAFE Treatment System - Reed/Gravel Bed Filter

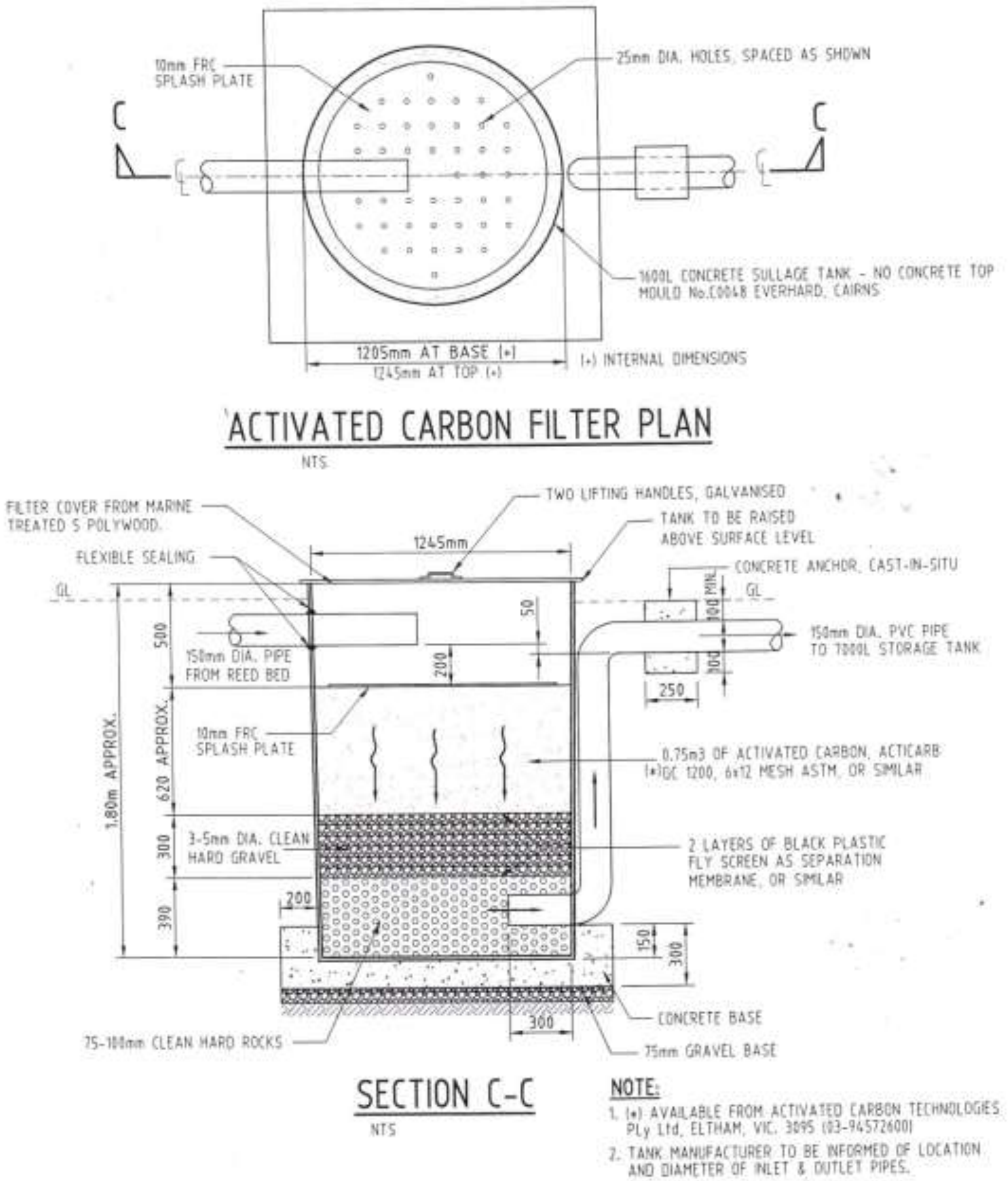

Figure 3.4b: Cairns TAFE System - Activated Carbon Filter

The activated carbon filter was included to intercept the insecticides, fungicides, and pesticides which would have been present, at low levels.

It is understood that the treatment system is now operational. This method of treatment will require some form of performance monitoring to ensure that the final discharge is a quality suitable for the receiving environment. It was intended that monitoring would to be undertaken by the students, using the TNQ Institute of TAFE facilities.

Runoff from nurseries and greenhouses typically contains high concentrations of nitrogen (mostly as nitrate) and low concentrations of organics (Vymazal, 2009). Fortunately, high nitrogen was not the problem with the TNQ Institute of TAFE project. Vymazal (2009) also reports that the introduction of legislation in NSW to control runoff from agricultural and horticultural activities has encouraged commercial plant nurseries to collect and recycle irrigation drainage. Headley et al (2001) have tested pilot scale horizontal flow reed beds filled with 10 mm basaltic gravel and planted with *Phragmites australis* to treat nursery runoff. Total N and Total P removals were > 84% and > 65%, respectively, at hydraulic retention times (HRTs) between 2 and 5 days.

2.2.2 Lorong Halus Landfill, Singapore - Leachate Treatment Reed/Gravel Beds

The writer undertook the preliminary design of a large horizontal flow reed/gravel bed system for treating leachate from a closed landfill in Singapore (Waste Solutions Australia, 2006).

This project presented the main problem of being a large scale, widely fluctuating leachate flow with high levels of contaminants. The main challenges more specifically were as follows:

1. COD up to 800 mg/L but was typically 90-400 mg/L
2. High Ammonia - N which could be a maximum of 700 mg/L and typically around 200 mg/L
3. High Total N, to a maximum of 1,035 mg/L but typically 100-200 mg/L
4. High SO_4 which could be up to 120 mg/L but was typically 10 to 60 mg/L
5. High salinity which could be up to 6,000 mg/L but was typically 2,000 to 3,000 mg/L
6. The design flow was 2,500 m^3/day, which was subject to wide variations.

A layout plan is provided in Figure 3.5. This design project has been included in this Portfolio because it was a particular challenge to design a large wetland system receiving variable flows, widely varying pollutant concentrations and very high total N, which consumes dissolved oxygen and is toxic to surface flow wetlands.

Flow balancing was needed to cater for the variations in pumped raw leachate flows. Dilution was the means selected to reduce salinity levels to at most 2,000 mg/L so it would not inhibit the aeration process. This was to be undertaken by recycling stormwater runoff by making use of Singapore’s high mean annual rainfall of 2,200 mm and the fact that it experiences at least 180 thunderstorms per annum.

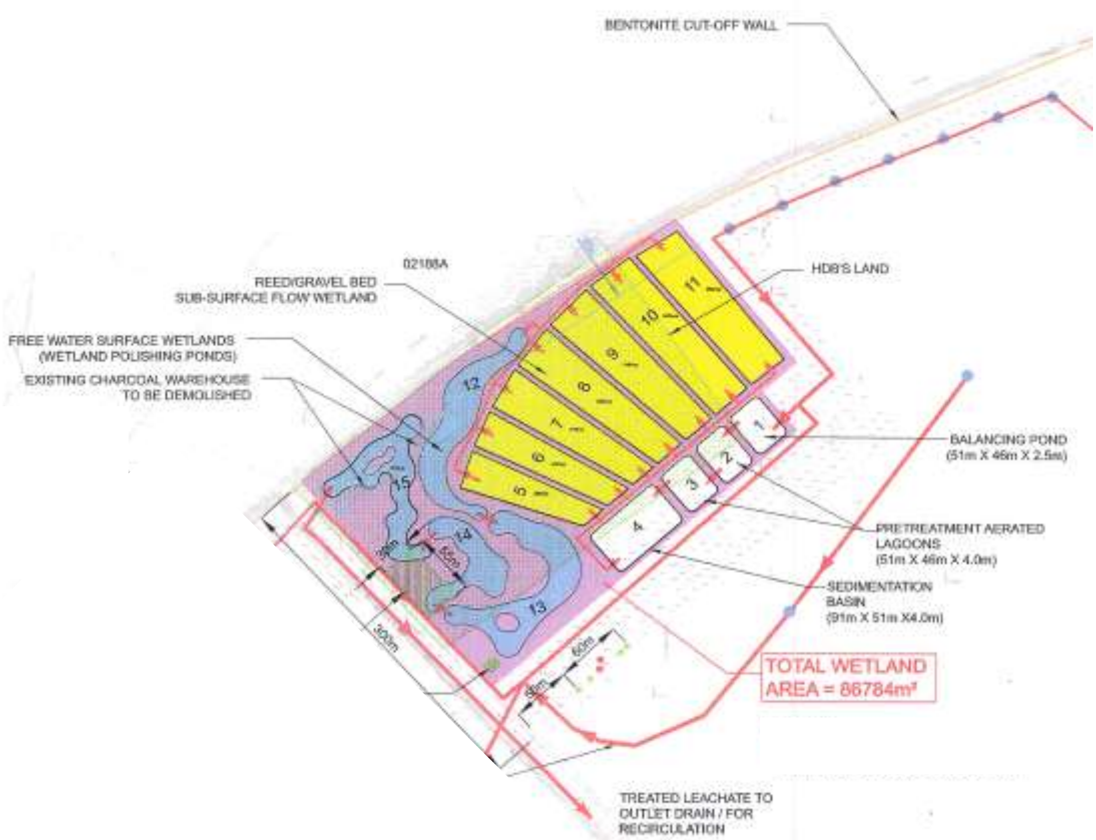

Figure 3.5: Layout Plan of Lorong Halus, Singapore Leachate Treatment System (NTS)

Aeration and settling was undertaken to break down organic matter and to nitrify the leachate. A deep settling zone was incorporated to carry out advanced settling and initiate the denitrification process. It had been found by Hervey Bay Water that denitrification can take place in deep ponds due to the anoxic conditions (Hervey Bay Water Technical Staff, pers. comm. 2006). This settling zone was followed by a large group of horizontal flow reed/gravel beds to further breakdown organic matter and to achieve denitrification. Refer to Figure 3.6 for a typical cross section of the reed/gravel bed.

The writer originally adopted a 500 mm water depth in the gravel beds. Following discussions with Dr Vymazal (pers. com. June 2006) the writer altered the water depth to 600 mm to provide more bed storage with anoxic conditions, to enhance denitrification. The design retention time for the reed/gravel beds was 3.4 days.

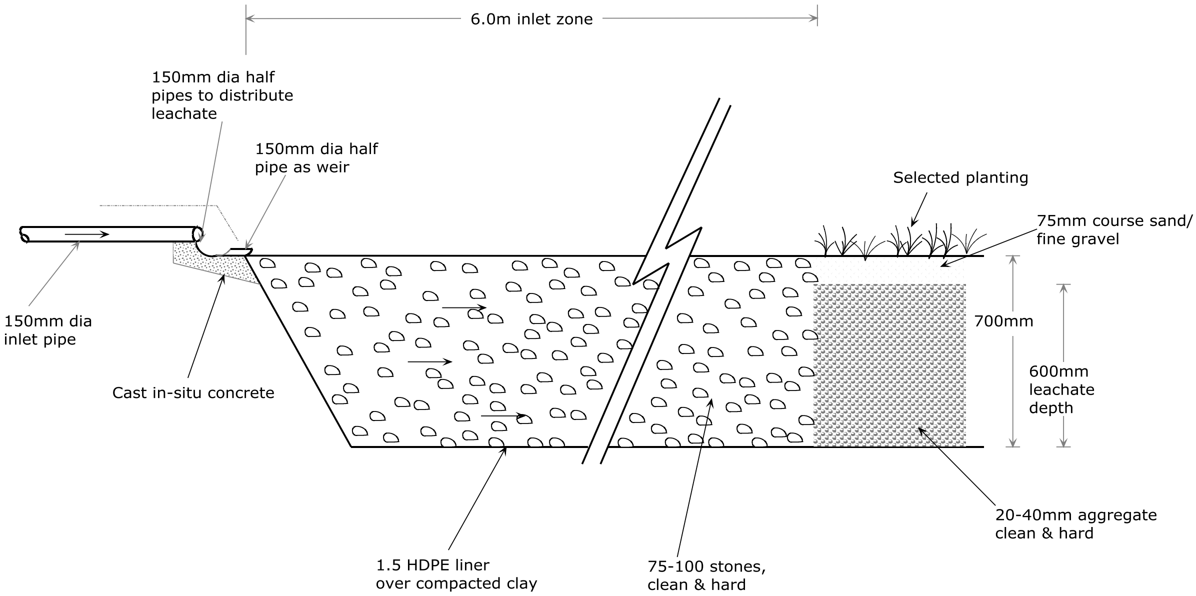

Figure 3.6: Typical Section of Lorong Halus, Singapore Reed/Gravel Bed

A carbon source is needed initially in reed/gravel beds to enhance denitrification. Since young reed/gravel beds lack natural litter and other forms of carbon, several sources of carbon were investigated. The alternatives are discussed later in Section 4.2.2. There was a definite need to reduce the ammonia-N to a level of around 3 mg/L since the treated leachate was to be discharged to a large open surface wetland system, as part of a recreational park.

It is understood that this large leachate treatment was opened in 2011. Aeration is undertaken by Landia air jets with mixers and the wetland system has a capacity of 12,600 m^3.

This system could readily be applied to a small community. In the case of a small community the Total N, COD, sulphate and TDS levels would be much lower hence, the wetland areas would be scaled down considerably and there would be no need for aeration.

2.2.3 Kalbar Reed/Gravel Bed for Treating Lagoon Effluent

The Queensland township of Kalbar, within the former Boonah Shire Council, had a wastewater treatment system consisting of an Imhoff tank followed by oxidation lagoons. The effluent discharge limits were being exceeded in terms of suspended solids, due to the high algal content of the lagoon discharge. Limited land was available for constructing an open surface wetland, which would act an algal filter.

The writer designed a reed/gravel bed system, due to the higher rate of treatment available when compared with an open surface wetland, with a facility for capturing the incoming solids and algal matter at the inlet. This involved a product called BIO-BLOCK © which is a tubular polythene material with a high specific surface area.

This inlet facility could be readily lifted out and hosed down with recycled effluent to clear the build-up of solids and biomass (Simpson, 1999). This system was not constructed and it is understood that Council opted for dissolved air flotation to remove the algal matter.

2.2.4 Selection of Reed/Gravel Bed Plant Species

Being involved with the development of wetland technology in Queensland has included work with many ephemeral and wetland plant species. Emergent plant species the writer has selected for various reed/gravel bed projects and mostly found to be successful are listed in Table 3.4.

Table 3.4: Plant Species Selection for Queensland Reed/Gravel Bed Projects

Project	Plant Name/Plant Species	Comments
Wamuran experimental reed/gravel beds, 1993	*Phragmites australis* *Typha spp* *Pistia stratiotes*	*Phragmites*, in particular, had problems establishing when treatment greywater alone. This was also experienced by Marshall (1999) in Northern NSW. The emergent plants performed much better when the greywater and blackwater were mixed. The *Pistia stratiotes* was used in a small open surface cell with considerable success in taking up P.
Oxley Pilot experimental reed/gravel bed	*Baumea articulata* *Carex fascicicularis* *Philydrum lanuginosum* *Schoenopletus mucronatus*	Assisted on species selection as a member of the project team.
McLean House, Woodford, August 2007	Canna lilies or similar	Aesthetic value was not a priority.
Ferris House, Commissioners Flat, February, 2007	Canna lilies or similar	Aesthetic value was not a priority
Van Rhyn House, Sandy Creek, September 2007	Canna lilies or similar	Aesthetic value was not a priority.
West House, Wamuran, April 2005	Selection of exotic Canna lilies	The exotic species had many colours.

Kwarciayi House, Cedarton, December 2004	Canna lilies or similar	Aesthetic value was not a priority.
Larong Halus Landfill, Singapore, 2006	*Typha augustifolia* *Phragmites karka*	These species were selected based on a field survey and discussions with Singapore Government Botanists.
Holcroft, House, Stanmore, April 2008	Canna lilies or similar	Aesthetic value was not a priority,
Grigor House, Stanmore, October 2009	Canna lilies or similar	Aesthetic value was not a priority.
Stanton House, Neurum, April 2008	Canna lilies *Lomandra spp* *Carex fascicularis* *Swamp foxtail*	This multiple plant selection has more aesthetic value.

Comments on plant selection for reed/gravel beds are as follows:

1. Earlier selection was based on the success of Victorian reed/gravel beds vegetated by *Canna* lilies. This species can be a seasonal grower in temperate climates as Victoria and New Zealand but during winter the roots still serve as a treatment enhancement mechanism. *Canna* lilies are a robust species with higher transpiration qualities
2. The use of four plants in the Oxley experimental reed/gravel system was based on the understanding that species diversity is a key objective, particularly in open surface wetlands
3. The use of four species for the Stanton house was based on the fact that the client desired species diversity
4. The Lorong Halus Landfill reed/gravel beds plants are a good example of the importance of selecting plants based on local knowledge and appropriate climate, in consultation with the local botanists.

The writer has had discussions in November 2009 with Brian Boreham, Plumbing Inspector, Moreton Bay Regional Council. He has a reed/gravel bed in place at Mt Mee, Caboolture for a number of years and has experimented with several plants. The most successful plant has been the *Agapanthus* (white and blue). The writer concurs with Brian Boreham since he used *Agapanthus* plants for evapotranspiration trench and beds systems in the late 1970s in Northland, New Zealand. ET systems are very similar to reed/gravel beds in construction. *Agapanthus* plants are evergreen, have large leaf areas and well development stomata; qualities which all lead to efficient transpiration.

2.2.5 Importance of Nitrification and Denitrification

Over the past 15 years, some of the writer's interest has focused on successful nitrification and denitrification in reed/gravel beds. The writer's understanding is that successful nitrification is readily achievable if detention times are sufficient, suitable emergent plant species are used and oxygen is able to be transferred to the roots and rhizomes. However, the technical challenge comes when denitrification is required to achieve substantial Total N reduction in effluent. The nitrogen cycle in wetlands is complex. Nitrogen transformation in wetlands occurs by five principal biological processes: ammonification, nitrification, denitrification, nitrogen fixation and nitrogen assimilation (Kadlec and Knight, 1996; Reed et al, 1995).

The technical background and discussion that follows outlines how Australian and overseas experimental work gave the writer some guidance on how to achieve denitrification in reed/gravel beds.

Early Experience

Early experiences reporting low nitrogen removal by Chalk (1989) and Bayes et al (1989), coupled with a focus on Total N nitrogen removal in New Zealand and some States of Australia, had resulted in some experimental work being undertaken. Some earlier reed/gravel bed designs focused on concentration reductions in BOD_5, suspended solids, nitrogen and phosphorus. Consequently, some earlier models were not based on removal process functioning in the wetlands.

The writer took an interest in the other early attempts on nitrogen reduction by Davies and Hart, (1990) and Cottingham (1993) within reed/gravel beds in Victoria. These researchers probably led the earlier nitrogen removal work in Australia. The writer also took an interest in the work of the Southern Cross University on reed/gravel treatment, which included nitrogen reduction (Davison et al (2000); Headley et al (2001); Bayley, et al (2003); Davison et al, 2001). From an international perspective, it was reported by Harberl and Perfler (1991) based on experimental work in Austria that nitrogen and phosphorus reduction varied considerably. In the case of Total N, in most instances, this was due to poor nitrification. This indicated to the writer that there was a need to focus on more favourable conditions for denitrification.

The nitrogen cycle in wetlands is complex. Nitrogen transformation in wetlands occurs by five principal biological processes: ammonification, nitrification, denitrification, nitrogen fixation and nitrogen assimilation (Kadlec and Knight, 1996; Reed et al, 1995).

A description of the nitrogen and denitrification processes is provided next since it is very important to have this chemical/biological understanding. A basic understanding of the processes for the removal of carbon and nitrogen are critical for optimising the treatment efficiency in reed/gravel beds (Burgoon et al 1995).

Nitrification

Nitrification is a twostep process catalysed by *Nitrosomonas* and *Nitrobacter* bacteria. In the first step, ammonia is oxidised to nitrate in an aerobic reaction catalysed by *Nitrosomonas* bacteria. The nitrite produced is then oxidized aerobically by *Nitrobacter* bacteria, forming nitrate. Nitrification is not only limited by dissolved oxygen availability. Temperature and adequate retention times also exert influence on the rate that the reactions take place. Oxygen is made available by plants, to their root zone.

Denitrification

The writer became to appreciate that denitrification occurs in reed/gravel beds when sufficient dissolved carbon is present under anoxic conditions (Bavour, 1995) but also appreciated that such conditions could be difficult to achieve without careful reed/gravel design and operation. Denitrification readily occurs in wetland systems when sufficient dissolved carbon is present under anoxic conditions. Available carbon for the process is produced during microbial hydrolysis of organic suspended solids and plant material, essentially a self-composting process. Oxidised nitrogen removal efficiency of greater than 95% is realistic. Denitrification occurs in the anoxic zones of the substrate and litter layer (Bavor, 1995).

Research that the writer assisted with or had a detailed knowledge of, undertaken by the Queensland Artificial Wetlands Research Program found that the lack of a carbon source, particularly during the initial wetland commissioning and development stages, was often a constraint.

Denitrification occurs when the nitrate is reduced in anaerobic conditions to gaseous forms. This reaction is catalysed by the denitrifying bacteria. Denitrification requires nitrate, anoxic conditions and a readily biodegradable carbon source (DLWC, NSW 1998). The lack of a carbon source initially is often a constraint. (Simpson pers. exp) and findings of the Queensland Artificial Wetlands Research

Nitrogen Reduction and HRT Correlation

The Artificial Wetlands Research Program committee also found that there was a general correlation between nitrogen reduction and hydraulic retention time (HRT) as reported by Greenway and Woolley (1999). There is a general correlation between nitrogen reduction and hydraulic retention time (Greenway and Woolley, 1999; Sakadevan et al, 1995). This was also noted by the writer during the Queensland Artificial Wetlands Research Program. For nutrient and bacterial reduction, the greater the retention time the better the reduction efficiency. M of P (1990) also confirm that Total N removal efficiency is highly dependent on hydraulic retention time (HRT) and it deceases significantly at HRTs of less than about 5 days.

Need for Oxygen

The environment within a reed/gravel bed is mostly anoxic and anaerobic. The availability of oxygen for the oxidation of carbon and nitrogen is a limiting factor in the effectiveness of reed/gravel beds (Burgoon et al, 1995). Some excess oxygen is supplied to the wastewater by the roots or rhizomes of emergent plants. This oxygen however, is used up on in the biofilm growth on the roots and rhizomes and it is unlikely to penetrate very far into the wetland media. Therefore, reed/gravel beds are effective for nitrate removal (denitrification) but not so effective for ammonia oxidation (nitrification) as oxygen availability is the limiting step in nitrification (DLWC, NSW 1998).

2.2.6 Design Rationalisation of Reed/Gravel Beds

The writer has perceived for some time from his industry-based projects that there is a need to develop criteria and a rationale for the design of horizontal flow reed/gravel beds within New Zealand and Australia. The Queensland DNR publication *"Guidelines for Using Free Water Surface Constructed Wetlands to Treat Municipal Wastewater*" produced in September 2000 gives limited attention to reed/gravel beds. The writer was responsible for the compilation for the reed/gravel bed section of the Guidelines which was based on the experience of the following:

1. The work of Crites (1992)
2. The experience from the household scale pilot system in Wamuran, Queensland
3. His practical knowledge of reed/gravel bed systems in Victoria and New South Wales
4. Experience in the USA and Europe (via Davies, 1988 and pers. comm.)
5. The writer's design experience of Queensland systems in the Moreton Bay Regional Council and Somerset Regional Council.

A summary of horizontal flow reed/gravel bed design parameters and criteria the writer has compiled is presented in Table 3.5.

Table 3.5: Typical Reed/Gravel Bed Design Parameters and Criteria

Design Criteria	Value (Reference)	Comments
Detention Time[1]	3-4 days (DNR, 2000) 3-4 days (Davies, 1991) 2-7 days (Crites, 1992) 2-6 days (Trotta and Ramsey, 2000)	(1)

BOD loading	Maximum of 80 kg/ha/d (Crites, 1992)	
Suspended Solids loading	0.04 kg/m^2/d (DNR, 2000)	Set to avoid bed media clogging. Not to exceed 0.08 kg/m^2/d.
Water Depth (within the stone media)	500 - 600 mm (Crites, 1992) 600 mm (Davies, 1991) (DNR, 2000) (2)	
Media Depth	750 mm maximum (Crites, 1992)	Includes a top cover of fine gravel or course sand
Aspect ratio	2:1 to 10:1 (Crites,1992) 4:1 to 6:1 but can be up to 10:1 (DLWC,NSW, 1989 p339) 3:1 (Trotta and Ramsey, 2000)	Considered by the writer to be very relevant, to avoid short-circuiting and to follow a definite flow path. (3)
Bed Gradient	0.1% (Crites, 1992) Up to 0.5 % (Trotta and Ramsey, 2000)	The writer has designed household scale reed/gravel beds with level beds. The writer considers that bed gradient is more appropriate to larger scale reed/gravel beds,
Plant Species	Phragmites australis, Typha spp, Canna Lilies (Browning and Greenway, 2003) Ginger lily, Calla lily, Iris	Used by Simpson, SE Qld and Davies, Victoria.
Planting diversity	Encourage diversity to develop a more diverse ecosystem and better potential resistance to disease and animal and bird attack (King and Mitchell, 1995).	The writer considers that plant diversity is more appropriate to larger scale reed/gravel beds.
Rule of thumb surface areas	4.0 m^2/EP - Qld, NT and Northern NSW (Davison, 2001; Marshall, 1999; Dallas, 2005) 5.0m^2/EP – Temperate climates Based on the writers experience.	

Inlet Zones – rock sizes	40-75 mm river stones 100 mm 70-90 mm (Davies, 1992)	Used by Simpson and former Caboolture Shire Council designs. Used by EPA Victoria
Gravel media sizes	13-75 mm (Crites, 1992) 20-30 mm 20-30 mm (Davies, 1992) 5-10 mm (IWA, 2001) 6-12 mm (IWA, 2001) Used by Simpson and former Caboolture Shire Council designs.	 Used by EPA Victoria UK practice European practice

Notes:

1. Detention times very much dependent on the pollutant being treated. Times required for BOD_5 and SS reduction are shorter than the times required for nutrient reduction (DNR, 2000; Greenway and Woolley, 1999).
2. Refer to Section 3 for some rationale
3. Long wetland drains are very effective, (Dr. Cath Bowmer, CSIRO pers. com. 1993).

Long wetland drain found to be very good for leachate treatment at the Whangarei City Landfill, where the writer was the Project Engineer in 1979. This interesting early experience convinced him that natural wetlands had potential for treatment.

One of the fundamental wetland design aspects is to avoid short-circuiting. A deeper bed however, could cause short-circuiting but it also provides more capacity for de-nitrification. On this basis, the writer adopted a 600 mm depth for the preliminary design of large scale reed/gravel beds for leachate treatment in Singapore, on the advice of Vymazal (pers. com, 23 June, 2006). It is suggested by the writer that for smaller scale red/gravel beds that a 600 mm depth is adopted with 200 mm high timber baffles, laid perpendicular to the flow path

It was reported by IWA (2000) that rooting depths for commonly used macrophytes do not exceed 800 mm, suggesting that there was no real advantage in using deeper beds. Lithium tracing showed that upper and lower bed mixing took place. It would appear that the upward action was a result of capillary rise, as a result of evapotranspiration (ET). The writer concurs with this, based on his work in ET disposal systems, reported later in Portfolio 6.

Certain characteristics of the bed media determine the rate and pattern of the flow and the efficiency of the treatment. Trotta and Ramsey (2000) advocated that medium size range (5-15 mm) tends to work better than coarse gravel for the following reasons:

1. Compared to coarse gravel, medium sized gravel offers a greater surface area where biological treatment can take place
2. The smaller spaces between medium gravel provide better support for plant growth
3. Medium gravel is more likely to promote the slow, even non-turbulent low of wastewater through the bed
4. Medium gravel is less likely to become clogged by the accumulation of solids, compared with fine gravel or sand.

The Hamilton County, Tennessee, Department of Health realised the need for innovative solutions to help solve problem areas with failed septic tank and disposal systems. The Tennessee Valley Authority undertook a demonstration program in 1986 of a reed/gravel bed system, as an alternative for small communities. TVA (1991) reported that reed/gravel beds can be effective, reliable, simple and a relatively inexpensive option. Being an earlier system, the writer noticed the following interesting features that had been incorporated in the design:

1. The beds were constructed with a 450 mm depth, to allow for contact with root zones
2. To minimise media blockages, the inlets and outlets contained 50-100 mm stones
3. Mulch was placed over the sand planted media to retain moisture for the plants
4. Berms were constructed, 150 mm high, around the entire perimeter, to minimise runoff.

It has been appreciated for some time that the inlets and outlets zones are filled with larger sized stones or rocks, to minimize possible blocking.

2.2.7 Rule of Thumb Sizing of Reed/Gravel Beds

Rule of thumb sizing of reed /gravel beds are generally given as square metres of reed bed per person equivalent (EP). Typical sizing of reed/gravel beds are given in Table 3.6**,** which the writer has compiled for application in his project designs.

Table 3.6: Rule of Thumb Sizing of Reed/Gravel Beds

Reference	Location	Sizing in m^2/EP	Comments
Cottingham (1993)	Frankston, Victoria	3.0	Wastewater – visited by the writer in 1992.
Davison (2001)	Northern, NSW	4.0	Wastewater
Davison (2001)	Nth NSW and Queensland	2.0	Greywater

DLWC (1998) reported as Dallas (2005)	NT, SA, WA & Qld S E Queensland	2.5 to 5.0 4.0	Wastewater Wastewater
Cooper (1995)	UK	5.0	Wastewater
DLWC (1998) reported as Dallas (2005)	UK	5.0	Wastewater
IWA (2001)	Shropshire, UK	5.6	Wastewater
Brix et al (2007)	Europe	5.0	Wastewater
Vymazal (1997), reported by Davison (2001)	Czech Republic	5.0	
Albuquerque et al (2009)	Beira Interior Region, Portugal	3.0 to 6.0	
Marshall (1999), reported by Dallas (2005)	NT, SA, WA, & Qld	3.0 for tropical conditions 3.75 for sub-tropical 4.5 for temperate conditions	Wastewater
Marshall (1996)	Northern NSW	2.0	Greywater
Bayes et al (1989)	Fife, Scotland	5.0	Experimental wastewater system

It can be seen from Table 3.6 that for tropical climates the approximate sizing is 3.0 m^2/EP, for sub-tropical climates it is 4.0 m^2/EP and for temperate areas it is 5.0 m^2/EP.

The writer has adopted 4 m^2/EP for combined blackwater and greywater influent, which the writer feels is suitable in South East Queensland. The former Caboolture Shire Council has been comfortable with this sizing. This sizing also matches with the figure by Davison (2001). The writer has been using a sizing of 2.0 m^2/EP for greywater reed/gravel beds for South East Queensland and Northern NSW. This figure compares with Marshall (1996) and Davison (2001).

In summary, the degree of effluent pre-treatment and climatic conditions greatly influences the range of sizing values given. In general, sizing of reed/gravel beds by rule of thumb tends to result in more conservative designs as it consistently predicts larger surface areas (Rousseau et al, 2004, as reported by Dallas, 2005).

3 Reed/Gravel Beds Research by Others

The research literature on reed/gravel bed systems for wastewater treatment is extensive. Some research outcomes on treatment performance, especially based on studies after the Queensland Artificial Wetlands Research Program, are outlined in this section. The promotion of denitrification and nitrification processes within reed/gravel beds is an important recent development. As these processes are essential for nitrogen removal, this aspect is also further discussed.

3.1 Treatment Performance of Reed/Gravel Beds

3.1.1 International Reed/Gravel Bed Studies

Horizontal flow reed/gravel/beds are commonly used to treat municipal and domestic (single houses and clusters of houses) wastewaters as both secondary and tertiary treatment stages. A list of examples with measured treatment performance, covering a range of climates, is in Table 3.7 (Vymazal, 2009).

Table 3.7: Performance Examples of Horizontal Flow Reed/Gravel Beds

Location	Country	EP	BOD_5 (mg/L)			Suspended Solids (mg/L)		
			In	Out	% redn	In	Out	% redn
Wigmore	UK	328	5.8	1.1	81	9.7	3.6	63
Leek Woolton	UK	825	8.5	2.3	73	17.7	3.8	79
Holtby	UK	612	189	18.5	90	135	19	86
Onsov	Czech Rep.	2,100	5.9	2.7	54	12.0	5.2	57
Kolodeje	Czech Rep.	4,495	204	15	93	102	11.0	89
Agronomica	Brazil	450	979	19	98	224	104	54
Carrion de los Cespedes	Spain	229	513	67	87	304	33	89
Hasselt-Kiewit	Belgium	896	232	6.0	70	196	9.0	95
Baggiolino	Italy	96	81	7.2	91	55	18	67
Pisgah	Jamaica	90	27	13	52	57	13	77
Bear Creek	USA	2,035	9.4	1.0	89	72	3.5	95
Brondum	Demark	437	330	16	95	392	10.0	97
Glavotok	Croatia	360	427	56	87	171	32	81

The following observations can be made from Table 3.7:

1. The locations/counties (climates) and the flows vary considerably and they represent the climatic regions of New Zealand and Australia
2. BOD_5 reductions range from 52% to 98%, mostly > 87%
3. Only two sites have produced an outlet BOD_5 of > 30 mg/L, but high reductions have been achieved
4. Suspended solids removal ranges from 54% to 97%, mostly > 61%
5. Only three sites have produced outlet suspended solids of > 30 mg/L, but reasonably high reductions have been achieved
6. Locations achieving very high matching BOD_5 and suspended solids reductions are Holtby, Kolodeje and Brondum. It is interesting to note that these are all colder climate areas
7. The warm climates of Spain and Jamaica have not necessarily achieved the higher reductions. This is interesting since metabolic reactions in warm and hot climates are known to be higher. This has been discussed in more detail in Section 2.1.2.

Two long term case studies of reed/gravel beds in the Czech Republic have been undertaken and reported by Vymazal (2009). The systems were installed in 1993 and 1994. The first system was for a single house with 6 EP and the second for an 800 EP. By the end of 2007, about 250 horizontal flow reed/gravel beds had been put into operation. The main plant species used was *Phragmites australis*. The smaller bed at Zitenice showed the following reductions: BOD_5 of 95.4%, COD of 91.2%, Suspended solids of 97.3%, average Phosphorus removal of 49.1% and Total Nitrogen removal of 35.3%. The low Total N was due to the lack of oxygen, hence the lower nitrification. This aspect is discussed later in Section 3.2.

It was concluded by Vymazal that horizontal flow reed/gravel beds are suitable solutions for treating wastewater when organics and suspended solids are the primary targets.

More recently, the combinations of various types of wetlands, so-called hybrid systems, have been used to enhance the treatment effect, especially for nitrogen reduction (Vymazal, 2005).

Earlier experimental work at Holtby, UK with reed/gravel beds produced very good BOD_5 and suspended solids removal efficiencies but Ammonia-N and Phosphorus removals were low (Chalk, 1988). The outcome of experimental wetlands work at Fife, Scotland produced the following (Bayes et al, 1989):

1. Valuable insight into choice of substrate. Work was undertaken on fine and course pulverized fuel ash, quarry waste and pea gravel. These materials showed promised in terms of hydraulic conductivity
2. The hydraulic loading was based on Darcy's formula
3. Some problems of vegetation establishment
4. Very acceptable BOD_5 and suspended solids removals\.

5. A bed area of 5m^2 per person is required for <20 mg/L BOD_5 removal
6. The wetland did not provide conditions which were conducive to significant Total N removal.

Eight locations in Texas with reed/gravel beds were monitored with the objective of improving their performance (Neralla et al, 2000). The results indicated that the organic load, faecal coliform density, and the Nitrogen and Phosphorus concentrations were reduced considerably. It was concluded that the reed/gravel bed provided an effective method of secondary treatment of on-site wastewater.

The efficiency of single house reed/gravel beds was studied in Ohio, USA over a period of 1994 to 2001 and reported by Steer et al (2002). The results are presented in a summary form in Table 3.8.

Table 3.8: Treatment Efficiency of Smaller Reed/Gravel Beds in Ohio

Parameter (mg/L)	**Effluent Concentration**	**Meeting US EPA Standards**
BOD_5	<30 mg/L	89 %
Suspended Solids	<30 mg/L	79 %
Faecal coliforms	< 1000 counts / 100 ml	74 %
Phosphorus	At 1.0 mg/L	50 %
Ammonia	At 1.5 mg/L	16 %

Notes:

1. P reduction was impacted by winter conditions
2. BOD_5 was impacted by winter conditions
3. The lowest reduction during the winter was Ammonia-N
4. The results generally show that the household scale reed/gravel beds are effective for BOD_5 and suspended solids

The performance assessment of a two stage sub-surface wetland was undertaken by the Department of Agricultural Engineering, University of Missouri-Columbia and reported by Burgan et al (1994). BOD_5 and suspended solids were consistently below state discharge requirements and two-log reductions (sometimes three-log) were bringing bacterial results close to state requirement of 400 colonies/100mL.

3.1.2 Australasian Reed/Gravel Bed Studies

Studies in Australia (Davison et al, 2000) have shown that reed/gravel beds are very effective at removing BOD_5 and suspended solids with removal efficiencies of over 90 % being common after three to four days retention. After a week retention time, Total N removal efficiencies of over 50% are commonly achieved (Davison, 2001).

In a study of two septic tank- reed/gravel bed-pond systems for a school, Headley and Davison (1999) investigated the aspects of seasonal effects, system maturity and peak loadings. One system was eight years old and the other was newly commissioned. Both reed/gravel beds were effective in treating BOD_5, suspended solids and Total N. There appeared to be no seasonal impact on the level of treatment.

A study of four reed/gravel beds was undertaken by the Southern Cross University and reported by Davison et al (2001). The mean removal efficiencies are in Table 3.9.

Table 3.9: Reed/Gravel Beds – Percentage Removal Efficiencies

Parameter	Removal Efficiency Ranges (%)	Comments
Suspended Solids	56-90	The lower range is not acceptable. The upper range is very good.
BOD_5	70-93	Acceptable in the lower range and very good performance in the upper range.
Total Nitrogen	38-66	It would appear that additional treatment is required, in the form of de-nitrification, particularly in the lower range.
Faecal coliforms	87-99.8	Acceptable performances.
Total Phosphorus	42-70	Higher than many similar studies. This performance could reduce in time.

It was concluded that the reed/gravel bed studies exhibited treatment performances in-line with the literature and that they should exhibit little variation in seasonal performance under sub-tropical climatic conditions.

The depth of gravel media is an important design element. Some interesting work on reed/gravel treatment at different depths and the effect of vertical mixing was undertaken at Lismore, in Northern New South Wales, and reported by Headley et al (2005). Experience in the Lismore area showed that 5 to 7 days residence time is usually sufficient to achieve secondary treatment and 50-60 % Total N removal.

Headley et al (2005) reported on system and application related factors. The system related factors include substrate size, bed depth, macrophyte species selection, maturity of the bed and climatic conditions. The reported application related factors include hydraulic loading rate, detention time, influent concentration, degree of pre-treatment pollutant proportions.

As a result of the study, Headley et al (2005) concluded that there is was no real benefit in increasing bed depths beyond the standard 500 mm.

A research project, consisting of four reed/gravel beds, was run at the Oxley Wastewater Treatment Plant, Brisbane to enable rigorous performance analysis and modeling (King and Mitchell, 1995). Later King (2002) reported the following outcomes in his thesis:

1. It was the first time that such a comprehensive set of wastewater residence time distribution studies, with supporting hydraulic conductivity studies, had been undertaken in a horizontal flow reed/gravel bed
2. Plant roots and rhizomes had increased the bed depth by about 2% and would have decreased the overall porosity of the beds
3. Evapotranspiration (ET) rates from the planted beds were found to be significant and were, therefore, considered in the water balances studies
4. The overall results of the studies showed that the size of the bed media and the presence or absence of plants are both important factors in determining the wastewater velocities and retention time distributions
5. Initial modeling of wastewater retention time distributions showed that a single parameter model, such as *Tanks in Series*, was not really suitable. Therefore, a multi - parameter model was used, which proved to be adequate. This was the first reported use of this model.

A further research project, consisting of four reed/gravel beds, was set up at the same Oxley site to study the following aspects:

1. Nutrient removal efficiency
2. Nutrient storage in plant biomass
3. Effect of cropping on plant regrowth
4. Effect of gravel size on wastewater treatment and plant growth.

The writer was part of a small advisory team that helped to set up this project. The wetland cells were planted with *Baumea articulate*, *Carex fascicularis*, *Philydrum lanuginosum* and *Schoenoplectus mucronatus*; all native species.

The following conclusions were reported by Browning and Greenway (2003):

1. Removal efficiencies ranged from 22 to 37% of NH_4-N, 22 to 75% of NO_X-N and 1 to 10% of PO_4-P
2. *Carex fasciularis* was the most suitable plant species in terms of regrowth
3. Plant biomass was the highest for *Baumea* and *Carex.*

Craven and Davison (2001) reported on the results of a preliminary study on nutrient loadings and treatment performance on four horizontal and vertical flow reed/gravel beds systems, located on the Far North Coast of NSW. The percentage removals are in Table 3.10.

Table 3.10: Percentage Removals of Four NSW Systems

System No and Description	**Suspended Solids (%)**	**BOD_5 (%)**	**Total N (%)**	**Total P** (%)
1 - Septic tank/horizontal flow reed/gravel bed (mixed wastewater)	76	92	38	26
2 - Septic tank/horizontal flow reed/gravel bed (greywater only)	89	82	83	18
3 - Septic tank/ vert flow and horizontal flow reed/gravel beds (mixed wastewater)	81	93	53	43
4 - Septic tank/ sand filters/ horizontal flow reed/gravel beds (mixed wastewater)	99	99	44	79

The following conclusions can be drawn from Table 3.10:

1. System 2 produced the best Total N removal of 83%
2. System 4 produced the best removal for suspended solids, BOD_5 and Total Phosphorus - a tertiary degree of treatment was achieved
3. System 4 produced one of the lowest Total N removals of 44% possibly due to denitrification not taking place
4. Overall the performances compared with other reported studies (Craven and Davison, 2001).

Cottingham (1993) arranged the construction of four beds within the wastewater treatment facility at Frankston in Victoria. The design was based on European experience. Each bed was 35 m long and 5m wide. Two beds contained 7 mm gravel and were planted in *Phragmites australis*. Two beds were filled with sandy soil and used as unplanted controls. The measured porosity of the gravel and sandy soil was approximately 46%. The Frankston beds were constructed with a length: width ratio of 6:1 in order to maintain plug flow conditions and reduce the possibility of short circuiting. Each bed included a gravel distributor section, intended to allow the ingress of oxygen and possible chemical dosing. It is interesting to note that dye testing was undertaken to establish flow paths in the beds. The dye testing showed that when the effluent entered the beds it flowed generally horizontally and at about 60% of the length it dipped towards the bottom of the beds. In the writer’s opinion, this would impact on the bed performance, particularly nitrogen removal, since the flow was below the root or rhizome zone. Cottingham (1993) recognised this constraint.

A photograph of the writer alongside the Frankston experimental reed/gravel beds in 1993 is in Plate 3.1.

Plate 3.1: The writer at the Frankston Experimental Reed/ Gravel Beds

The summary performance results for the Frankston beds over a three year period were as follows:

1. *Escherichia coli* (E coli) counts decreased by over 99%, in the planted and control beds
2. BOD_5 and suspended solids removal was generally over 70%
3. Nitrogen and phosphorus removal was generally < 20%. Aeration experiments were conducted in the gravel beds as an attempt to improve nitrogen. Nitrification increased with aeration but denitrification was limited by an apparent lack of a suitable carbon source.

A study was undertaken through Otago University in New Zealand and was primarily concerned with the design, construction and monitoring of a reed/gravel bed. The system was functional and able to met discharge requirements (Khan, 1996). During 10 months of monitoring, the following was achieved:

1. Mean COD removal of 75%.
2. Mean suspended solids removal of 88%
3. Faecal coliform removal was of two orders of magnitude.

It was seen that that a simple reed/gravel bed was able to remove significant amounts of contaminants from domestic wastewater and which easily allowed the effluent to be irrigated with a minimal health risk.

The Western Australian Government has been investing research funds in practical cases that will assess the viability of decentralized systems within urban villages. Two types of experimental sub-surface wetlands in south Perth are described by Strang et al, (2007). These systems have been applied to recycling greywater. They have effectively intercepted nutrient flows to the groundwater and have been well operated and maintained.

3.2 Nitrogen Removal by Nitrification and Denitrification Processes

Nitrogen removal is considered to be reliant firstly on the ability of macrophytes to transfer oxygen to the root zone and secondly, the degree of contact the wastewater has with the root zone (Gersberg et al, 1986; USEPA, 1993). If, for example, the root zone does not extend to the base of the bed, the preferred flow path is likely to bypass the root zone completely (Pilgrim et al, 1992). The plant rhizomes of emergent macrophytes contain small aerobic microzones (Davison et al, 2001). However, oxygenation of the rhizosphere is often insufficient; hence incomplete nitrification causes limited nitrogen removal. As a result there has been a growing interest in achieving fully nitrified effluents.

Experimental work on nitrogen removal in reed/gravel beds at Lismore NSW by Bayley et al (2003) provided the results in Table 3.11**.**

Table 3.11: Total Nitrogen Reduction and Retention Times

Gravel Bed No	**Total N Reduction** (%)	**Hydraulic Retention Time** (days)
Gravel Bed 1	84.5	5.6
Gravel Bed 2	87.4	10.2

Comments on Table 3.11 include:

1. The high Total N reductions occur within the first year, this stabilises to more like 60%. The likely reason for this is the expanding growth of microbial and macrophyte communities during the first year (Davison et al, 2001)
2. The extension of the retention time is no obvious advantage
3. It was concluded that a retention time of 5 days achieved a 20 mg/L BOD_5 and 30 mg/L suspended solids standard of effluent
4. It was concluded from the study that it may be worthwhile experimenting with beds deeper than 500 mm. This possibility has been mentioned by the writer in Section 2.2.6.

The slow rate of nitrification was suspected to be the limiting step in the removal of nitrogen from two pilot reed/gravel beds in Victoria (Davies and Hart, 1990). Compressed air was used to aerate the two beds in an attempt to enhance the nitrification. An increase of 22 to 24% nitrogen removal was observed and further work was needed to optimise the relationship between aeration and nitrogen removal.

As described in Section 2.2.5, there may be a limited supply of organic carbon to fuel the process of denitrification, since BOD_5 and suspended solids are removed early within a wetland system (Lennon, 1993). Established vegetation, in the form of dead leaves, will contribute organic carbon but it has been found that denitrification rates are higher where there is decaying vegetation. A carbon source may have to be added, for example methanol or liquid glucose (PCPL, Sydney pers. comm.). Optional carbon sources are mentioned later in Section 4.2.2.

The average values of nitrogen components, COD and suspended solids of final effluent from a horizontal flow reed/gravel bed system in Capinha, Portugal are in Table 3.12 (Albuquerque et al 2009). The design EP was 800 and pre-treatment was undertaken in an Imhoff tank.

Table 3.12: Nitrogen Component Reductions in Capinha, Portugal Reed Bed

Parameter (mg/L)	Influent (mg/L)	Effluent (mg/L)	% Reduction
COD	413.6	140.4	66
Suspended solids	118.6	51.7	66
Total N	31.0	7.4	76
NH_4 - N	26.8	5.7	79
NO_3 - N	1.6	0.45	72

Comments on the results in Table 3.12 are as follows:

1. COD and suspended solids reduction in the reed beds was reasonable considering the Imhoff tank was responsible for some reductions
2. Total Nitrogen reduction was very good considering the Imhoff tank was responsible for limited reduction
3. NH_4 -N and NO_3-N reductions were very good.

A pilot scale for treating a mix of municipal and industrial wastewater was undertaken in Beijing, China (Li et al, 1995). This tested the viability of reed/gravel beds for long term operation in cold weather. The removal of nitrogen was slower in colder weather than in warm seasons. The annual average removal of Total N, Nitrate N and Ammonia N was 64.6%, 73.2% and 59.4% respectively. Nitrification occurred and because there a good supply of carbon in the beds, de-nitrification also occurred.

Research was undertaken on nitrogen removal in a reed/gravel bed in Stephenville, Texas and reported by Johns et al (1998). The results indicated that about 1.53 kg/ha/day of ammonia-N was being achieved. This removal rate indicated that there was significant potential for using reed/gravel beds to reduce nitrogen in wastewater.

After two decades of experience with reed/gravel beds in Denmark, it has been concluded by Brix et al (2007) that this technology consistently provides reliable removals of suspended solids and BOD_5. The horizontal flow reed/gravel beds do not however, produce a nitrified effluent. Newly constructed reed/gravel beds in Denmark are compact vertical flow systems that have a much higher capacities per unit area and they provide a nitrified and denitrified effluent (Brix and Arias, 2005; Brix et al, 2007).

The vertical flow reed/gravel beds are not discussed in detail in this portfolio, but this concept is capable of achieving 90% nitrification and the necessary reed bed area is 3.2 m^2/EP with an effective depth of 1.0 m (Brix and Arias, 2005). Breen (1997) investigated vertical upflow reed/gravel beds, finding that planted cells removed on average 93% of Total N, while unplanted cells only removed up to 61%.

Interesting inclusions in the vertical flow reed/gravel beds are vertical aeration pipes and surfaces covered in wood chips. It is important that the transfer of oxygen to the bed media is high. Wood chips as a treatment media is discussed in Portfolios 3 and 4.

Three reed/gravel beds and one intermittently dosed sand filter were subjected to primary settled wastewater at the Lismore Wastewater Treatment Plant in NSW. All the reed beds were planted with *Phragmites australis*. Davison and Bayley (2001) reported that all systems achieved suspended solids removals of 95% or better. Reed/gravel bed 1 achieved a high Total N reduction of 84.5% with a retention time of 5.7 days, coupled with a suspended solids reduction of 97.8% and a BOD_5 of 93.8% reduction.

The value of using reed/gravel beds for treating nitrified effluent was studied over 12 months by Headley et al (2001). A retention time of 2 days achieved an 80% removal of Total N and Total P. The reeds played an integral role in nutrient removal. The reed beds generated enough organic carbon to fuel de-nitrification, with an up to 41% removal. Evapotranspiration losses from the reed beds ranged from 130 to 327 mm per month, with crop factors between 1.1 and 4.2.

Experimental research was undertaken by Kurup (2007) on a reed/gravel bed planted with *Monto vetiver* grass, and an unplanted bed, using high strength and high nutrient meat wastewater. The ammonia-N removal ranged from 40 to 60%. The main pathway of the ammonia-N appeared to be sorption to the gravel media. De-nitrification appeared to be the main pathway for nitrate removal and the limiting parameter was the carbon source. Over 80% of nitrate was removed in 48 hours. *Monto vetiver* was suitable for the nitrification process but this plant species did not survive for the de-nitrification process. Kurup (2007) also concluded that there were not many proven methods of sustainable P removal from reed/gravel beds, without the use of chemically enriched media, chemical precipitation and P removal in a separate filter containing media with a high phosphorus binding capacity. This Portfolio has not discussed P removal in any detail because it is widely accepted that surface flow wetlands and reed/gravel beds of conventional design have limitations. This was confirmed by the findings in the *Queensland Artificial Wetlands Research and Advisory Committee.*

Kadlec (2008) concluded that reconfiguration and the loss of vegetation in wetlands both markedly lessened the ability of the systems to process nitrogen and plant litter supplies the energy need for heterotrophic denitrifiers.

3.3 Vertical and Hybrid Reed/Gravel Beds

Recent literature points towards the application of vertical flow reed beds, as well as the introduction of hybrid systems involving multiple treatment stages, with a mix of horizontal and vertical flow reed/gravel beds. These alternatives to the basic horizontal reed beds are discussed in this Section.

Earlier comparative work on horizontal flow and vertical reed beds was reported by Breen and Chick (1989). A horizontal trench style reed bed was compared with an experimental scale vertical flow system. The system characteristics are provided in Table 3.13. Mean influent – effluent quality results of both systems are in Table 3.14.

Table 3.13: Horizontal and Vertical Flow Reed/Gravel Bed Characteristics

Characteristic/Parameter	Horizontal Flow	Vertical Flow
Design	Longitudinal flow plastic lined trench	Upflow 20 L bucket
Scale	Pilot	Experimental
Substratum	5-10mm gravel	3-7 mm gravel
Plant Species	*Eleocharis sphacellata*	*Schoenoplectus mucronatus* and *Cyperus involucratus*
Loading Method	Continuous	Batch
Retention Time	7 day	5 day
System Age	24 months	14 and 18 weeks

Table 3.14: Mean Influent – Effluent Results in Horizontal and Vertical Flow Systems

Parameter (mg/L)	Horizontal Trench In	Horizontal Trench Out	Vertical Buckets In	Vertical Buckets Out (1)	Vertical Buckets Out (2)
COD	161.4	79.1 (51%)	391.2	50.4 (87%)	23.4 (94%)
TN	57.91	47.05 (19%)	31.73	1.4 (96%)	1.63 (95%)
Amm. N	43.64	39.74 (9%)	17.35	0.05 (98%)	0.05 (98%)
TP	8.76	10.15 (19% increase)	5.42	0.24 (96%)	0.36 (93%)

Notes:

1. Denotes Schoenoplectus mucronatus planting
2. Denotes Cyperus involucratus planting
3. Bracket % denote parameter reductions

The following observations can be made from Table 3.14:

1. COD reductions are reasonable, but much higher in the vertical flow buckets
2. Total N and Ammonia-N reductions for the horizontal trench system are low
3. Total N and Ammonia-N reductions are high in both the buckets
4. Total P shows an increase in the trench. This was also experienced in the Queensland Wetlands Research Program
5. Total P reduction in the buckets is very high. In the writer's experience this was likely to be due to the short testing periods when phosphorus uptake tends to be maximised.

Clearly the performance of the experimental vertical flow buckets was far superior to the horizontal flow trench system. The trench system was outside whereas the buckets were operated under a ventilated glass house (Breen and Chick, 1989). In the writer's experience this would explain some of the performance differences with the bucket scale testing in a controlled environment.

New guidelines were developed by the Danish Ministry of Environment for vertical flow reed/gravel beds treating wastewater for up to 30 EP and were reported by Brix and Arias (2005). The Danish Department of Environment regulations required at least 90% Total N removal since they have clay soils, shallow groundwater and sensitive water bodies. The total surface area was 16 m^2 which was based on 3.2 m^2 per EP. The total filter depth was 1.4 m and it consisted of a drainage layer of 0.2 m, a 1.0 m layer of filter sand and an insulation layer of 0.2 m. The system was passively aerated by vertical pipes. There was some facility for re-circulation. The beds were planted with *Phragmites australis* and the effluent was pressure distributed.

Very useful comparative performance data provided by Brix and Arias (2005) from a single household are given in Table 3.15.

Table 3.15: Comparative Vertical Flow Reed/Gravel Bed Performances

System	**Parameter** (mg/L)	**% Removal**	**Comments**
VF (without re-circulation)	SS	91	Very acceptable
	BOD_5	92	Very acceptable
	NH_{4-} N	78	Acceptable
	Total N	43	Limited opportunity for de-nitrification in VF systems, as discussed later in this Portfolio.
	Total P	25	Study showed Total P reduction was typically 20-25 % without chemical precipitation.
VF (with 100% re-circulation)	SS	96	Marginally higher with re-circulation
	BOD_5	89	Marginally lower with re-circulation
	NH_4 - N	85	Higher NH_4 - N reduction with re-circulation
	Total N	23	Limited opportunity for de-nitrification.
	Total P	0	

The surface design of 3.2 m^2 per EP for vertical flow systems is lower than the temperate loading rates of 5 m^2 per EP suggested for the UK and Europe (Cooper, 1990; Brix et al, 2007; Vymazal, 1997) for horizontal flow reed/gravel beds. These mixed system or hybrid reed/gravel beds have much potential but the optimum combination depends on the target pollutants (Vymazal, 2007).

The performance of vertical flow reed/gravel beds treating only greywater from a 4 person household in Denmark, as reported by Brix and Arias (2005) are provided in Table 3.16.

Table 3.16: Performance Results of Vertical Flow Reed/Gravel Beds for Treating Greywater

System	Parameter (mg/L)	% Removal	Comments
VF- 17 m^2 area	SS	92	Very good result.
	BOD_5	98	Excellent result.
	NH_4 - N	76	Very acceptable for a VF system.
	Total N	46	Acceptable result for a VF system.
	Total P	64	Very good result.
VF – 8 m^2 area	SS	97	It is apparent that the 8m^2 area is well sufficient.
	BOD_5	99	Excellent result
	NH_4 - N	98	Excellent result
	Total N	63	Very good result considering de-nitrification is very limiting in VF systems
	Total P	2	Very low reduction

Note:

1. VF – denotes vertical flow reed/gravel bed

The following additional comments can be made from the performance results in Table 3.16:

1. The overall performance results are very good, considering that greywater is characterized by an in-balance of the N: P ratio, as discussed earlier in Section 2.1.1
2. The smaller area of 8 m^2 is sufficient for greywater treatment but there is very minimal Phosphorus reduction
3. The results indicate that a vertical flow system, to focus on nitrification, should be the first treatment process, followed by a horizontal flow reed/gravel bed to undertake de-nitrification.

Pollutant removal within mixed pond/reed/gravel beds or hybrid constructed wetlands in tropical climates was studied by Yeh and Wu (2009). The treatment train consisted of an oxidation pond, two serial surface flow wetlands, a cascade between the surface flow wetlands and a sub-surface flow wetland. The hybrid system performances are summarised as follows:

1. Average suspended solids reduction was 86.7%
2. Average BOD_5 reduction was 86.5%
3. Average COD reduction was 57.8%
4. Copper reduction was 72.9%
5. Zinc reduction was 68.3%
6. TKN reduction was 65%
7. Ammonia N reduction was 68%
8. Total N reduction was 53%.

Yeh and Wu (2009) concluded that the mixed system or hybrid wetland treatment systems in tropical climates can successfully reduce heavy metals, organic material and nutrients, with the exception of phosphorus. They are also environmentally sustainable, socially acceptable and cost effective processes.

Based on the research of three wetlands in Victoria, a 3 unit wetland system, operating in series, was recommended by (Davies et al, 1991). Unit 1 was a lined reed/gravel bed with a retention time of approximately 1.5 days, mainly for BOD_5 and suspended solids reduction. Unit 2 was a small surface flow wetland to provide oxygen for nitrification, with a 0.5 day retention. Unit 3 was a reed/gravel bed with smaller diameter aggregate, to undertaken de-nitrification with a 1 to 1.5 day retention. Refer to Figure 3.7 for the layout of the multi-stage system.

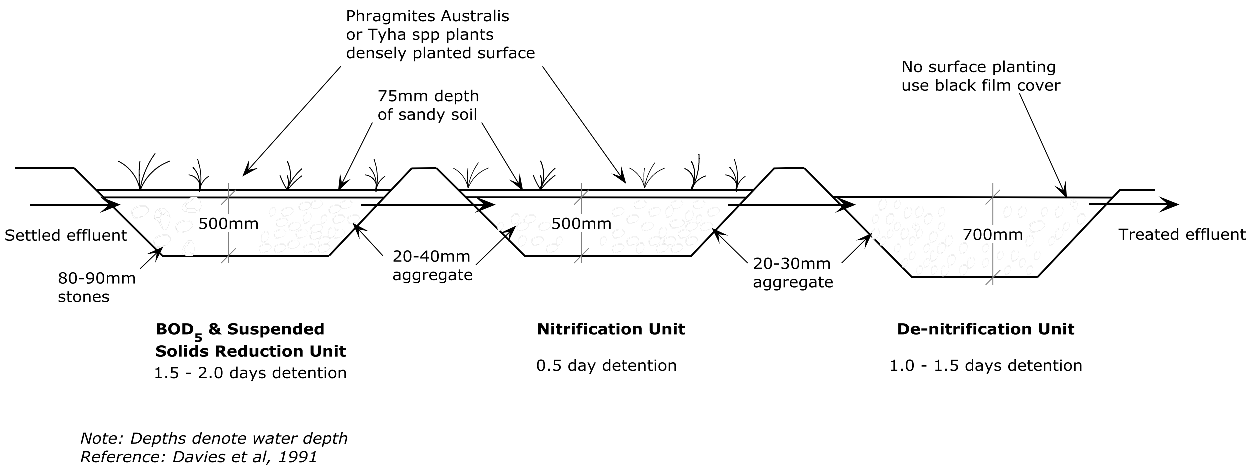

Figure 3.7: Multiple-Stage Reed/Gravel Bed (NTS)

The reed/gravel bed units have been planted with *Phragmites australis* or *Typha latifolia*. In the writer's opinion there is considerable merit in adopting this sequence of wetland cells, for achieving nitrification/denitrification. His opinion has been based on the Queensland Wetlands Research Program and work undertaken at Southern Cross University in Lismore (Davison et al, 2001) and the hybrid concept by Vymazal (2007).

The reported high Total N reduction of 84.5% over a retention time of 5.7 days in reed/gravel bed 1 (Davison and Bayley, 2001) is noteworthy. This performance was coupled with a suspended solids reduction of 97.8% and a BOD_5 reduction of 93.8% and a 99.71% faecal coliform reduction. Reed/gravel bed 1 had the following characteristics:

1. Horizontal flow
2. 10 mm gravel
3. 40 % porosity
4. A depth of 600mm, 500mm wetted depth
5. 0.75 mm polythene liner
6. *Phragmites australis* planting.

The above features are most worthy of consideration for smaller reed/gravel beds where the key pollutants are BOD_5, suspended solids and Total N.

4 Reflections and Recommendations on Reed/Gravel Beds

4.1 Reflection on Salient Points

Before reflecting on the design and performance of reed/gravel beds, the writer has made the following salient points on the content of this Portfolio:

1. Reed/gravel beds are more capable of treating higher organic loads and suspended solids than open surface wetlands
2. Reed/gravel bed technology has proceeded beyond the earlier problem of media blockages, particularly at the inlet zones (DNR, 2000)
3. There is merit in designing a reed/gravel bed based on climatic similarity
4. Higher temperatures enhance the rate of metabolism by organisms and consequently the treatment efficiency increases
5. Nitrification and denitrification can be achieved by incorporating special design considerations, by encouraging aerobic followed by anoxic conditions
6. Treating greywater in reed/gravels beds can be a problem because of the low N: P ratio which inhibits plant growth

7. Multistage or hybrid systems, comprising of horizontal flow beds, vertical flow beds and sand filters, have emerged as a viable option for more efficient treatment
8. Rule-of-thumb surface area designs for wastewater treatment of $4m^2$ per person for sub-tropical and tropical areas and $5m^2$ per person for temperate climates are suitable.

4.2 Rule of Thumb Sizing of Reed/Gravel Beds

It was earlier reported by Rousseau et al (2004) that rule of thumb sizing consistently predicts larger bed surfaces areas. Ronald John Crites, a well known authority on constructed wetlands, commented that when using the Reed et al (1988) design approach, detention times were shorter (Crites, pers. comm, 1992). On the basis that rule of thumb designs are a tool that is based on experience over several years the writer supports this approach. For example, the rule of thumb of 4 m^2 per person for wastewater reed/gravel beds in sub-tropical Queensland is considered to be sound since it has been based on the following:

1. The experience of the former Caboolture Shire Council inspectors and engineers
2. The writer's own experience in South East Queensland
3. The experience of Dr Leigh Davison and others at Southern Cross University.

On further reflection, the writer does not consider rule of thumb sizing to be an issue with single home or slightly larger reed/gravel beds, since this provides a factor of safety. However, with larger reed/gravel beds this introduces an economic issue as gravel costs are high. In these cases it is advisable to take the first-order plug-flow approach offered by Reed et al (1988) as reported by Crites (1992). He has found it to be a helpful design approach to try a rule of thumb and compare this with an alternative formula of model, and then weigh up all the factors and select a surface area.

The writer's other sizing recommendations for reed/gravel beds treating blackwater and greywater are as follows:

1. Rule-of-thumb sizing of 4 m^2 per EP for Queensland and Northern NSW and 5 m^2 per person for temperate climates. This applies to the treatment of mixed wastewater from individual household systems only
2. Rule-of-thumb sizing of 2 m^2 per EP for Queensland and Northern NSW for the treatment of greywater.

4.3 Designing Reed/Gravel Beds for Nitrogen Removal

4.3.1 General Recommendations for Nitrogen Removal

Based on the writer's wetlands and water quality research experience and a literature review, the following recommendations are made to enhance nitrification/de-nitrification in gravel/reed beds:

1. Adopt the Victorian model (Davies et al 1991) described in Section 1.1, for multiple residences and smaller institutions.
2. The effective design depth to be 600 mm
3. Install 200 mm high baffles on the wetland beds, perpendicular to the flow, to minimise short-circuiting and increase the flow path.
4. Install a system of vertical air vents, to aerate the initial or anaerobic wetland cells, to enhance nitrification
5. Length to width ration 3:1 as a minimum
6. Investigate sources of organic carbon, until such time that a build up of natural litter occurs.

On the question of reed/gravel bed depths, it was reported by Vymazal (2005) that based on studies there is no benefit in increasing the bed depth beyond the standard 0.5 m, and that shallower beds may in fact perform better. On reflection (following the writer's more recent discussions with Vymazal (pers. com 2006) and the writer designing a reed bed for leachate treatment in Singapore facilitating de-nitrification), the depth should be increased to 600 mm, as reported earlier in this Portfolio. This is in some conflict with the above original comment by Vymazal (2005). To a degree it demonstrates the value of joint discussions to find a solution to a specific problem. The agreed solution in this instance was that the extra depth would enhance the anoxic zone depth, hence, the de-nitrification process.

Also, it is recommended that the design of reed/gravel beds consider the following points:

1. Length to width ratio of 3:1 minimum and Crites (1992) quotes a maximum of 10:1
2. Select plant species with a proven ability to take up moisture and nutrients
3. Select a diverse range of plants, particularly in larger reed/gravel beds
4. Consider plants with some aesthetic value.

In revisiting the design of the Lorong Halus landfill leachate system, the writer would have considered the following process design additions and alterations to the preliminary design:

1. Extended aeration/settling processes to reduce the Ammonia-N impact on the reed/gravel beds, following pilot testing
2. Aerate the first 10% of the reed/gravel beds, using vents or forced aeration, to continue and complete the nitrification process
3. Deepen the reed/gravel beds to a water depth of 600 mm for the last 90% to enhance anoxic conditions and de-nitrification

4. Investigate the planting of emergent species with deeper root systems, to enhance treatment processes
5. Incorporate some form of carbon initially, until a vegetation based biomass became established, to enhance de-nitrification.

4.3.2 Carbon Sources in Reed/Gravel Beds

As previously discussed, the de-nitrification process in the reed/gravel bed systems requires a carbon source. It has been reported by Brix et al (2007) that the dead plant material and natural litter on the surface of reed/gravel beds improves performance, largely because they are carbon sources. The writer would certainly concur with this comment but a substitute carbon source will probably have to be introduced during the initial commissioning and earlier operational stages, to stimulate the de-nitrification process. Other sources of carbon that the writer has sourced and is recommended for use in reed/gravel beds include:

1. Sugar
2. Methanol (PCPL, Sydney)
3. Molasses
4. Liquid glucose (PCPL, Sydney)
5. Dried dog food (used by EPCO, Brisbane)
6. Methanol, maltose and acetic acid (A study the writer was associated with, undertaken by David Solley, Process Engineer, Brisbane City Council reported in February 2000 that these additives gave similar de-nitrification rates when working with landfill leachate, with a high ammonia content)
7. Wood chips and/or bark.

A carbon source can be provided at no cost by introducing a small percentage of untreated influent into the reed/gravel bed about 75% up the bed length, where most of the nitrification occurs (Thomas Davies pers. com, August 2009).

4.3.3 Oxygen Transfer in Reed/Gravel Beds

Oxygen is required in reed/gravel beds in order to enhance nitrification. On reflection, the writer considers that the inclusion of gravel distributor sections in the Frankston experimental red/gravel beds was sound planning (refer Section 3.1.2). He would also recommend the inclusion of at least three slotted PVC vent pipes within each gravel distributor to further enhance the introduction of air.

4.4 Vertical and Mixed System or Hybrid Reed/Gravel Beds

On reflection, based on the findings particularly of Vymazal (2005) and Vymazal (2007)**,** it is apparent that various types of wetlands may be combined in order to achieve higher degrees of treatment. Vymazal (2009) reports that horizontal flow reed/gravel beds suffer from a lack of oxygen therefore, nitrification tends to be inhibited. In order to enhance ammonia removal in horizontal flow systems they are commonly combined with vertical flow systems in a staged manner.

Vertical flow beds have a greater oxygen transfer capacity which provides better conditions for nitrification. But it must be noted that that very limited to no de-nitrification occurs in vertical flow beds.

The long monitoring period of two different scaled horizontal flow reed/gravel beds in the Czech Republic, as reported by Vymazal (2009) was invaluable. In the writer's experience the monitoring period of many projects is often too short due to limited availability of funding or the support of management.

Reflecting again, the optimum combination of various types of wetlands depends on the target pollutants.

4.5 Towards Model Designs of Reed/Gravel Beds

4.5.1 The Standard Caboolture Reed/Gravel Bed

Some six years ago, a need arose within the former Pine Rivers and Caboolture Shire Councils for a natural secondary treatment system to be used in sensitive water quality catchments. The writer approached the former Caboolture Shire Council to work jointly on a single household reed/gravel bed, based on the experience gained from the following:

1. The Queensland Wetlands Research Program, particularly the Wamuran pilot reed/gravel bed systems
2. The findings of the experimental work in Northern NSW, based at Southern Cross University, at Lismore. The writer feels that the climatic similarity between Lismore and Caboolture was the design key.

The writer concurred with the former Caboolture Shire Council that the loading for wastewater would be based on 4 m^2 per person. Details of a household reed/gravel bed are shown in Figure 3.8.

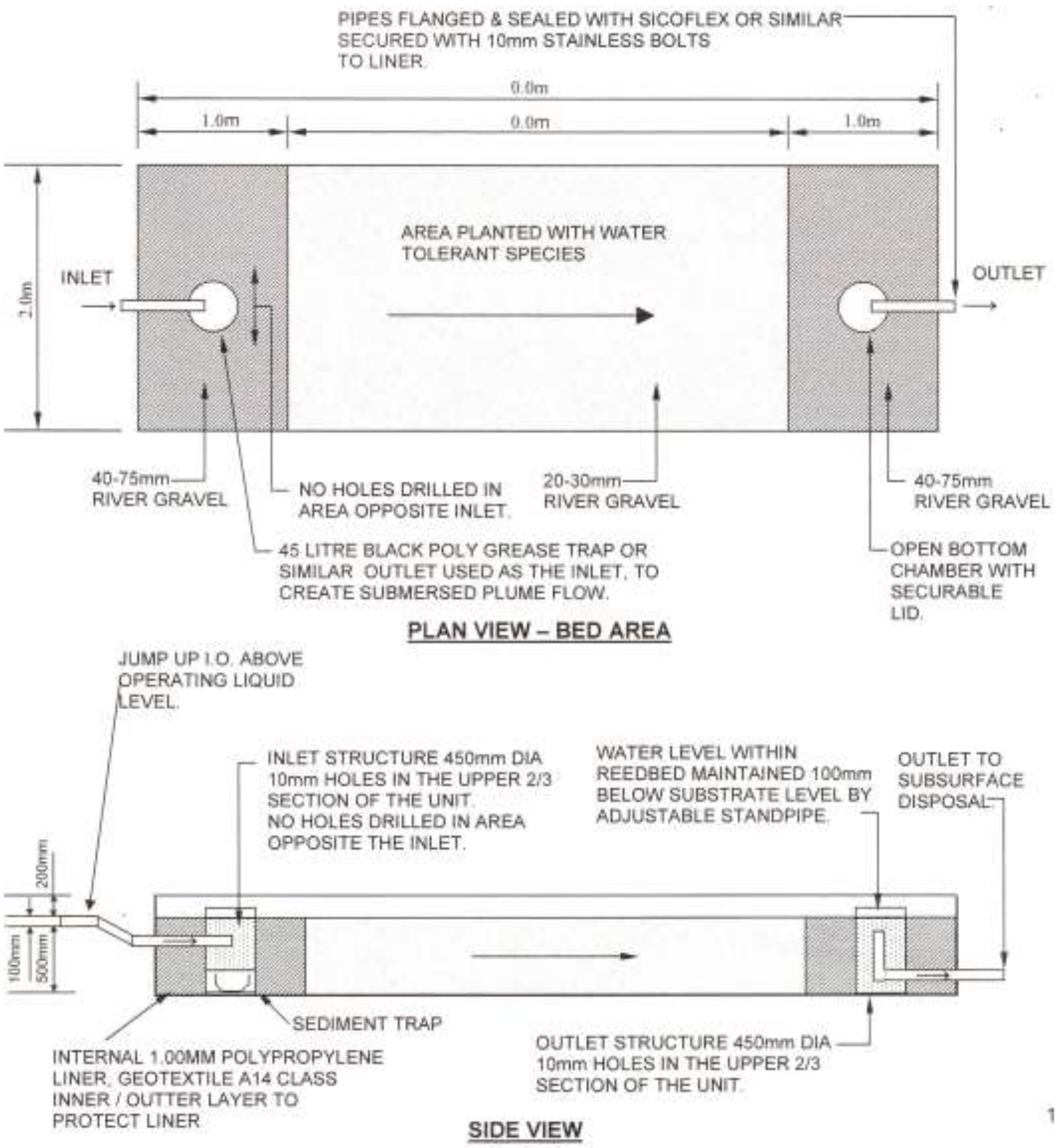

Figure 3.8: Standard Caboolture Reed/Gravel Bed (NTS)

Notable features of this "standard" reed/gravel bed include:

1. 40-75 mm stones in the inlet and outlet zones, to minimise blockages
2. 220-30 mm clean gravel bed media, with a porosity of about 45%
3. The 20-30 mm gravel has less surface area than smaller gravel, for biological treatment, but there are sufficient voids to allow the ingress of oxygen and plant roots/rhizomes
4. Traps in the inlet and outlet zones, to allow the capture and removal of solids
5. A 75 mm depth of finer gravel over the surface, to support plant establishment
6. The options of upper and lower outlets for system versatility.
7. 1.0 m long inlet and outlet zones, which are not included in the design of surface areas

8. 10 mm holes in the upper sections of each trap, facing the direction of flow, to improve trapping efficiency
9. Inclusion of a 150 mm high mound around the bed perimeter, or alternatively a cut off drain, to divert runoff.

Many of these model reed/gravel beds have been installed in the Caboolture area over the past seven years. The writer has designed at least 17 systems.

4.5.2 Enhanced Treatment using Wood Chips

By reflecting on the writer's interest in the use of wood chips for treatment coupled with his interest in de-nitrification in reed/gravel beds, he has developed a concept of reed/gravel bed polishing treatment by the three stages shown in Table 3.17 and summarised as follows:

1. Undertaking initial wastewater treatment and nitrification in a reed/gravel bed
2. Following up the reed/gravel bed with a cell, initially partially filled with woodchips as a carbon source
3. Followed by an unplanted but deeper gravel bed.

Table 3.17: Nitrification and Denitrification – Gravel Bed and Wood Chip Filter Concept.

Step No.	Process Description	Type and Degree of Treatment and Parameters	Comments and References
1	Biological breakdown and suspended solids reduction. Oxygen supplied, via the stems and roots, for organic breakdown and nitrification. Additional oxygen supplied via three slotted PVC pipes, placed vertically.	Reed/gravel bed - secondary treatment 1.0 day retention time.	Based on the rational design discussed earlier. Simpson pers.exp.
2	Filtration and further breakdown through wood chips which also acts as an economic carbon source.	Filtration and additional biological breakdown – post secondary treatment	Robertson et al (2005).

3	Further biological breakdown and de-nitrification.	Deeper gravel bed - tertiary degree. 10-15 mm diameter media. 1.25 days retention time. Other parameters based on Victoria multiple stage reed bed (unit 3).	Unplanted gravel bed, to prevent the ingress of oxygen. At least 800mm deep to obtain anoxic conditions for de-nitrification. Surface covered with bark or mulch to minimise weed growth. (Davies et al, 1991)

The writer feels that the above described reed/gravel bed and wood chip bed concept has considerable merit for further development. It embraces sound wetland technology, the requirements and environments for near completed nitrification and de-nitrification and the benefits of wood chip treatment. A tertiary standard of effluent should be produced from this entirely natural treatment concept.

4.6 Progress on Reed/Gravel Beds Since 1992

In May 1992, the writer attended the Gold Coast - Albert Sewage Disposal Alliance Seminar and provided an overview of wetland and reed/gravel bed treatment technology at that time (Simpson, 1992). As part of this thesis, it is appropriate to reflect and review the current technological state. These 1992 seminar conclusions and his matching reflective remarks are in Table 3.18.

Table 3.18: Wetland Developments in Australia since 1992

Seminar Conclusions (Simpson, 1992)	Reflective Remarks by Simpson
Wetlands have the potential for further treating secondary effluent. Further reductions in BOD_5, suspended solids, harmful bacteria and in some cases nutrients are possible.	Wetlands have the potential for treating primary as well as secondary effluents. Reed/gravel beds have been proven as higher rate systems. There is now the capability for the removal of nitrogen, particularly in reed/gravel beds, based on the work in Victoria, Northern NSW and Queensland. The removal of P is more limiting.
Until more is known about the long term performance of root zone wetlands, open water surface flow systems containing a polyculture of aquatic plants are preferred.	We have now progressed beyond this claim. A polyculture of plants is still to be encouraged. Horizontal flow reed/gravel beds should be considered in conjunction with other unit processes and/or vertical flow reed beds, to further enhance performances.

Wetlands for sewage treatment are in the developmental stage in Australia. More research is needed regarding the pollutant removal efficiency, appropriate types of aquatic vegetation, a better understanding of operational and maintenance requirements and costs and the functioning of wetlands in sub-tropical and tropical regions.	We are well out of the developmental stage. The Queensland Wetlands Research Program has provided many of the answers, lead by Greenway, Simpson and Woolley. This has been aided by such Australian researchers as Davies, Bavor, Cottingham, Breen, Geary, Davison, Dallas, and Headley. Overseas researchers making considerable developments in reed/gravel bed technology include Cooper, Kadlec, Brix, Vymazal, Crites and Tchobanoglous. All these persons are referenced in this portfolio.

5 Conclusions

Constructed Wetlands Study Tour

The constructed wetlands study tour the writer undertook in New South Wales, Victoria and South Australia in 1992 was very enlightening in terms of comparing notes with other noted wetland researchers and by the provision of information on the site selection, planning, design, monitoring and management of wetlands systems, particularly in different climatic areas (Simpson, 1993).

Queensland Artificial Wetlands Research Program

The Queensland Artificial Wetlands Research Program is seen as being a worthy investment for the procurement of wetland planning, design, commissioning and management information for Queensland and other areas with similar climatic conditions (Simpson and Woolley, 1995). The extension studies of the Artificial Wetlands Research Program in conjunction with Queensland universities gave a greater insight and understanding into the diversity of the physical, chemical and biological mechanisms occurring within constructed wetlands systems, within the range of climatic conditions in Queensland. It is appreciated that the main focus of the program was on surface flow wetlands but several of the experimental findings could be applied to reed/gravel bed systems.

Reed/Gravel Bed Viability

Reed/gravel beds are a viable treatment method and are capable for achieving nitrification and denitrification for treating mixed wastewater. In the case of treating greywater they have the treatment capability but attention has to be made of the selection of suitable emergent plant species. It is advisable to avoid using *Phragmites australus*, due to inferior growth and development and the tendency to get rust disease (Borthwick, 1995). We have progressed well beyond the early 1990s when open surface flow wetlands were preferred. There have been significant developments in reed/gravel bed wetlands, within a full range of climates.

Reed/gravel beds are capable of treating mixed wastewater as shown by Borthwick

(1995), Simpson and Gibson (1993), Craven and Davison (2001), Headley and Davison (1999) and Davison (2001). The writer concluded earlier that the reed/gravel bed system was a viable option for treating primary wastewater from individual homes, groups of homes, camping grounds, schools and other small institutions (Davies, 1988; Cottingham, 1993; Simpson and Gibson, 1993). The later work by Davison and Wallace (1999), Marshall (1996) and Dallas (2005) have confirmed this earlier assessment of reed/gravel bed potential.

Reed/gravel beds are also a suitable natural technology for treating greywater to a secondary standard, as shown by Davison (2001), Marshall (1996) and Dallas (2005).

Wamuran Pilot Reed/Gravel Bed

The Wamuran pilot reed/gravel bed system was successful in terms of treating mixed household wastewater. The effluent quality was very close to a secondary standard. Difficulties were encountered when treating greywater alone (Borthwick, 1995) and this finding was paralleled by Marshall (1996).

The writer and Borthwick (1995) both advocate that reed/gravel beds have a most promising application for the treatment of domestic primary quality of septic tank effluent in Australia and New Zealand. This applies to individual houses, clusters of houses, smaller communities and institutions, in rural or rural residential areas that often do not have access to the necessary financial resources and technical expertise required for complex treatment systems.

Lorong Halus Wetland System

A similar design concept could be applied to a small community. Aeration would not be required and the wetland areas reduced considerably.

Nitrification and Denitrification

It is important to have a full understanding of the processes of nitrification and denitrification when designing reed/gravel beds, so they produce a secondary standard of effluent. The importance of providing suitable anoxic conditions and a carbon source, for nitrification and de-nitrification, is also to be appreciated. Single stage reed/gravel beds cannot achieve high nitrification due to the inability to provide aerobic and anoxic conditions, at the same time. Vertical flow reed/gravel beds achieve very good nitrification but very limited denitrification. Hybrid units exploit the advantages of horizontal and vertical flow reed/gravel beds (Vymazal, 2009).

Multi-Disciplinary Design

For larger wetland systems, due to the complexity and sensitivity, it is important to undertake the planning and design using a multi-disciplinary team consisting of at least an environmental engineer, plant biologist/ecologist, chemist/soil scientist and a microbiologist (Simpson and Gibson, 1992 and Simpson and Anderson, 2001).

Reed/Gravel Bed Design on Square Metre Basis

The design of household scale reed/gravel beds on a square meterage per person basis, depending on the climatic conditions, is sound practice, as reported by Dallas (2005), Davison (2001) and Marshall (1999). Based on the writer's experience, he also concurs with these researchers.

Model Reed/Gravel Bed

The model reed/gravel bed, which has been developed by the former Caboolture Shire Council and in conjunction with the writer, shows promise as a household scale system, in sub-tropical and tropical regions.

Wood Chips

Using wood chips can enhance the treatment potential of reed/gravel beds and settled greywater. The writer plans to undertake further experimental work on the wood chip treatment of greywater.

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Portfolio 4 – Domestic Wastewater Composting and Greywater Treatment

This Portfolio is divided into three parts.

Part (A) covers composting of domestic wastewater and kitchen wastes with a focus on a modified Clivus Multrum system in Kerikeri, New Zealand. The waste does not included greywater which requires a separate treatment system.

Part (B) covers ventilated improved pit (VIP) privies or latrines.

Part (C) reports on greywater characterisation and treatment methods.

Part B

Part C

1 Portfolio Introduction and Aims

A composting toilet is a self-contained, waterless system for treating human and organic household waste. It uses the principles of aerobic composting to breakdown human excreta and kitchen scraps into rich humus. The process uses no external sources of water, chemicals or energy (Theios, 1979).

Publications by Simpson Appropriate to Portfolio 4

1. Simpson, J S 1977 a, Composting Domestic Sewage in New Zealand, *Soil and Health Journal*, June/July, pp. 8-9

2. Simpson, J S 1977 b, First Progress Report on Domestic Compost Convertor, R D Forbes, Rangitane, Kerikeri, NZ, January

3. Simpson, J S 1979, Candidates Report for Professional Interview Conducted by the Institution of Public Health Engineers (UK)

4. Simpson, J S 1978, Second Progress Report on Domestic Compost Convertor, R D Forbes, Rangitane, Kerikeri, NZ, October

5. Simpson, J S 1981, Clivus Composting Systems in Northland, *Alternative Sewerage Seminar*, School of Engineering, University of Auckland

6. Simpson, J S 1983, Institute of Water Pollution Control (UK), report for Professional Interview for Corporate Membership, University of Auckland

The aims of this Portfolio are to confirm firstly, that composting toilets are a suitable and sustainable system for rural/residential and isolated areas. Secondly, that ventilated improved pit toilets (VIPs) still have an application in semi-rural or more isolated areas. And thirdly, that greywater can be treated to a standard suitable for reuse.

The objectives to achieve these three aims are:

1. Document a modified Clivus compost toilet installed in Kerikeri, New Zealand
2. Document my trial of a ventilated improved pit latrine, for the Simpson Family beach home, in Teal Bay, New Zealand
3. Examine greywater characteristics and methods of greywater treatment. The treatment methods are largely based on projects the writer has been associated with or he has taken a definite interest in. His research with Dr Margaret Greenway at Griffith University was an exception when there were problems effectively treating greywater in an experimental reed/gravel bed in Wamuran, Caboolture. This situation is reported in more detail in Portfolio 3

4. Undertake a further literature review to identify more recent (since 1980) developments in composting toilets, VIP systems and greywater treatment
5. Reflect on the past and more recent findings, make recommendations for further research, and draw conclusions from the Portfolio as a whole.

Domestic wastewater generally consists of wastes generated from the toilet, kitchen sink, bath, shower, wash hand basin and the laundry. Toilet wastes, generally referred to as blackwater, makes up about 25-30% of the total household flow. The remaining waste, referred to as greywater, comprise 70-75% of the total flow. The blackwater contains the major part of the BOD_5, suspended solids, nutrients and bacteria (Ho and Mathew, 1995).

Part (A) Composting of Domestic Wastewater and Kitchen Wastes

1 Technology Background

The *Clivus Multrum* domestic compost converter was invented by a Swedish Engineer, Rikart Lindstrom, in 1939 (The Ecologist, 1972). It was reported by Rybczynski and Ortega (1975) that *Clivus Multrum* was first available in the USA during the 1970s. These devices receive faeces, urine, paper, and kitchen wastes into an air circulated container, with a sloping floor, where they are decomposed naturally by bacteria and other microorganisms.

A diagram of a standard *Clivus Multrum* system is in Figure 4.1.

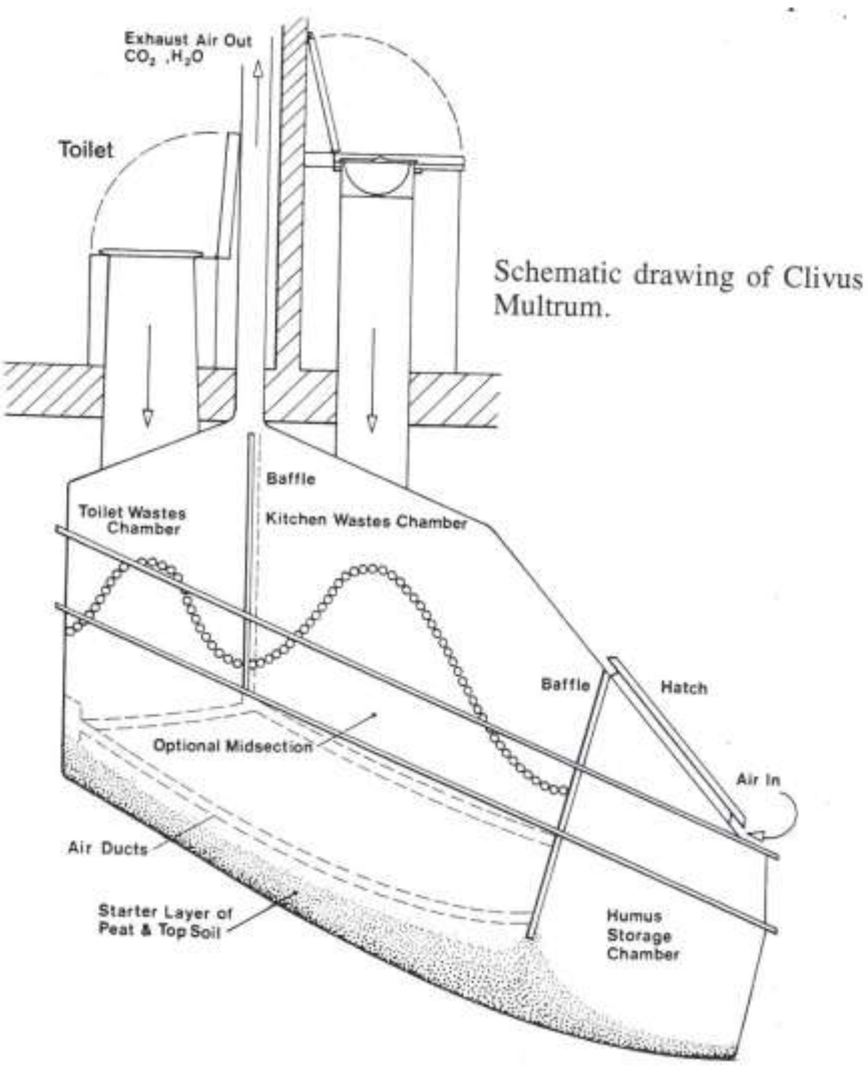

Figure 4.1: Standard Clivus Multrum (Van der Ryn, 1978)

In some cases the updraft is assisted by a fan inside the vent stack, as shown in Figure 4.1, or a wind driven turbine on top of the stack. The accumulation of liquid, along with the pile compaction, is responsible for a pile going anaerobic. This is prevented by the inclined floor. The decomposition process generates its own heat. Temperatures inside the larger units never get as high as 65 degrees C, which is considered to be ideal for good decomposition. The continuous circulation of air removes some of the self generated heat, just enough to slow down the decomposition, but not enough to stop it. In the writer's experience, some units in the colder areas in New Zealand installed insulated walls to enhance decomposition in winter.

Most moisture is evaporated off through the vent stack. The volume reduction is up to 90%. Small amounts of humus - like residue are left which can be used as a soil conditioner. Excessive quantities of urine slow down the decomposition process. In this case peat moss, sawdust or dry leaves can be added to restore the balance. A constant supply of air is important. Placing a layer of good soil or compost enhances the decomposition process. Exhaust gases are restricted to water vapour and carbon dioxide (Van der Ryn, 1978).

The *Clivus Multrum* composting toilet was originally designed to cater for holiday area accommodation in Scandinavia. However, Gunn (1977) reported that the investigation into a greater variety of models and other applications was taking place at the time. The Clivus provides an aerobic alternative to the anaerobic pit privy. The Clivus units have the additional advantage of permanence compared to the need to replace pit privies periodically. For many years the New Zealand Department of Health preferred to see the Clivus unit sited as a "sewage tank" as defined in the Drainage and Plumbing Act, 1978. To site composting toilets to meet the 3.0 m clearance requirement and yet retain convenience of all weather access required careful design of the dwelling, using a verandah or covered walkway (Gunn, 1977). This was seen as a constraint or deterrent to using a Clivus or similar composting unit, by the writer and other designers. The installation of a modified Clivus type compost converter in the Bay of Islands helped to overcome this problem.

Gunn (1977) reports that the main interest in composting toilets has been from environmentally motivated persons who feel that composting is by far the best ecological solution to human waste disposal.

2 Clivus Multrum Type Composting Research by Simpson

2.1 Kerikeri Modified Clivus Compost Convertor

An experimental compost converter was given conditional approval by the former Bay of Islands County Council in Northland, New Zealand in 1976. The writer assisted his former brother-in-law (owner) Ross Forbes to design a Clivus type converter for a residence, on the condition that a septic tank and effluent trenching would be installed, if a failure occurred.

The Ross Forbes house, with its solar heating, cooling wall and modified Clivus compost converter won a New Zealand Mobil Environmental Award and it featured on the Otago TV channel.

Special features of the Kerikeri, Bay of Island project included the following:

1. All Clivus systems in temperate countries have a small fan to enhance the removal of gases. In this case, a 6.0 m high exhaust flue was constructed from black PVC, to enhance the upward movement of gases, thereby removing the requirement for a fan
2. The climate in the Bay of Islands tends to be almost sub-tropical in summer and relatively mild in winter, with few frosts and moderate wind exposure
3. The incorporation of the kitchen waste chute, placed externally for convenience
4. The installation of a tapered stainless steel toilet pan (see Note 1)
5. The placing of the toilet inside the dwelling. Approval for compost converters had previously been granted in New Zealand, provided they were 3.0 m from the dwelling, as discussed earlier.

Note 1: The purpose of the outward tapering pan was to avoid fouling with faeces. It is to be noted that the Australian/New Zealand Standard on waterless composting toilets (AS/NZS 1546.2:2008) requires a narrow opening with a maximum dimension of 190 mm at the bottom of the pedestal. The writer has noted that composting toilets in parks and along roadsides in Queensland can have fouling problems under this standard.

Refer to Figure 4.2 for the typical cross section of the Kerikeri, Bay of Islands modified compost converter (Simpson, 1977a).

Figure 4.2 highlights some of the modifications to the conventional Clivus composting toilet mentioned above.

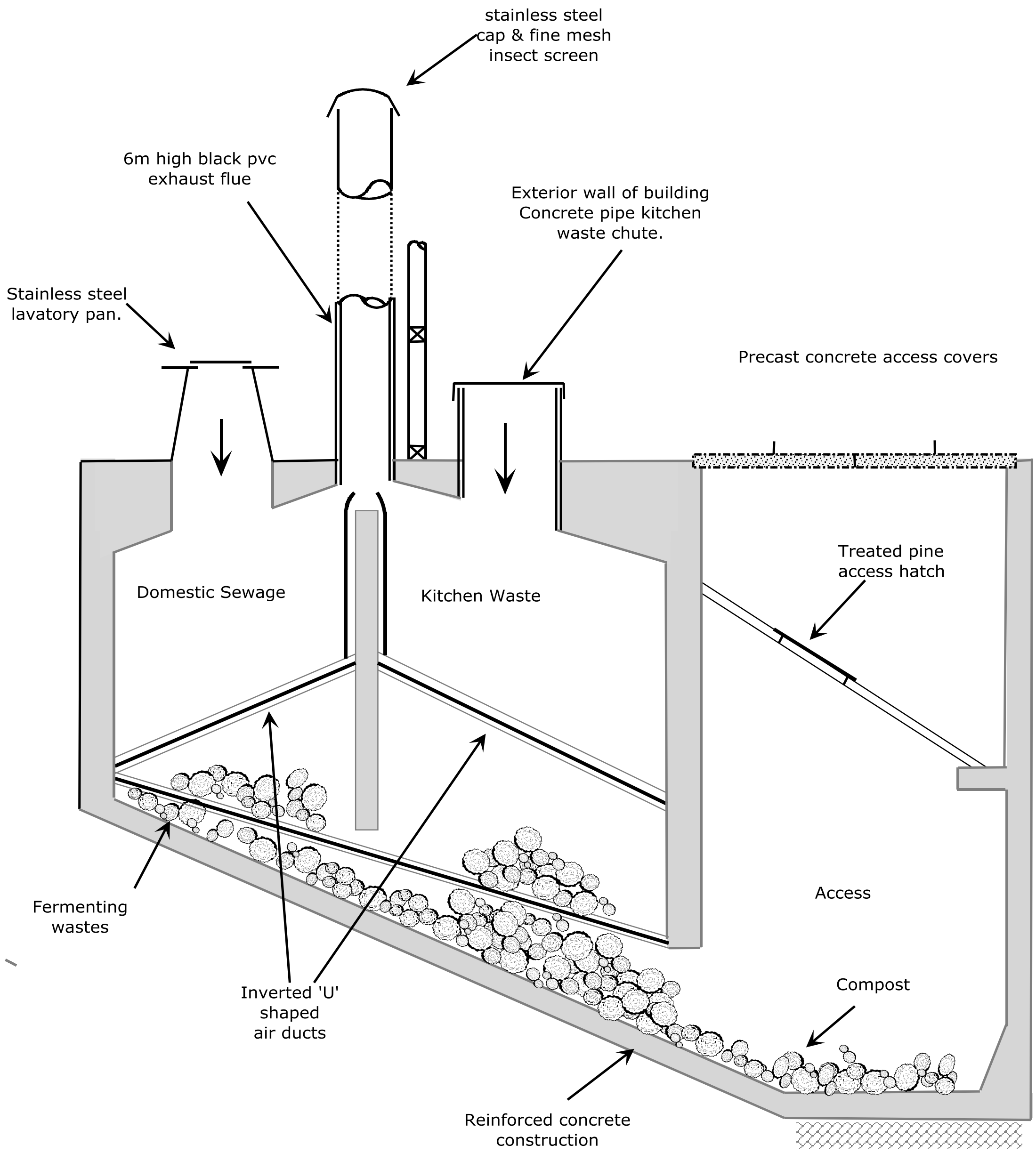

Figure 4.2: Kerikeri Modified Clivus System (NTS)

2.2 Design of Kerikeri Composting Toilet

Normally about 30% of the domestic wastewater and most of the organic kitchen wastes are treated in the compost converter. The design capacity is based on 5 persons. The design modifications and improvements, to the standard Clivus, implemented for the Bay of Islands research compost converter are as follows:

1. A completely controlled air supply system
2. An exhaust flue, which is partially inside the dwelling at room temperature, to enhance the draw of water vapour and carbon dioxide
3. The flue is composed of black PVC pipe to absorb heat and enhance upward draw

4. A 6.0 m high flue was installed to ensure better diffusion of exhaust gases
5. Installing a wind driven rotating cowl on the top of the flue. This was later removed since it was seen as being unnecessary and replaced with a weighted stainless steel fly screen.

Reinforced concrete structural design was implemented to ensure wall stability and overcome buoyancy or the tendency for the system to float when empty.

Plate 4.1 shows the Ross Forbes house at Kerikeri, with the black flue, vegetable chute and composter ventilation components, about to be installed.

Plate 4.1: Forbes House at Kerikeri, New Zealand

The writer reported that the total amount of compost generated per person per annum is equivalent to about 36 L of soil.

2.3 Functioning of the Kerikeri Composting Toilet

The system is basically an aerobic fermentation chamber for toilet and organic kitchen wastes. The fermentation process takes places in two phases:

1. Larger particles are broken down during which time heat is produced, which helps to evaporate excess urine
2. The material is mineralised; this takes some months or longer, depending on climatic conditions.

The wastes are kept aerobic by a controlled system of air venting. Exhaust gases, water vapour and carbon dioxide, are drawn up a flue and released above the dwelling. An earth bed was prepared on a 30 degree sloping floor. This contains the soil bacteria needed for the decomposition of the human and kitchen wastes. Urine is drained off by the sloping floor and decomposed by nitro-bacteria.

2.4 Advantages of the Kerikeri Composting Toilet

The installation of this experimental compost converter had the following advantages:

1. No toilet flushing is required
2. Water savings of around 40% have been estimated by Ross Forbes. A figure of up to 50% has been reported by Jenssen (2002) whereas Ho and Mathew, 1998 report a 30% savings in water usage
3. Garden compost is eventually produced. There are savings in council domestic waste collection
4. No sub-surface disposal of blackwater is required. The compost converter completely contains and treats 99% of the solids, 90% nitrogen and 50% phosphorus of the total wastewater
5. There are no unpleasant smells, but on occasions an earthy smell can be detected.

2.5 Reported Reservations of Composting Toilets

It is interesting to note the more recent reservations of composting toilets. Lustig (1990) reports on the following reservations:

1. People can be unwilling to clean the waste chute
2. Some people could be uncomfortable about looking down the waste chute
3. People may be unwilling to remove composted material every four months
4. In cold areas the heater may break down.
5. The exhaust fan may break down.

In the writer's opinion the above reservations are to be expected from some people. As a consultant he has always ensured that people proposing to install composting toilets are made aware of the above possible problems. The writer has found that these people have a particular interest in low impact technology and/or organic gardening. A heater may not be required in cold areas. Some form of insulation can be installed to overcome this need. As found in the Kerikeri modified unit a fan is not always required. A composting toilet requires very minimal energy and operation and maintenance expense, when compared with most alternative technologies.

2.6 Health Aspects of Composting Toilets

There are, unfortunately, a limited number of documented studies of pathogenic die-off within composting toilets, but these indicate consistent findings (Peasey, undated). The two most influential factors appear to be pH and residence time.

Testing by the National Swedish Bacterial Laboratory on eight samples obtained from the Kerikeri composting toilet (Simpson, 1978**;** Simpson**,** 1977) found no E coli bacteria in the final compost.

Health officials world-wide have researched the aerobic process, the toilet design and the quality of the humus end product. Their studies have found that, in composting toilets that are properly designed, properly ventilated, and properly maintained, disease causing organisms are not able to survive (Theios, 1979). According to the writer this is likely to have been due to natural predation or die-off.

To minimise fouling of the sides of the toilet pedestal it is wider at the bottom. This was a particular innovation by the owner Ross Forbes. Since the system operates under aerobic conditions, odours released are very occasional. As the inside of the compost converter operates at a lower pressure, odours are not normally released in the bathroom or the dwelling. The writer proved this aspect by observing the flow of smoke during testing in 1977.

Refer to Plate 4.2 for a photograph of the writer undertaking smoke testing.

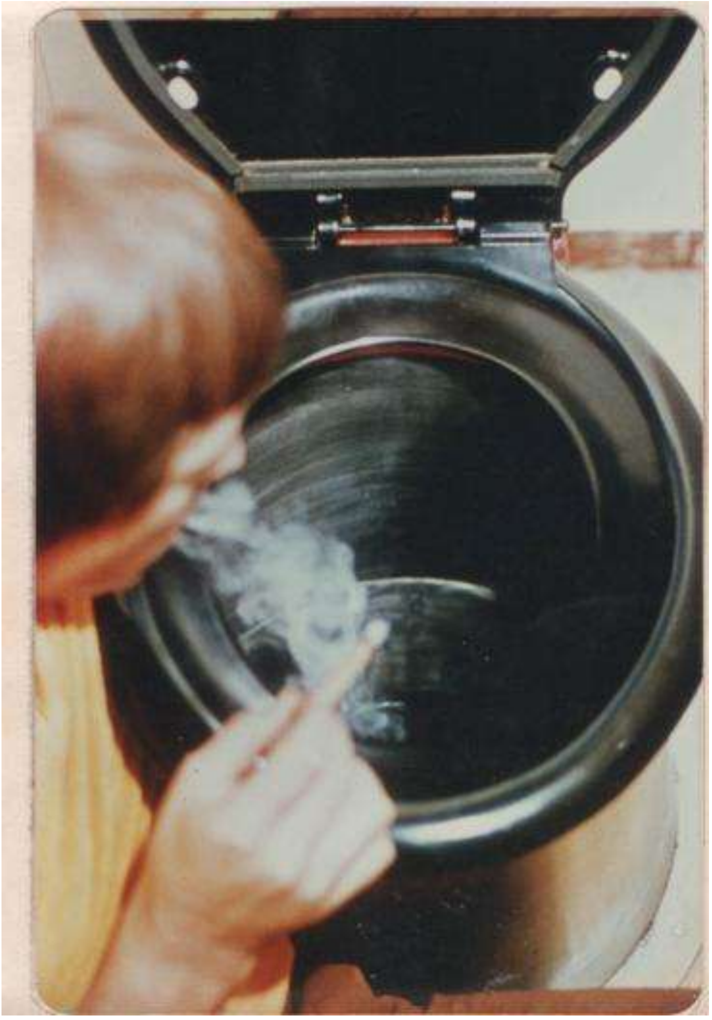

Plate 4.2: Kerikeri Compost Convertor Smoke Test

2.7 Commissioning of Composting Toilets

The following procedure for setting up the experimental unit was undertaken:

1. Spread a 200 mm layer of peat moss over the sloping floor of both digestion chambers. Peat moss can absorb liquid about ten times its own weight, information sourced by Ross Forbes at the design time. This was very important at the early stages of operation when there is a disproportionate amount of liquid relative to the accumulating toilet and kitchen wastes (Ross Forbes, pers. comm. October 2010)
2. Overlay the peat moss with 50 mm of topsoil
3. Place a further 100 mm layer of grass clippings and leaf mould. The application of grass clippings which added a substantial initial bacterial load in addition to the other layers, was a Ross Forbes initiative
4. Excessive moisture is a known characteristic of system start up. Further peat moss was placed in the access chamber to absorb excess liquid during the initial operating period.

The domestic compost converter in Kerikeri was commissioned on 25 December, 1976. Greywater was directed to a greywater tank and then to conventional shallow trenching. The writer conducted inspections, in the company of Ross Forbes, on 1 January, 1977 and 16 January, 1977. The observations that were made included the following:

1. A lack of odours and there were no signs of flies in or near the toilet
2. Smoke tests undertaken on three occasions showed a steady movement of smoke towards the toilet seat and into the first chamber (see Plate 4.1)
3. A considerable saving of 40% in roof water consumption (the only water supply available)
4. No encrustations were evident on the stainless steel toilet pan
5. No odours experienced during the still of the evening and early mornings
6. There was some odour feedback, but not major, if the kitchen waste chute was opened when the toilet seat was up. This affected the pressure balance and it was important that the kitchen waste chute was kept closed when not in use.

The following conclusions were made (Simpson, 1977a) and (Simpson, 1977b):

1. The compost converter settled to a state of biological balance fairly quickly since there was no build up of urine and odours being emitted into the dwelling
2. Domestic flies were not a problem
3. System operation and maintenance was not demanding. The toilet pan needed to be cleaned down weekly with a wet sponge
4. The research project had proven to be most encouraging and there was every indication that the Clivus type compost converter could be adapted to northern New Zealand climatic conditions, if designed and operated correctly. An exhaust fan and possibly tank insulation could be required in colder areas in New Zealand.

The writer also concluded (Simpson, 1977a) that based on limited operational experience at the time that the domestic compost converter tended to be more suitable for persons with an organic garden or self sufficiency interests. The consequence of poor operation could be a concern, compared with optional and more conventional treatment systems.

In the writer's opinion, the main concerns are that non-biodegradable refuse is placed in the chambers and poor air circulation causes the system to turn anaerobic.

Due to the interest created by this research project, copies of progress reports were sent to the University of Auckland (School of Engineering), Department of Health, Northland Catchment Commission and Regional Water Board, Auckland Regional

Water Board and the Bay of Islands County Council.

Based on further design and operating experience, the writer identified the following further merits and conclusions (Simpson, 1983):

1. The domestic compost converter, based mainly on the experience with the Kerikeri unit, was able to handle widely fluctuating loads
2. Units can function with little or no loadings for long periods without detrimental biological impacts or odour problems
3. Savings in domestic waste collection
4. Units can be located below ground level or within building basements
5. Installations could prove to be economically competitive with other options
6. Power requirements are nil and water savings continue to be about 40%
7. Final compost from the Kerikeri unit was not available until 3 years after commissioning (Simpson, 1981)
8. Brown tinted liquor accumulates in the final chamber and this has no odour. It was found that only small amounts accumulate per year and this is readily bailed out and disposed of into a garden
9. During 5 years of operation of the Kerikeri unit, problems have been minor and simple solutions have been found (Simpson, 1981)
10. The compost converter has an application in remote areas, camping grounds, caravan parks and national parks in New Zealand and Australia.

2.8 Management Aspects of Composting Toilets

Clivus systems must have fly screens. This unfortunately does not exclude small fruit flies. System monitoring by Ross Forbes (owner) found that kitchen wastes, such as fruit peelings, can attract fruit flies. If they are left on the sink, fruit flies lay eggs, which hatch out within the decomposition chambers. The answer to this problem was to apply pyrethrum spray and an occasional dressing of pyrethrum powder, which causes no detrimental effects (Simpson, 1977).

Peat moss is still used as a bulking agent. Peat moss is used as a dusting agent over the surface of the toilet chamber, not so much now for moisture control but, with its dark colour, for aesthetic reasons (Ross Forbes, pers. comm. October 2010).

3 Literature Review - Wastewater Composting Research by Others

A review of the technical literature has been undertaken for the following reasons:

1. To identify more recent work on domestic wastewater composting

2. To identify any work that supports my research and project findings
3. To stimulate ideas for the formulation of reflections for this Portfolio
4. To identify work of a more innovative nature on wastewater composting that is worthy of further research and development.

3.1 International Composting Toilet Studies

The State of California, in an attempt to better understand alternative wastewater disposal systems, initiated a study to evaluate composting toilets and greywater reuse systems. This was the first comprehensive evaluation of these systems in the USA (Walker et al, 1980). The study involved 30 composting toilets and 10 greywater reuse systems, located in 11 counties. The monitoring program involved monthly visits to each unit. For the composting toilets, Walker et al (1980) checked on the presence of odours, presence of vector problems, took temperature readings and moisture contents of the compost waste piles, and checked on the maintenance techniques. For the greywater units they checked on grease build-up, vectors as flies and midges, flow rates, drainage and for short circuiting.

The 12-month study provided a better understanding of operation and maintenance measures and the relative risk involved, when compared with conventional on-site alternative methods.

A Norwegian study of 21 composting toilets inoculated with polio virus found that zero survived after four weeks (Guttormsen, 1980).

Based on a study by the University of Texas, of about 90 prefabricated composting toilets that were installed on the USA- Mexican border, it was reported by Redlinger et al (2001) that the primary mechanism for faecal coliform reduction was desiccation rather than biodegradation.

3.2 Australasian Composting Toilet Studies

A study in Northern New South Wales by Safton (1996) found zero survival of parasites and commensals and concluded that the heap material, with the exception of viruses, could be considered pathogen free.

A survey of New South Wales councils by Schwizer and Davison (2001) identified a total of 641 households incorporating waterless composting toilets. It was apparent that this technology was growing in popularity.

A study by 20 owner-built composting toilets, built to three generic designs, is reported by Davison and Walker (2003). Fifteen of the twenty owners rated their toilets as “excellent” or “good”. The only “poor” rating came from a toilet system where bulking material was not added. The study raised a number of questions regarding the standard requirements for the composted end product. A summary of the end product characteristics is:

1. Only two units experienced odour problems
2. Salmonella organisms were not present in any units
3. The average C:N ratio varied from 6.7:1 to 18:1, the Clivus Multrum unit had a 10:1 ratio.
4. pH varied from 5.6 to 9.0.

Over 20 years ago, Lee Davison (Southern Cross University, NSW) built the first legally approved composting toilet in his state. In the eco-community where he now lives there are now 14 units producing an excellent end product with no odour problems (Meares, 2005).

Compost temperature varied from 9 degrees C to 17 degrees C, the overall average was about 14 degrees C. Studies reported by Peasey (undated) showed that the waste pile did not heat up as much as expected; therefore any pathogen die-off that occurred was not the result of heat generation.

The C: N ratio of the heap should be approximately 30:1 and the moisture content in the range of 12 to 40%. Because the C: N ratio of excreta is <10:1 and wet, it is necessary to add carbonaceous bulking material such as wood shavings, paper and straw to achieve optimal conditions for the breakdown process (Davison and Schwizer, 2001).

It has been reported by Trotta and Ramsey (2000) that composting is carried out by bacteria and fungi. These micro-organisms thrive best in warm, moist and well aerated environments.

The writer compiled a composting toilet package, with the assistance of Ross Forbes, which was posted out (in the late 1970s) to at least 60 interested persons in the north and south islands of New Zealand and Fiji. He designed composting toilets and greywater treatment/disposal systems for many persons in the late 1970s.

In New Zealand, the first place you are likely to encounter a composting toilet is in a public facility. The Department of Conservation and some councils are installing them in environmentally sensitive areas such as national parks. For example, in the Rotorua Lakes area composting toilets have been installed at camping grounds, huts and remote sites since the early 1990s (Meares, 2005). The composting toilets have sealed vaults to prevent them from leaching. Apparently the Rotorua Lakes Area has experienced problems when people use the units to dispose of rubbish or contents of chemical toilets from caravans. Otherwise, it is understood that that they are easily maintained and work well, even at peak times (Meares, 2005).

Composting toilets in New Zealand and Australia must be designed and managed in accordance with AS/NZS 1546.2 *Waterless Composting Toilets*. Composted material must be retained within the composter for at least 12 months and then buried for at least 6 months before being used for food crops.

3.3 Scale-up Challenges

Bhagwan et al (2008) reports on updated experiences with dry sanitation technologies. Once a dry composting system is full it can no longer perform its function. There is a high cost of dealing with full dry composting units. As a result, long term operation and maintenance support is required if scaling up is undertaken. Poor design and construction is likely to result in problems of flies and odours. The writer certainly agrees with these classic statements.

3.4 Management of Composting Toilets

According to Gunn (1977), because of the management responsibilities associated with supervising the on-going operation of Clivus units, it is unlikely that many New Zealanders will forgo the convenience of more traditional toilet systems. The writer concurs with this opinion and suggests that is just likely to be the situation in Australia.

If the proportion of human waste is considerably higher than the kitchen wastes, it was found to be beneficial to supplement with leaves, grass clippings from time to time. This helps to avoid the waste pile becoming too moist.

The following materials are not to be placed into the compost converter:

1. Glass
2. Metals
3. Detergents
4. Chemicals, plastics and other synthetic materials

The toilet pedestal should be regularly cleaned, using a brush, cloth and limited water. It was found that the air inlets and the exhaust flue must be fly-screened.

The waste pile should be examined weekly to ensure adequate moisture levels and to add a bulking agent if necessary. It is suggested that the pile is leveled quarterly and the compost removed from the lower chamber annually (Ho and Mathew, 1995).

The need for ongoing operation and maintenance of Clivus composting units is very important. Based on Davison and Schwizer (2001) and Stoner (1977) major problems with appropriate controls are outlined in Table 4.1. These problems can be readily overcome and are therefore not viewed as being disadvantages.

Table 4.1: Major Problems and Appropriate Control Measures

Problem	**Management**	**Reference**
Slow decomposition	Improve drainage. Increase bulking material. Low voltage heating panels have been used in colder climates to enhance decomposition rates.	Not a problem in Kerikeri unit. Tends to be sub-tropical in summer and mostly frost free in winter as near the harbour – Simpson and Forbes.
Too much moisture, with no odour problem	Improve chamber drainage. Increase bulking material.	Undertaken by Simpson and Forbes.
Low temperature	Insulate chamber. Take advantage of solar heating.	
Flies and midges	Install fly screens over air intakes and vents. If insects are too numerous use an organic insecticide.	Installation of screens needed to be undertaken by Simpson and Forbes.
Cockroaches	Sprinkle with Borax pyrethrum powder or spray.	Not a problem – Simpson and Forbes.
Odours	Improve heap aeration. Can add lime or soil to minimise odour problems. Decrease moisture content with bulking. Increase temperature.	No odour problems experienced, except very rare back draughts – Simpson and Forbes
Blocked fan	Avoid using fine saw dust as bulking agent	No fan needed in Kerikeri unit. A high black flue was very effective to draw water vapour and carbon dioxide- Simpson and Forbes.
Low air flow	Ensure air ducting not partially or fully blocked.	Experienced by Simpson and Forbes
Liquid build-up is fast with an Ammonia smell	The compost may not be sitting firmly on the bottom – stoke the compost pile. Add peat moss as an absorber media for urine breakdown and evaporation.	Not a problem in Kerikeri unit – Simpson and Forbes
DO NOTS	Put in inorganic wastes as cans, bottles and plastics. Put in chemicals, antiseptics, medicines, etc. Put in larger volumes of liquids. Put in warm ashes. Put in larger volumes of newspapers and cardboard.	These DO NOTs fully appreciated by Simpson and Forbes, at the commissioning stage.

It was found from the study of 20 owner-built composting toilets in Lismore, NSW (Davison and Walker, 2003) that most management problems were overcome by minor adjustments to management practices, on the part of the owners.

The experiences with composting toilets and greywater treatment in the ecovillage of Toarp, Sweden provided valuable management information. Whilst the greywater treatment was successful for the reduction of BOD_5, COD, Total N, Total P and thermostable coiliforms, the composting toilets were implemented without sufficient knowledge and usage directions. As a result the majority of the composting toilet had to be replaced by water toilets (Fittschen and Niemczynowicz, 1997).

3.5 End Product of Composting Toilets

The end organic product of a composting toilet undertaken by *Enviroscope Inc,* California under the auspices of the Swedish Health Department, is shown in Table 4.2 (Stoner, 1977):

Table 4.2: Constituents of Composting Final Solid Product

Component	**Content % (by weight) or as shown**
Dry substance	45 %
Humus	30 %
Total N	1.9 %
Ammonia Nitrate	0.025 %
Phosphate	0.8 %
Calcium	1.5 %
pH	7 - 8.5
Carbon/nitrogen ratio	15 - 20
E coli	0 – 100 per 100 g

The constituents of Clivus compost has been compared with other sludges and manures in Table 4.3 (Hills, 1972).

Table 4.3: Comparative Compost and Manure Constituents

Parameter	Nightsoil %[1]	London sludge %[2]	Clivus compost %	Dano compost % [3]	Farm yard manure %
Water	13.9	10.0	19.32	36.4	76.0
Nitrogen	6.74	3.0	2.13	1.13	0.64
Phosphorus	3.12	2.0	0.36	0.62	0.23
Potash	2.16	2.16	trace	0.38	0.32
Organic matter	63.7	50.0	24.49	33.0	-
Ash	3.45	35.09	52.66	30.6	-

Notes:

1. Rochdale Nightsoil is an organic manure
2. Morganic is a product from Mogden.
3. Dano Municipal compost is from Edinburgh. The writer was familiar with a Dano plant that operated in Auckland for many years

The characteristics in Tables 4.2 and 4.3 shows that Clivus compost is a good soil conditioner. It can be seen from Table 4.3 that Clivus compost, when compared with the other products, has average amounts of nitrogen and phosphorus, lower organic matter and the highest amount of ash.

The comparison of Clivus compost with chemical fertilizers is given in Table 4.4 (Lindstrom, 1973).

Table 4.4: Comparison of Clivus Compost with Chemical Fertilizers

Parameter	Clivus compost %	Chemical Fert – Vak.Oulu Y lannos %[1]	Chemical Fert – Normal Super Y lannos %[2]
K_2O	19.6	18.0	15.0
P_2O_5	16.7	15.0	20.0
Total N	12.2	13.0	15.0
NO_{3-} N	8.0	5.2	4.3

Notes:

1. Chemical fertilizers brand
2. Chemical fertilizers brand, Y lannos is from Finland

Having stated, based on Tables 4.2 and 4.3, that Clivus compost is a good soil conditioner, it can be seen from Table 4.4 that the value of Clivus compost also compares well with chemical fertilizers.

4 Reflections on Composting Toilets

4.1 Historical Considerations

It was reported that there were in excess of 1,300 composting toilet installations in Sweden by 1977 (Simpson, 1978). This gave the writer the confidence to risk the installation of the Kerikeri modified compost converter in the Bay of Islands, New Zealand.

It seemed logical that if this number of units had been installed in a cold climate, over a relatively short period, the technology must be worthy of further development.

4.2 Advantages and Disadvantages

Since it is more than 30 years since the writer co-designed and monitored the Forbes experimental compost converter in the Bay of Islands, he has taken this opportunity to reflect and further comment on the advantages and disadvantages of composting toilets, as reported by (Peasey, undated**)** in Tables 4.5 and 4.6, respectively**.**

Table 4.5: Reflections on Advantages for Composting Toilets

Factor	**Advantages**	**Comments (Simpson, pers. exp.)**
Water requirements	No water is required for flushing away human waste.	Reported household water savings are 40 % (Forbes pers.exp) and 50 % (Jenssen, 2002).
Construction	Construction is relatively simple and the upkeep is minimal	Agree that construction can be simple. Construction materials have included reinforced concrete, plaster and mesh, fibreglass and treated pine.
Spread of disease	Human waste not accessible to animals	Experience and monitoring has shown that the potential for disease transfer is very low, both during the decomposition process and with the final product.
Environmental contamination	Raised chambers with concrete floor avoids water contamination	Considered to be a major factor conducive to sound operation and water pollution control.
Environmental	End product can be used as soil	Concur that final product can be used

sustainability	conditioner. Systems reduce the quantity and strength of wastewater to be disposed of on-site. Systems divert nutrients and pathogens from the soil, surface water and groundwater.	as a soil conditioner. Concur with the reduction of nutrients and bacteria going to the soil and water resources.
Community acceptance	If the introduction of dry sanitation is gradual and time is spent on training and supporting users, dry sanitation is accepted by the community. Less distrust by children than with pit toilets.	Agree that dry toilets require a gradual educational process since they are essentially non-conventional.
Site suitability	Suitable for rural /residential and remote sites	Agree, and this can be dictated by town planning.
Power consumption	Low power need – for driving flue fans.	Agree, and fans in sites with some wind exposure and using black flues, may not require a fan.
Pollutant load	Loading on an existing communal sewerage system is reduced (Lustig, 1990).	Agree
Odour problems	Virtually no odours.	Agree, only the occasional earthy smell is detected (R Forbes pers.exp).
Kitchen wastes	Systems can accept kitchen solid wastes.	Agree, by introducing solid kitchen wastes improves the decomposition process and nutritious value of the final product. This was found with the Forbes experimental system.

Table 4.6: Reflections on Disadvantages of Composting Toilets

Factor	Disadvantages	Comments (Simpson, pers. exp.)
Usage	Successful use requires the operator and user to understand the basic principles of dehydration or decomposition involved. Can be difficult to keep the toilet basin above the chamber clean, since it is desirable to use only limited amounts of water for cleaning. Composting units must be used in conjunction with greywater treatment and disposal systems.	Concur with reported usage disadvantage. It takes co-operation not to foul the toilet basin and to manually keep it clean. In summary, composting toilets require more responsibility and commitment by the owners and users than conventional wastewater systems. All concurred by Lustig (1990). The need for an associated greywater system is so often not understood or appreciated.
Spread of disease	Incorrect usage and maintenance can result in pathogens surviving in the end product. The addition of insufficient ash, soil or lime will effect moisture and pH of the pile and result in flies and odours. Seasonal lows in ambient temperature and increased humidity can result in reduced temperature and increased moisture – reduction in pathogen die-off.	Concur with spread of disease risks. The Ross Forbes family went through a gradual learning process and they all co-operated with the usage and maintenance, to the extent no major problems have been encountered.
Cleaning of the toilet chute	The toilet can become fouled without the education and co-operation of the users.	An acceptable regular chore in my experience.
Community acceptance	Periodic addition of ash, soil or lime and periodic mixing of the pile may be considered onerous by the user. Trial periods in a community over several years are necessary to demonstrate the advantages of dry sanitation.	There has been considerable community acceptance in the use of dry toilets since the mid - 1970s. They have been installed in semi-rural townships as public toilets, national parks toilets, road side toilets and in smaller and more remote camping grounds, with the major benefit of considerable water saving. Australia's water shortage situation and the trend to be more conscious of resource reuse, has enhanced the use of composting toilets in Australia.
Smaller units	Smaller units may not have the capacity to handle peak loadings.	Agree, pile temperatures may be reduced and the moisture content more difficult to control.
Composting process stability	Improperly designed, installed and maintained systems can produce odours and poorly processed material.	Agree, it would be difficult to maintain the composting process in a state of near equilibrium.
Moisture control	The poor control of the moisture content of the pile and liquid build up can disrupt the process.	Agree, this aspect was noted during the earlier days of the Forbes experimental system. A bulking agent must be applied when the waste pile becomes too moist.

4.3 Acceptance of the Kerikeri Compost Convertor

On reflection, the planning, design and operation of the Ross Forbes compost converter in the Bay of Islands in New Zealand had considerable foresight and innovation for the following reasons:

1. The unit was suitable for siting within the dwelling, thereby overcoming plumbing and health related regulations
2. The high flue, consisting of black PVC, successfully removed decomposition odours and gases
3. The installation of a chute for kitchen solid waste enhanced the decomposition process and fertilizer value of the final product.
4. The construction of the toilet was incorporated into the design of the foundations and walls, as reinforced concrete
5. The early treatment of fruit flies and spiders was of merit
6. The care taken with the initial commissioning of the composting chamber was obviously worthwhile
7. The light brown liquid that accumulates in the bottom of the composting chamber was used to fertilise fruit trees on the property.

The writer can confirm that this experimental system helped to achieve acceptance of this low technology option in New Zealand, and eventually in Australia. Each progress report on the modified Clivus Multrum was circulated to influential authorities such as; the Department of Health, Bay of Island County Council, Northland Catchment Commission and Regional Water Board, Auckland Regional Council, School of Engineering (University of Auckland) and the Soil Bureau. The writer presented technical papers as a means of informing people and other consultants on the merits of the modified Clivus Multrum (Simpson, 1977a; 1977b; 1978; 1981). The modified technology eventually became recognised in Australia, via conferences and study visits.

Since the mid–1970s composting toilets have had some success in markets where a municipal sewerage system was either inappropriate or unavailable. The reasons for this have been the development of the various innovative and alternative technologies that provide substitutes for one or more components of a conventional system, while providing the same degree of environmental and public health protection. These systems are becoming more widely used for cost effective upgrades of old failed systems on difficult sites which cannot accommodate a conventional system (Riggle, 1996). The writer has reflected on Riggles opinion and feels that this trend has extended to the present day.

Gunn (1977) reported that composting toilets had been favoured by people with an interest in environmental issues and organic gardening. The writer feels that this is still the case. The low energy, low pollution risk and reuse of waste potential are attractive to many.

5 Conclusions

The experimental compost convertor installed in December 1976 in Kerikeri, New Zealand has proven to be a very viable system for a semi-rural and isolated situation. This system is still operating satisfactorily today which is evidence that it can function in a sustainable manner when it is well operated and maintained. This unit is suitable for use in northern North Island of New Zealand climatic conditions since temperatures rarely go below zero. Insulation or heating may be required in colder areas and a fan may be required in less exposed locations or if lower flues are installed.

The writer reported in the mid to late 1970s that the domestic compost convertor was suitable for the organic gardening enthusiast who is genuinely interested in the composting process and its end result. It is however, not generally suitable as a viable alternative to the septic tank or other conventional treatment systems (Simpson, 1977). The writer concurs with Gunn (1977) that due to the management responsibilities for the operation of composting toilets it is unlikely that many people will forgo the convenience of the more traditional treatment systems.

Part (A) has achieved the aim that composting toilets are a suitable and sustainable system for rural/residential and isolated areas in New Zealand and Australia. Today the writer holds the same opinion of composting toilets. He agrees with Meares (2005) that realistically, and given the extent of existing sanitation infrastructure, the composting toilet is unlikely to become a universal fixture in Australasian homes in the near future.

Composting toilets have applications in Australasia for individual homes where the owner understands the treatment process, for picnic areas, national parks, lodges, huts, cabins, and smaller communities. The writer feels that domestic wastewater composting technology is sufficiently advanced, over an extensive period of time, so there is no real need for more testing and research.

Based on the operational experience by *ENVIROMAT AUSTRALIA* the Clivus has considerable potential as a blackwater treatment system in the humid and tropical conditions in North Queensland. These units compensate for the humid climate if the air flow is maintained and a good C: N ratio produces an inert final product.

Part (B) Ventilated Improved Pit (VIP) Toilet

1 Technology Background

This part of the Portfolio covers ventilated improved pit (VIP) toilets otherwise known as improved pit latrines. The best known dry toilet is the pit privy, which has also been referred to the old fashioned outhouse. Since no water enters the system, greywater must be handled separately. The liquid in the pit privy pile escapes as vapour through a vent.

According to Ho and Mathew (1995 and 1998), in Australia the ventilated improved pit (VIP) toilet is a product of the Centre for Appropriate Technology in Alice Springs. Its special construction features ensure minimum odour and fly problems. It is possible for a family of 5 to use these units for 10 years.

In 1995, VIP systems were installed in outstations in Australia. According to Marshall (1999) they have proved to be very successful and are now the only toilets installed on outstations as a matter of policy.

In Australia they have been found to be suitable for camping places in National Parks, main highway rest areas and remote communities (Walker, 1985; Ho and Mathew, 1998).

Unventilated pit privies have their draw backs in that they are limited to rural areas. They must be located some distance from houses, and they can be a source of bad odours if there is too much moisture and decomposition becomes anaerobic.

Van de Ryn (1978) reported on the following recommendations for privy location, design and use:

1. Pit privies should be installed on site where there are no signs of rock fissures and where slopes are not extreme
2. Groundwater levels shall be at least 1.5 m below the bottom of the pit, during the wettest time of the year
3. The pit privy must be located at least 50 m away from any water supply
4. Pits should be wood-lined and have a minimum depth 1.5 m
5. Surface water must be diverted away from the pit privy
6. A cut off trench, filled with aggregate, must be installed above the pit privy, if there is any downhill surface slope
7. A vent must extend above the roof line
8. The vent must be fitted with a cowl to prevent rainfall entry and a screen to prevent the entry of flies and insects.

According to Gunn (1981), environmental effects of a well designed pit privy can be negligible. They have an application for:

1. Public facilities at seasonal areas (ski fields, picnic areas, scenic reserves)
2. Intermittently occupied facilities in low density use recreational areas (lakeside, bushland, mountain areas, smaller holiday homes, lodges and huts).

Rapid urbanisation in South Africa has brought thousands of people to formal settlements where water supply and sanitation services often constitute major problems for authorities. One of the adopted alternatives for low cost sanitation is the use of unventilated pit privies. However, the following aspects should be kept in mind by designers (Schroeder, 1994):

1. The flow of water through the pit – this must be avoided since the faecal matter will undergo rapid part decomposition of the soluble solids
2. Degree of pit ventilation – the extent of air exposed organic matter surface is a factor of decomposition and liquefaction
3. Flooding – there is a need to ensure that the pit contents are not washed out. The tops of the pits must be impervious to the passage of water
4. Flies – every effort must be made to exclude flies
5. Cleansing material – avoid using newspaper as a pit privy cleansing material since it occupies pit volume
6. Pit location – every effort should be made to avoid water courses, earth banks and other positions which enable disease bearing effluent to flow into areas of human contact
7. Odours – to be minimised by attempting to avoid anaerobic conditions in the pit and the associated hydrogen sulphide emissions.

The Stockholm Environment Institute (Winblad and Simpson-Hebert, 2004) has reported on VIP toilets. This system is shallow and trees and vegetables are grown on the filled pits. Each pit can last from 6 to 24 months before filling with wastes and the superstructure is moved to a new location. The odours are controlled by fitting vents and the liberal addition of wood ash, leaves and soil, which obviously shortens the effective life of the pit. The vents are fitted with corrosion resistant fly screens. Deeper pits can be brick lined.

The improvement of the *Arbloo* pit privy into a VIP toilet, by the Stockholm Environment Institute, 2009 has been reported as a pre-production draft by Morgan (2009). The portable toilet and shed is able to be shifted to an adjacent pit. Of particular note are the vent pipes which are 2.1 to 2.5 m in height. In Europe the Clivus systems are fitted with exhaust fans so the flue heights are lower than the modified Clivus in Kerikeri, New Zealand which had no exhaust fan.

Davis et al (2008) maintains that, since there are at least 10,000 VIPs in the USA, the system cannot be ignored. They offer an alternative pit construction being 1.0 m ^{2} and 1.5 m deep, constructed from 200 mm wide and 20 mm thick timber slabs with 20 mm gaps.

2 VIP Toilet Research by Simpson

2.1 Teal Bay VIP Toilet

During the early 1970s the writer can recall that the VIP toilet was emerging as an improvement to the old pit latrines for more remote areas, particularly in Africa. He saw an opportunity to experiment with the VIP system in the milder climate of coastal Northland, New Zealand.

With the consent of the former Whangarei County Council, in Northland, New Zealand, the writer designed and constructed an experimental VIP toilet for the Simpson family beach house at Teal Bay, during 1975. The family beach house, consisting of a ground floor, upper floor and mezzanine floor, was to be constructed over a two year period. Until such time as the ground floor was covered in, a VIP toilet was to be used.

The writer intentionally under-designed the below-ground capacity in order to truly test its performance. It was merely a 300 mm diameter hole in sandy subsoil, to a depth of 3.0 m. This cleared the standing groundwater level by some 2.0 m.

The VIP system was to be subjected to the following range of uses:

1. Normal daily family use
2. Use by more than one family for up to a week
3. No use for some weeks.

The VIP had a concrete slab base, corrugated iron walls and roof, concrete pipe base, PVC lid and PVC vent pipe work. A small PVC hand basin was installed, and roof water was used for washing hands. The main modification to the old pit privy was the inclusion of a ventilation system to minimise odours. No fan was used. A sectional diagram of the Simpson VIP toilet is in Figure 4.3.

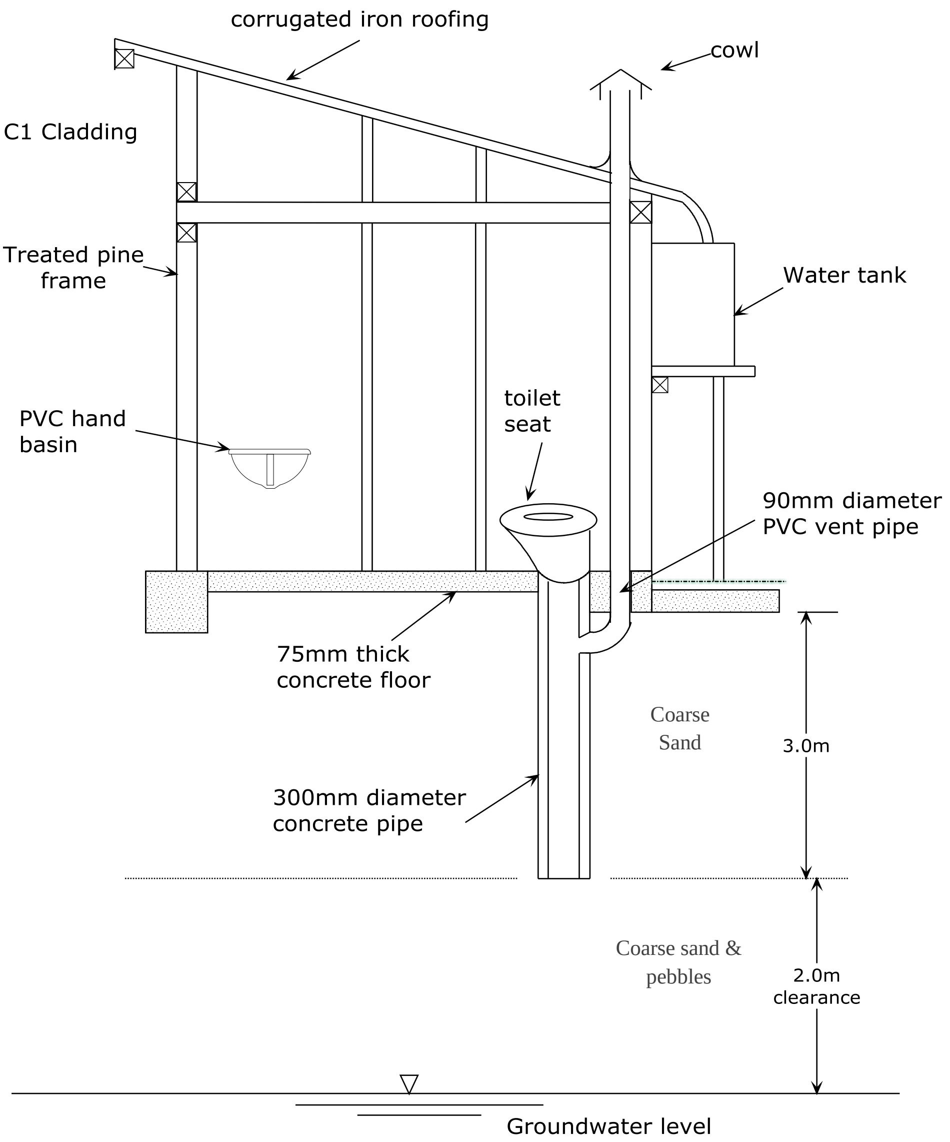

Figure 4.3: Section of Simpson VIP System (NTS)

This experimental VIP toilet had a certain uniqueness in that it was probably one of the earlier systems in New Zealand and Australia. This was seen as a major advantage since the pit privy was on a small coastal sand block and sited only 4.0 m from the beach house. The potential impact of the wastes on groundwater was not included in the project scope. During the summer, which can be wet and humid, lime was dosed on the pile when flies or odours were a nuisance. This was needed on an infrequent basis only.

The Simpson VIP system continued to be used for some years until a conventional on-site sewerage system was installed.

3 Literature Review – VIP Research by Others

3.1 Australian Experience with VIP Systems

Design modifications have been incorporated to overcome some of the deficiencies of the pit privy (Ho and Mathew (1995).

A Centre for Appropriate Technology survey of 900 Australian outstations reported by Marshall (1999) found that 32% of the systems used were either standard or improved pit latrines. They have proved to be more successful than septic tanks, since they have no moving parts, no water requirements and they require very limited maintenance. In Australia they have been found to be suitable for camping places in National Parks, main highway rest areas and remote communities (Walker, 1985).

4 Reflections on VIP Systems

4.1 Simpson Family Experimental VIP System

Based on Simpson (1983) and more recent reflections, the writer feels that the Simpson family VIP system was a success for the following reasons:

1. Flies were only a very occasional problem, during summer's humid conditions. The application of hydrated lime or sawdust very soon overcame the problem
2. The VIP was economic to construct
3. The build-up of solids did not cause any concern
4. The ventilation proved to be successful in that odours were only a very occasional problem
5. Maintenance was minimal
6. Water savings were significant, which is important for situations reliant on roof water
7. The family and guests accepted this technology.

4.2 Suitability and Life of VIP Systems

If site and soil conditions are favourable, the writer considers that the VIP system is most suitable for widely fluctuating wastewater loads in rural areas or smaller outdoor camping sites. Based on his experience he sees the key advantages of VIP systems over alternative low cost sanitation facilities are their ability to dispel odours and to minimise fly populations. These key advantages have recently been confirmed by Dumpert et al (2009).

It has been reported earlier by Ho and Mathew (1995) that the projected life of a pit privy unit, for a family of five, is 10 years. The writer feels that with an increased depth (if the soil profile and the depth of the groundwater table were favourable) and increased width or diameter, it would be feasible to extend the life of the system.

As a guide, pit life in relation to depth and diameter has been estimated by Winblad and Simpson–Hebert (2004). The estimates have been made with small additions of soil and ash, to minimise odours. The depth/diameter estimates are listed in Table 4.7.

Table 4.7: Pit Life in relation to Depth and Diameter

Pit Diameter (m)	Pit Depth (m)	Volume/m^3	Life (years)
1.5	3	5.3	17.6+
1.5	2	3.5	11.6+
1.3	3	4.0	13.3+
1.3	2	2.6	8.6+
1.1	3	2.8	9.3
1.1	2	1.9	6.3
1.0	3	2.3	7.6
1.0	2	1.5	5.0
1.0	1.5	1.1	3.6

Observations to be made from Table 4.7 data are as follows:

1. A 1.5 m diameter pit to a depth of 3.0 m is feasible to construct, for long usage
2. Using the above Ho and Mathew (1995) estimated life for a family of five, a pit diameter of 1.2 m to a depth of 2.5 m compares with Table 3.7
3. A 1.0 m diameter pit to a depth of 1.5 m would not be economic, in terms of life, to construct.

Mihelic et al (2009) report the following VIP latrine design aspects, based on more recent experience:

1. The vent pipe should be a dark colour to facilitate upward air movement. The writer concurs with this, based on the experimental compost convertor in Kerikeri
2. The top of the vent pipe should be at least 0.5 m above the roof line
3. The floor slab should be at least 150 mm above the surrounding ground.

The writer considers that this technology still has a role in rural/residential areas, farms and beach side locations. This is mainly because these are typical low density population areas. The writer concurs with Ho and Mathew (1995) that in Australia the VIP system is suitable for camping places, national parks, main roads rest areas, and remote smaller communities.

5 Conclusions

A ventilated improved pit (VIP) has definite advantages over a pit latrine since it has no problems with odours and flies. The operational key is the flow of air to enhance aerobic conditions for the breakdown of the human wastes.

The Simpson Family experimental VIP system proved to be a worthwhile trial by confirming that it rarely attracts flies or emits odours. The VIP system from the Centre of Applied Technology in Alice Springs confirmed the above merits.

The VIP system has the capability of handling intermittent and widely fluctuating loads, as shown by the Simpson Family unit and the experience of the Centre for Applied Technology, Alice Springs. It is a viable option for isolated areas where there is no water supply.

The VIP unit is seen as an economic option, particularly when compared with the alternatives.

Part (B) has achieved the aim of demonstrating that the VIP system still has an application in isolated areas, smaller and scattered communities, main road rest areas, and camping areas in national parks in Australasia.

Part (C) Greywater Treatment

1 Technology Background

This part of the Portfolio covers characteristics of greywater and methods of greywater treatment.

Blackwater, which is human faeces and urine (Clean Water Act, 1974), is treated in a domestic composting system, as covered in Part (A). Greywater is the liquid and solid wastes from domestic fixtures and water saving devices (Clean Water Act, 1974), which requires separate treatment if a composting toilet is used. It has been accepted that because of the absence of faecal matter in greywater the health risks associated with human contact are considerably reduced.

Greywater contains some lint, bacteria, viruses and nutrients from detergents, soaps and cleaners. Van De Ryn (1978) recognised that greywater, since it contained pathogenic matter, must be treated.

In a household the greywater is specifically derived from showers, baths, hand basins, washing machines and kitchen sinks (AS/NZ 1547:2000). Greywater is typically between 68% (Emmerson 1998 cited by Wiltshire, 2005) and 70% (DPI, 1996) of the total wastewater volume.

It is most advisable that greywater undergoes pre-settlement since untreated greywater can quickly clog disposal trenches. Winnebegger (1974) reports on the following functions of greywater tanks:

1. Removal of solids and other materials which include lint, hair, grease, food particles
2. Biological treatment by anaerobic means
3. Sludge and scum storage; sludge to be digested in the bottom and floating solids as a scum on the surface.

Numerous studies have been conducted on the treatment of greywater with different technologies, which vary in both complexity and performance (Li et al, 2009). However, studies on the appropriate technologies for greywater reuse/recycle are scarce and specific guidelines for greywater reuse are either not available or well compiled. One exception is the more recently introduced greywater reuse guidelines in Queensland, The Queensland Plumbing and Wastewater Code, 2008 (QPWC). Because of the potential health and environmental risks associated with the improper use of greywater, it now requires some treatment or specific management.

QPWC (2008) permits the segregation of wastewater at source, into greywater and blackwater. The code provides the technical requirements for the use of greywater for garden and lawn irrigation in sewered areas and for a greywater use facility in an unsewered area.

1.1 Greywater Composition and Characteristics

The water quality of greywater is very site specific, varying in strength and composition. As stated earlier, greywater varies in composition and reported performances, so for these reasons different studies have been reported. Generally it can be said that greywater contains low fractions of organic matter, nutrients and additionally has a low microbial contamination. Greywater has about 10% of the nitrogen and 20% of the phosphorus present in wastewater (Mullegger et al 2003).

A typical composition of greywater, compared with raw wastewater, is given in Table 4.8 (DPI, 1996).

Table 4.8: Typical Greywater and Raw Wastewater Compositions

Parameter	Unit	Greywater Mean[1]	Raw Wastewater Range
Suspended solids	mg/L	115	100-500
BOD	mg/L	160	100-500
Ammonia -N	mg/L	5.3	10-30
TKN	mg/L	12	20-80
Nitrate -N	mg/L	<0.1	5-30
Nitrite -N	mg/L	<0.1- 0.8	1-10
Total P	mg/L	8	5-30
Sulphate	mg/L	35	25-100
pH		7.5	6.5-8.0
Sodium	mg/L	70	70-300
Copper	ug/L	130	10-200
Zinc	ug/L	100	55
Hardness (Ca and Mg)	mg/L	45	200-700

Note:

1. Based on work in Brisbane City Council by Jeppersen and Solley (1994)

The University of Massachusetts studied the variability and characteristics of greywater from five commercial locations in Massachusetts (Veneman and Stewart, 2002). The summary results are in Table 4.9.

Table 4.9: Greywater Variability and Characteristics

Parameter	Mean	Range
BOD_5 – mg/L	128.9	22.1 – 358.8
Suspended Solids	53.0	8 - 200
TKN – mg/L	11.9	3.1 – 32.7
Nitrate -N – mg/L	1.5	<0.8 – 17.5
Orthophosphate – mg/L		<0.5 – 3.6
pH	7.0	5.3 – 10.8
Faecal coliforms – cu/100mL		Zero – 10,000

The results in Table 4.8 and Table 4.9 confirm the findings of Mullegger et al (2003). *E Coli* was not detected in any of the samples in Table 4.9. Greywater is generally characterised by rapidly degradable BOD_5, low nutrient levels and fluctuating high volumes (Dr Chris Tanner pers. com. NIWA, NZ). Greywater is high in dissolved salts, turbidity, and low in nutrients and it contains significant amounts of pathogens (Wiltshire, 2005).

The contaminants of greywater are further listed as follows by DOH (2006):

1. Disease causing organisms (bacteria, viruses, protozoa and helminths)
2. Suspended solids, organic matter, fats, oils, dirt, lint, food matter, hair, body cells, and traces of faeces, urine and blood
3. Chemicals derived from soaps, shampoos, dyes, mouth washes, toothpaste, detergents, bleaches and disinfectants and other products (e g Boron, Phosphorus, Sodium, Ammonia and other nitrogen based compounds).

This list shows that the constituents of greywater are diverse.

Compared with domestic wastewater, greywater contains less nutrients, as shown in Table 4.8. Domestic wastewater has a typical BOD:N:P ratio of about 100:20:5 whereas greywater typically has a ratio of 100:4:1. The optimum ratio for heterotrophic growth is 100:5:1. Therefore biological treatment of greywater without the addition of nutrients is possible (Mullegger et al, 2003).

In the writer's experience, for many years the pollutant potential of greywater tended to be underestimated in both New Zealand and Australia. Little was known about the harmful bacteria and virus contents of greywater. In fact, there appeared to be little concern for the health risks of greywater. However, the findings of Jepperson and Solley (1994) changed the picture on the risk of greywater and the need for treatment and the potential for reuse (Beavers and Gibson, 1996). The figures in Table 4.8 and Table 4.9 demonstrate that greywater has many characteristics of a weak to medium sewage and it is generally contaminated with pollutants.

It is surprising that few studies have reported on the microbial content of greywater. The work of Rose et al (1991) and Brandes (1978), as reported by Beavers and Gibson (1996), show faecal coliform levels in greywater in Table 4.10.

Table 4.10: Faecal Coliform Levels in Greywater

Faecal Coliforms/100ml		
Source	**Rose et al, 1991**	**Brandes, 1978**
Bath/shower	6 x10^{3} CFU	<10 to 2 x 10^{3}
Laundry wash water	126 CFU	-
Laundry rinse	25 CFU	-
Kitchen	-	< 10 to 4 x 10^{6}
Combined Greywater	6 to 80 CFU (1) 1.5 x 10^{5} CFU (2) 1.8 x 10^{3} to 8 x 10^{6} CFU	8.8 x 10^{5} (3)

Note:

1. (1) – denotes families without children, (2) -- denotes families with children, (3) – denotes kitchen and bath only

1.2 *Greywater Treatment Methods*

There are several treatment methods for improving the quality of greywater, which include (DPI, 1996):

1. Screening which is necessary to prevent the blockage of pumps and irrigation systems
2. Settlement which allows solids to settle out in a tank
3. Filtration systems which include granular or membrane filters to provide both physical and biological treatment.

Methods of greywater treatment are listed in Table 4.11. This includes a comprehensive and up to date compilation of greywater treatment by Li et al (2009).

Table 4.11: Methods of Greywater Treatment

Greywater Treatment Method	Reference	Comments
Sand/Gravel filters	Van der Ryn (1978), Simpson (1983), and Jenssen (2002)	Suitable as a minimal treatment standard – suitable for trench disposal.
Reed/Gravel beds	Simpson (1983), Dallas (2005), Mullegger et al , (2003) and Gross et al (2007)	Simpson studied greywater treatment with reed/gravel beds, as reported in Portfolio 3. Dallas (2005) comprehensively proved the effectiveness of reed/gravel beds for treating greywater in Central America. Gross et al (2007) considers reed//gravel beds as the most environmentally friendly and cost effective technology for greywater treatment and reuse. Mullegger (2003) reports on reed/gravel bed treatment producing greywater suitable for reuse.
Biofilters	Jenssen (2002)	Jenssen (2002) reports that greywater can be treated using biofilters, sand filters and reed/gravel beds or combinations of these.
Rotating Biological Contactors (RBCs)	Mullegger (2003)	RBCs produce a standard of greywater that is suitable for non-potable reuse.
Lime and Magnesium Salts	Mawson (1970) and Simpson (1986)	This process results in a very acceptable degree of treatment but it is not a conventional method. Simpson (1986) followed up work by Mawson (1970) and conducted jar tests and batch pilot plant trials of greywater with acceptable results, as reported in Portfolio 8.
Chemical Coagulation, Ion Exchange and Granulated Activated Carbon	Li et al (2009)	In the past, ion exchange has not necessarily been economic. The writer has used granulated activated carbon as a post treatment unit, following a reed/gravel bed in Cairns.
Physical Processes	Li et al (2009)	Physical processes alone are not sufficient to guarantee reductions in organics, nutrients and surfactants.
Anaerobic Processes	Li et al (2009)	Not suitable for greywater treatment due to poor removal of organic material and surfactants.

Comments on bark/wood chip filters have not been included in Table 4.11 but they are very worthy of some mention. Work was undertaken by Lens et al (1994) with domestic wastewater on test columns containing various fresh and aged organic materials. The results on fresh organic materials tended to be mixed and limiting overall. Results on aged organic materials gave pollutant reductions of suspended solids of 91%, COD of 50%, BOD_5 of 99%, and NH_4-N of 93%.

Denitrification testing of agricultural drainage waters, in wood chip reactors, at variable flows was undertaken by Greenan et al (2009). NO_3-N removal rates varied from 30% to 100% depending on the flow rate. Denitrification was found to be the dominant NO_3-N removal mechanism.

Denitrification beds lined with wood chip and sawdust for treating domestic wastewater, dairy effluent and glass house effluent have been proven to be effective and inexpensive to construct and operate in New Zealand (Schipper et al, 2010). NO_3-N removal rates can be high.

The above described work on bark, wood chip and sawdust filters indicates there is much potential for the use of these materials for treating greywater and domestic wastewater.

For more information on the treatment of domestic wastewater and greywater reference can also be made to Section 2.5.

2 Industry Projects Involving Greywater Treatment by Simpson

2.1 Background

The writer's experience (since 1976) with greywater treatment technology has involved the design of systems undertaken for the purpose of servicing consulting clients who have elected to install composting toilets. He has had a definite interest in this area since this time and advocates greywater pre-treatment, in a sullage tank or grease trap. There is a need for further simple and effective alternatives for treating greywater. The need for treatment has been generated by the more recent approval of greywater for reuse in Queensland.

Some feasible methods for treating greywater are discussed here.

2.2 Pebble Filters

Van der Ryn (1978) reported on the use of 44 gallon (Imperial) drums for use as greywater filters. The drum was positioned vertically and effluent distributed on the top surface, on a splash plate. The top media was coarse sand, on pea gravel or medium gravel, which is laid on coarse gravel in the base.

The advantages of this simple drum filter are:

1. Additional drums can be added if flows increase

2. The drum filters can be covered in the rainy season to avoid saturation
3. A grease trap prior to the filter will prolong the life of the system
4. Two alternating drum filters will be more efficient in the longer term.

The writer designed many of these pebble filters in both the north and south islands of New Zealand, during the period 1977 to 1984. They were based on the design by Van der Ryn (1978). The operation was simple, the filter system was robust and it was readily maintained by removing a layer of blocked media and replacing it. No performance monitoring was undertaken on the basis that the operation was simple and the filtering action produced a solids-free liquid. The writer's contributions to improve the operation were:

1. Paint the inside of each drum with tar, to extend the life of the container
2. Drill a pattern of 6 mm holes in the splash plate to further improve greywater distribution
3. Periodically, at least 50 mm of the top sand was removed and replaced with clean sand, to prevent blocking
4. On occasions he placed a top layer of bark, on pea gravel, on coarse gravel.

Figure 4.4 is a typical section of a Simpson version of a pebble greywater filter.

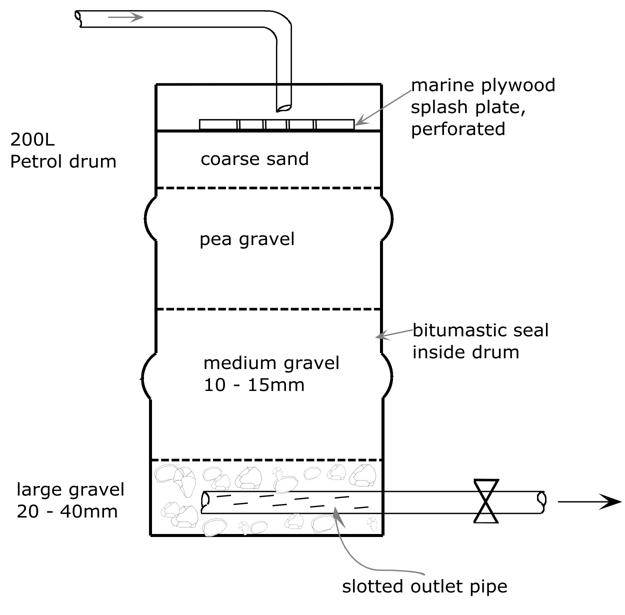

Figure 4.4: Section of Simpson Pebble Greywater Filter

2.3 Peat Filters

Peat has been used for the treatment of domestic wastewater in other parts of the world with great success (Brooks et al, 1984). Since the early 1970s the writer has developed a particular interest in natural treatment including peat based systems.

A full scale and successful research project on peat filters were undertaken in New South Wales (Patterson et al, 2001). The performance results from four peat beds are in Table 4.12.

Table 4.12: Performance Results from Four Peat filters

Parameter	Units	Minimum	Mean	Maximum	% Reduction[1]
pH		5.44	6.17	6.47	19
SAR		2.8	3.4	4.2	49
Ammonia -N	mg/L	0.5	3.16	9.02	95
TKN	mg/L	11.0	15.3	17.9	77
Total N	mg/L	30.8	39.0	57.0	44
Total P	mg/L	0.59	1.09	2.13	84
Faecal coliforms	cfu/100 mL	167	1084	2973	99.5

Note:

1. Calculated from monitoring results from septic tank effluent from five houses

By examining the mean figures, the percentage performance results of particular note in Table 4.12 show that there has been acceptable Total N reduction and high Ammonia-N, Total P and Faecal coliform reductions.

It is to be noted that peat is freely available in New Zealand whereas, in Australia, there are very limited peat resources.

In the 1970s the writer was interested in the use of Minnesota peat and sand filters, for on-site effluent treatment reported by Osborne (1975). Based on the work by others, Osborne reported that small outdoor peat filters achieved high phosphate, BOD and coliform removals. In addition, grass on the filter surface was particularly effective in biological N and P uptake from effluent. Particular features of the Osborne peat/sand filter were:

1. An upper layer of peat
2. An intermediate layer of peat and sand mix
3. A bed of medium sand
4. An effluent collection layer of pea gravel
5. Effluent distribution was by a sprinkler system.

The system operated better in the summer when the warmer temperatures and the grass cover enhanced the treatment processes.

The merits of peat filters, as reported by the University of Minnesota (Gustafson et al, 2001), are as follows:

1. Have a high water holding capacity
2. Have chemical properties that make them very effective for treating effluent
3. Unsterilized peat contains a number of different microorganisms including bacteria, fungi and microflora.

Much of the peat is harvested from the northern regions of the State of Minnesota. It is harvested from large beds and screened for consistency.

Other important aspects of peat filters, as reported by Gustafson et al (2001), include:

1. The effluent must pass through the peat under unsaturated conditions
2. The depth of the peat is to be 600 to 800 mm
3. Because of its high organic content, typically 88%, the filter media must be replaced periodically.
4. The life expectancy is from 10 to 15 years.

In the year 2000 the writer was part of a small team that sought funding for greywater treatment in Queensland using combined peat and sand filters. It was appreciated at the time, based on experience on peat filters in Australia and overseas (Patterson, 1999), that these units had potential for reducing nitrogen, phosphorus, heavy metals and pathogens. The peat product, called *Spill Sorb,* was being marketed in Queensland. The writer witnessed this product's ability to work as a very successful oil and paint absorbent, which indicated that it was likely to have potential for contaminant removal.

The plan was to set up applied research projects, with the possible assistance of *Clivus Multrum (Qld) Pty Ltd,* for treating greywater. The writer prepared a report "Peat Filter Development – Funding Proposal" for approaching the Queensland Department of Natural Resources, to undertake a six month monitoring period and report for ultimate State Government approval of a greywater peat filter. This project unfortunately did not proceed as the *Spill Sorb* marketing person retired, and without his enthusiasm, the company lost interest in this marketing opportunity.

In the writer's experience simple peat filters were used extensively along the west coast of the South Island of New Zealand, as a follow-up unit to domestic septic tanks. Based on reports by New Zealand plumbers, these systems were successful. In New Zealand, it has been the practice for more than 50 years to combine greywater with blackwater for treatment purposes. Since the introduction of AS/NZS 1547:2000, this is now the requirement in both countries. The writer saw the combination of greywater and blackwater as a step forward for improving the treatability of wastewater.

Fletcher (2007) reports on a peat filter system in New Zealand at the Lake Taupo Christian Youth Camp treatment system, with a flow of $70m^3/d$. This includes a 610 m^2 peat filter bed where the target of a 40% reduction in Total N was being achieved. The treatment performance has improved with time, with Total N reduction consistently better than 60%, BOD_5 is 99.8% and the suspended solids at 97% (David Napier, Harrison Grierson Consultants, pers.com. April 2010). Effluent is being irrigated and earlier odour problems were overcome by spreading a 150 mm layer of bark to rectify the problem. According to David Napier this has worked perfectly.

Peat biofilters significantly reduce levels of BOD_5, suspended solids, NH_3-N and faecal coliforms (Schafran, 1999). A study in Virginia during 1997-1998 showed that the performances of 12 systems were not significantly affected by the season of operation.

2.4 Plastic Media Pre-filter

Clivus Multrum (Qld) Pty Ltd developed a plastic media pre-filter for greywater treatment. This follows a grease trap and it consists of cut polythene pipe randomly packed in a polythene tank. Two long nylon socks capture solids which are periodically cleaned out. The filter media operates as an aerobic system. The pre-filter unit also has facility for periodic cleaning out.

It is possible that the *Clivus Multum (Qld) Pty Ltd* cut pipe pre-filter could still discharge finer solids into an effluent system. The writer feels that a secondary filter, consisting of pebbles or coarse sand on a base of larger aggregate would effectively enhance the final effluent quality.

2.5 Bark and Wood Chip Filters

The use of bark and wood chip for wastewater and greywater treatment has been reported on for many years.

In 1982, the writer installed a bark/wood chip filter as a pre-treatment unit for an aeration plant at a small shopping centre in Kumeu, near Auckland, New Zealand. Refer to the filter section in Figure 4.5.

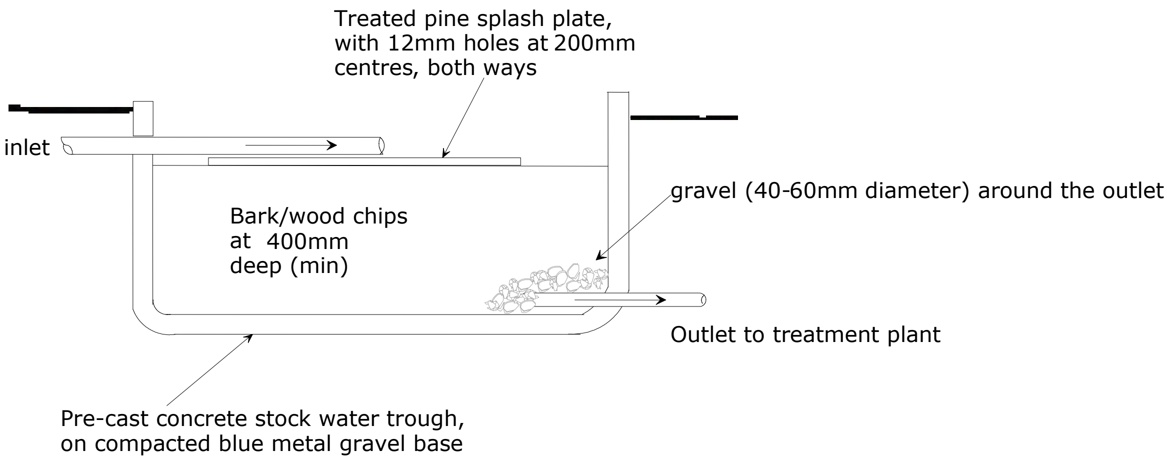

Figure 4.5: Section of Kumeu Bark/Woodchip Pre-Treatment Filter

The writer has discussed the role of the bark/wood chip filters as he considers that this system would also function well for treating greywater. In the case of the Kumeu project there had been problems with the reliable performance of the extended aeration plant, due to a butchery, bakery and a hairdresser discharging greasy and partially toxic wastewater. The wood chip/bark filter rectified the pre-treatment problems. The writer reported this to the Institute of Water Pollution Control (UK), along with other experimental projects, for corporate membership (Simpson, 1983).

No detailed monitoring was undertaken but the fact that the extended aeration plant was able to function in a steady state was proof that the filter had performed its pre-treatment tasks of reducing toxicity and grease.

Further to the discussion on bark/woodchip filters in Section 2.1, the use of aged barks (5-8 years old) and raw bark were evaluated as absorbents for shock loads reported by Lightsey (undated). Three sizes of each bark were tested. The aged bark was considerably more effective than the raw bark in both the rate of absorption and amount of absorption of shock loads. A possible explanation for aged bark's relative success is that fungi and possibly actinomycetes are the organisms which breakdown wood chip to make it more attractive for other organisms such as bacteria (Dr Rabi Misra, University of Southern Queensland, pers. comm. November 2009). This is confirmed by Dr Bernadette McCabe (University of Southern Queensland, pers. comm. June 2010) who advised that it is the lignin, cellulose and tannins in wood chips that make it more difficult to breakdown. Soil fungi are responsible for much of the initial breakdown in wood which then allows the denitrifiers to carry out the process of de-nitrification.

Douglas Fir bark was tested in trickling filters 300 mm, 600 mm and 1.0 m in depth with variable loads of poultry manure slurry at the Oregon State University (Cropsey and Weswig, 1973). Aspects of particular interest were as follows:

1. As the depth of the filter and rate through the filter increased, the removal of total solids increased
2. The BOD and suspended solids concentrations lowered at a faster rate when the larger bark was used.

OSCA (1979) covers the use of bark filters, in conjunction with sand, for treating septic tank effluent. The sand and wood chip filter range from 3.0 to 6.0 m^2. The effluent is distributed into a layer of 20 mm crushed stone 200 mm deep, underlain with 600 mm of sand, then on 80 mm of pea gravel, and then on 200 mm of crushed stone with a collection pipe. The wood chip is used as a replacement or option to the sand.

No specific performance data is given, other than the statement that the filter "dramatically improves the quality of the effluent being discharged to the leaching bed. The addition of an effluent filter to all systems is strongly recommended."

Nitrate concentrations can cause problems in groundwater. Robertson et al (2005) report on a maximum of five years of monitoring of on-site filter systems for enhancing nitrogen removal. In each case the septic tank effluent was pre-treated in a sand filter then passed through a wood chip filter. NO_3-N reductions ranged from 87 to 98%. These filters offer a simple and practical solution to nitrate control in small to medium sized treatment systems.

Some work has been done on the denitrification of wastewater within agricultural tile drains and wood chips, as the final stage of wastewater treatment in New Zealand. The wood chips show promise as a means of slow release of a carbon source and as a media for denitrification. Based on six years of data the process shows promise in terms of hydraulic conductivity and denitrification potential (Dr Chris Tanner, pers. com. NIWA, NZ). More specific performance information cannot be supplied because of commercial confidentiality issues on this industry based project.

The writer has taken an interest in a company called *Nature Loo*, which has a factory in Narangba, Queensland. It manufactures composting toilets and a greywater filter called *Nature Clear GWS10*. This compact filter (about 1.0 m^3) consists of pine bark media on top of sand and gravel. As of 2005, the *Nature Clear GWS10* unit has been approved only in Queensland and only as an advanced primary treatment standard. Limited effluent monitoring from this filter has been undertaken over 2003-2004 by Brisbane City Council and the former Noosa Council. Refer to Table 4.13 for the summary results.

Table 4.13: Monitoring Results of Nature Clear GWS10 Greywater Filter

Parameters		
Monitoring Date	**BOD_5 (mg/L)**	**Suspended Solids (mg/L)**
2/12/02	12	28
16/04/03	28	41
9/09/03	7	5
17/09/03	37	15
8/01/04	14	12
Average	23.7	30
Maximum acceptable for secondary effluent	20	30

A comment on the results presented in Table 4.13 is that the average BOD_5 and suspended solids are 23.7 and 30 mg/L respectively, and these are very close to the accepted Queensland secondary treatment standards of 20 mg/L BOD_5 and 30 mg/L suspended solids.

These results indicate that the *Nature Clear GWS10* filter unit has much promise as it is capable of achieving a secondary standard of effluent. The writer has agreed to undertake experimental work and performance monitoring for *Nature Loo* on a new wood chip filter which has the potential to achieve advanced secondary treatment standards, including Total N reduction.

3 Reflections on Greywater Treatment

3.1 Pre-Filter Units

The writer has inspected several of the *Clivus Multrum* pre-filter units in South East Queensland and they would appear to effectively act as a roughing filter. The units are capturing larger solids which can readily be cleaned out.

On reflection, there appears to be a need for a bark, coarse sand or pebble filter, prior to discharge, to capture finer solids. This is to prevent a build up of fine solids within the greywater trench system and its eventual failure.

3.2 Bark and Wood Chip Filters

Bark and wood chip filters have several merits as a treatment unit for both wastewater and greywater, for the following reasons:

1. Useful source of carbon
2. Acceptable Nitrogen reduction (Joe Whitehead, University of Newcastle, pers.com, 2009)
3. Enable access of oxygen.
4. The media is economic to purchase
5. The top layer of bark or wood chips is easily replaced.

The *Nature Clear GWS10* filter has the potential to achieve a high secondary or tertiary standard of effluent by increasing the detention time, by enlarging the surface area and volume, or by installing a second filter unit. The writer plans to be involved in further trials with greywater to confirm the effective BOD_5 and suspended solids reductions and the nutrient reduction capability.

The writer is familiar with the water pollution constraints in the Lake Rotorua and Lake Taupo areas in New Zealand. The areas are characterised by free draining pumice soils in which nitrogen and phosphorus pose a threat to the enrichment of groundwater and lake waters.

The writer has found it to be worthwhile following up and reflecting on the successful use of bark filters in New Zealand. Fletcher (2007) reports on two bark filter systems within the Taupo District Council in New Zealand:

1. Kinlock wastewater treatment plant – effluent is polished successfully by a bark filter; the average total nitrogen has been 3.9 mg/L. (The target is a 40% reduction in Total N)
2. Motutere settlement treatment system is by submerged air flotation followed by a bark filter bed. Again, the target is a 40% reduction in Total N is being achieved.

In the writer's opinion, this work by the Taupo District Council confirms the potential for using simple bark filters for treating wastewater in Australasia.

3.3 Greywater Pebble Filters

With respect to the pebble greywater filters that the writer has designed, on reflection he would have installed the following:

1. A vertical 90 mm diameter slotted PVC air vent, in each unit, to enhance the aerobic biological activity deeper in the filters
2. Pre-treatment for the greywater in a 250 L or commercial sized grease trap, to allow the greywater to cool, and to capture hair, lint and other solids
3. A top layer of untreated bark to further enhance the passage of oxygen into the body of the filter.

3.4 Peat Filters

The writer supports the use of peat filters for wastewater and greywater treatment in Australia and New Zealand, as developed by Dr R Patterson, Armidale, and by the University of Minnesota, for the following reasons:

1. Very good peat filter performance results, in particular Ammonia N, Total P, Faecal coliforms and the acceptable Total N reduction (Patterson et al 1986; Patterson, 1986)
2. High water holding capacity
3. Suitable chemical and physical properties
4. Presence of useful microorganisms
5. Life expectancy of 10 to 15 years (Gustafson et al, 2001).

On reflection, the writer concurs with the researchers that the peat filters must function under unsaturated flow conditions, to maintain ideal effluent to peat contact and to maintain an aerobic environment. There may, however, be changes in the performance of peat filters within the wide range of climatic conditions, in both countries, as experienced by the impact of the seasons on the Minnesota peat/sand filter (Osborne, 1975).

3.5 Greywater Management Practices and Reuse

Thinking back, in the writer's opinion the introduction of greywater treatment and management practices in the recently introduced Queensland code (QPWC, 2008) was a positive move. The Queensland drought helped to provide the impetus for this publication. Basically, if greywater is to be disposed by sub-surface means then primary treatment is needed. This ensures that there is no direct human exposure to harmful bacteria present in the greywater. It is important that solids, hair and lint are removed to avoid blocking of the trench soil interface.

If the greywater is to be spray irrigated the greywater must be treated to a secondary standard. Secondary treatment ensures that organic matter is broken down and harmful micro-organisms are removed. Disinfection, by chlorination or UV treatment, may be required to eliminate public health risks.

4 Greywater Treatment Conclusions

Greywater Quality

The pollutant potential of greywater had not been considered a real problem for many years. This was mainly due to the absence of faecal material therefore, health risks were considerably reduced. This is correct to a degree, but the work of Jefferson and Solley (1994) showed that there was a need for more than settling and stone filtration, or primary treatment.

Greywater Tanks and Treatment

It has been the writer's experience that it is important that pre-settlement is undertaken, in at least a 1,600 L capacity tank or a commercial sized grease trap, to cool down greasy liquid and capture lint, hair and other solids. A failure to do so will eventually cause blockage problems in the disposal system and the need for replacement of the trenching.

The use of a tank, followed by a pebble filter of the type the writer has used, is the minimal degree of greywater treatment since it removes lint, hair, grease, and food particles.

Peat filters, operating under unsaturated conditions, have been well proven in Australia as an effective greywater treatment method. Peat and sand/wood chip/bark filters are suitable for removing organics, nutrients and some bacteria. Aged wood chip have the capability to nitrify and denitrify greywater which the writer sees as a distinct merit.

Secondary Treatment Potential

Sand and woodchip/bark filters are capable of achieving a secondary degree of treatment. In the case of the *Nature Clear GWS10* units, by enlarging the filters and increasing the retention times, it is possible that an advanced secondary treatment can be achieved. This is worthy of further research and development, which the writer hopes to undertake in 2012.

Reed/Gravel Beds

Based on the work undertaken by Marshall (1995), the writer's own research and in conjunction with others (Simpson, 1995; Borthwick, 1995) and the work in Central America by Dallas (2005), reed/gravel beds have been demonstrated to be a simple and practical alternative for greywater treatment, to a standard suitable for reuse. Greywater can be treated to a standard suitable for non-potable reuse.

Greywater Reuse

Greywater treatment has emerged as an important need in Queensland, particularly with the introduction of greywater reuse guidelines, the QPW Code (2008). It is to be treated to a secondary standard if reused by surface application. The thermotolerant coliform count must not exceed 200 organisms per 100mL, with no sample exceeding 1,000 organisms per 100 mL.

Greywater use in Australia has recently been reviewed by Malawaraarachchi et al (2011). There are differences in both detail and content of greywater regulations in each state and territory of Australia. There is a definite need for the development of a standard national policy on greywater reuse as this may simplify the implementation of greywater reuse.

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Portfolio 5 – Effluent Disposal Survey and Innovative Treatment and Disposal Systems

This Portfolio covers an Australian-wide survey of on-site effluent disposal systems undertaken by the writer Part (A). A selection of innovative wastewater treatment and effluent disposal systems are covered in Part (B). The effluent disposal systems in Part (B) have been sourced from New Zealand and Australian experiences.

Part A

Part B

Part (A) Australia-Wide Mail Survey of On-site Effluent Disposal Systems

1 Introduction

This Portfolio reports on a mail survey of Australian effluent disposal systems and innovative wastewater treatment and effluent disposal systems.

The writer was interested to know more about the types of effluent disposal systems used throughout Australia. The writer undertook a postal survey of Australian effluent systems and presented a paper at the *ON-SITE* '*01* conference in Armidale, NSW in 2001. This on-site wastewater treatment and effluent disposal conference was normally attended by researchers, engineers, scientists and inspectors from Australia and New Zealand, with some keynote speakers from overseas.

The issue as to whether the system is appropriate to the site and climatic conditions is often questioned. The assessment of existing on-site effluent disposal systems is an on-going need so their performance and suitability can be evaluated.

Comprehensive studies of alternative effluent disposal systems, at an on-site scale, have not been widely undertaken within Australia. Traditionally this type of work has been undertaken in North America (Bouma 1974, Otis 1976, Jones 1977, and Ward et al 1983).

Due to past and pending planning tribunal and court cases, it was appreciated that the information sought on effluent disposal systems may be politically and environmentally sensitive for some councils and authorities. For this reason the systems covered in this mail survey have not been directly identified with the local authority or organisation that partook in the survey.

Publications by Simpson Appropriate to Portfolio 5

1. Simpson, J S 1974, Wastewater Disposal in Smaller Coastal Communities, Dip PHE dissertation, School of Engineering, University of Auckland

2. Simpson, J S 1986, The Evaluation of Sites and Alternatives for On-site Effluent Disposal, *New Zealand Journal of Environmental Health*

3. Simpson, J S 2001, Results of an Australia Wide Mail Survey of Existing On-site Disposal Systems, *Onsite'01 Conference Advancing On-site Wastewater Systems*, University of New England, Armidale, NSW, pp. 339-346, 25-27 September

2 Survey Approach

2.1 Objectives and Methodology

In order that the postal survey followed a system, a number of objectives were established:

1. To undertake a mail-type survey of domestic on-site effluent disposal systems commonly used throughout Australia. This included a limited number of trial systems
2. To understand the performance of effluent disposal systems
3. To note the merits and problems encountered
4. To note management needs
5. To assess the site suitability of each system type
6. To identify potential improvements to existing systems or optional disposal systems based on the outcome of the survey.

The methodology used was to contact various universities, key local authorities, State Government Departments and water authorities and send them a detailed questionnaire regarding the disposal systems known to be used in their region.

These various contact organisations then invited councils and water authorities involved with on-site treatment and disposal systems to fill out the questionnaire and return it to the writer.

The questionnaire included the following aspects:

1. Type of system
2. Approximate age
3. Pre-treatment and/or polishing system used
4. Degrees of performance rating (1 being the lowest to 5 being the highest)
5. Soil types prevailing
6. Suggested modifications to improve performance
7. Specific site and /or climatic suitability
8. Typical flows
9. System areas, depths and widths
10. Aggregates and other materials used
11. Management needs
12. Potentially feasible options.

2.2 Survey Returns

The survey response was estimated to be about 60%, but this varied between States. The following councils and authorities returned questionnaires:

1. Huon Valley Council, Tasmania
2. Sorell Council, Tasmania
3. Launceston City Council, Tasmania
4. Brighton Council, Tasmania
5. City of Mandurah, Western Australia
6. City of Gosnells, Western Australia
7. City of Blue Mountains, New South Wales
8. Wingecarribee Shire Council, New South Wales
9. Port Stephens Council, New South Wales
10. Redland Shire Council, Queensland
11. Maroochy Shire Council, Queensland
12. Kingston District Council South Australia
13. Mitchell Shire Council, Victoria
14. Department of Human Services, South Australia.

2.3 Mail Survey Limitations

The mail survey of Australian effluent disposal systems was targeted to a limit of at least five invitations from each State, due to the restricted time available to send out returns, analyse them and compile the paper (Simpson, 2001). The detail requested was also limited due to the restricted time schedule.

The assessments and ratings of effluent disposal systems were generally largely subjective. In some cases the information could have been politically and/or environmentally sensitive, for example, in areas that had already experienced publicity on sub-standard surface and groundwater quality. The survey responses tended to be undertaken by various individuals within each council or authority, rather than by an individual undertaking the entire survey and ratings.

It was appreciated that staff resources are often limited, therefore the questionnaire had to be reasonably concise. The detail and quality of the information provided varied considerably, which made it difficult to assimilate.

3 Survey Outcomes

3.1 Survey Responses

The types of systems reported and rated included the following broad categories:

1. Shallow infiltration trenches and beds
2. Brick pits
3. Leach drains
4. Deep trenches
5. Alternate dosing trenches and beds
6. Sand filters
7. Mounds
8. Surface irrigation
9. Mulch and garden beds.

A significant amount of the information supplied in the survey has been presented in a summary form in Tables 5.1 and 5.2. Some of the detail has not been reported as it would have been difficult to do so without particular reference to the Council. Table 5.1 covers the system age, pre-treatment, effluent standard, permeability, performance rating, soil type and supporting comments. Table 5.2 focuses on management needs and suggested modifications to improve the system performance and its sustainability.

Some of the design and operational problems, environmental concerns, suggested modifications and management needs have been further described later in this Portfolio.

Table 5.1: Summary of Effluent System Characteristics and Ratings

Categories	Age (yrs)	Pre-treatment type	Effluent standard	Soil type	K (m/d)	Site suitability	Rating	Comments
Absorption trench		Filtered septic tank	Primary	Clay loam	0.5-1.5	Suited to more sandy soils	3	
Absorption trench	50+	Septic tank	Primary	All types	-	Not suited to shallow low permeability soils	1	55% failure rate, being phased out
Absorption trench	25	Dual septic tanks	Primary	Light clays	0.06-2	High-medium permeability	3	< 8° slope
Absorption/ET [2]	10-15	Septic tank	Primary	Sandy loam	>3.0		4	
Alternating beds	0.3	Various	Primary	Clay soils	<0.01	Not suited to steep slopes	2	Management of alternating system required
Brick pit	25	Septic tank	Primary	Duplex soil	high	Only sandy soils with deep ground water	3	Rarely used now, often used in confined spaces
Deep trenches	3	Septic tank	Primary	Fractured mud, silt stone	0.05-1	Exposed areas	5	Suitable for 5-8° slope
ECOMAX [3]	1-2	Septic tank	Primary	All types	broad range	Suitable for sloped and flat sites	4	
ET bed + surface irrigation	10	Septic tank	Primary	Clay loam - light clay	0.06-1.5	Lower permeability soils	4	For greywater only

Infiltration bed	10	Septic tank	Primary	All types	0.01-0.1		3	Not suited to steep slopes
Infiltration bed/trench	100+	Septic tank	Primary	Sandy soils	High	Suited to sites with permeable soils	3	Keep clear of flood prone areas
Infiltration trench - alternating	0.7	Various	Primary	Sandy/silty clay or better	0.01-0.5		2	Management of alternating system required
Leach drain - nutrient retention	10	Septic tank	Primary	Sand	High	Suited to high water table	4	Suitable for sensitive areas; amended soil required
Leach drains	60+	Septic tank	Primary	Sand	High	Suited to sandy soils		>1.2 m above ground water
Modified leach drains		Dual septic tanks and HWTP[3]	Secondary	Sandy, heavy	Broad range		4	<5% failure rates
Overland flow	10+	Greywater tank	Primary	Sandy-clay loam	0.06-0.6		4	For greywater only, occasional problems
Pressure irrigation - mulch covered	4+	HWTP [3]	Secondary	Sandy-clay loam	0.06-0.6			Performance to be assessed
Recirculating sand filter	2.5	Septic tank	Secondary	All types	Broad range	Exposed areas, silty sandy clays	5	Suitable for 5-8° slope
Sand mounds	5	Septic tank	Primary	All types	0.01-5	Suited to low permeability soils, high water tables	4	Generally not used on sloping sites >8°
Shallow infiltration trench	50+	Septic tank	Primary	Loam and sandy	Broad range	Broad, lower ground water + rock	4	<10° slope

Shallow infiltration trench		Septic tank	Primary	Loam - rocky	0.06-2		3	Trenches installed along contours
Shallow infiltration trench	5	Septic tank	Primary	Duplex soil	0.01-1.0	Good in duplex soils to maximise upper horizon	4	Not suited to very steep slopes; most soils
Shallow infiltration trench	3	HWTP [3]	Secondary	Clay loam	0.5-1.5		2	Can have root blockage problems
Shallow subsurface irrigation	2	Septic tank	Primary	Silty clay	<0.06	Exposed areas all soil types	4	5-10° slope
Shallow subsurface irrigation	3	Septic tank	Primary	Duplex soil	0.1-1.0	Flat sites, terracing required on slopes	4	For greywater only
Slow sand intermittent + wetland	1.5	Septic tank	Secondary	Silty clay	>0.06		5	Dispersive clay
Subsurface irrigation	0.5	Filtered septic tank	Secondary	All types	Broad range		5	System being trialled, 5-8° slope
Surface + subsurface irrigation	10	HWTP [3]	Secondary	All types	0.06-2.8	Suitable for smaller blocks	5	Preferred subsurface system
Surface irrigation	10+	HWTP [3]	Secondary	Sandy loam/ light clays	0.06-3.0	Varied	2	Steep contours are a constraint
Surface irrigation	12	HWTP [3]	Secondary	Loamy - heavy clay	Broad range	Broad	4	<10° slope
Surface irrigation	10	HWTP [3]	Secondary	Sand	High	High permeability soils	5	Soil imported at times

Surface irrigation		HWTP [3]	Secondary	All types	Broad range	Cultivated topsoil over irrigation area	3	Not suited to steep slopes
Surface irrigation/raised garden bed	3	HWTP [3]	Secondary	Loam	1.5-3	Suited to a range of sites	5	No failures reported
Surface irrigation/raised garden bed	4+	HWTP [3]	Secondary	Sandy-clay loam	0.06-0.6			Performance to be assessed

Notes:

1. Evapotranspiration beds/trenches
2. ECOMAX raised bed system
3. Home Wastewater Treatment Plant
4. Performance rating based on 1 to 5, 1 being poor and 5 being excellent

Table 5.2: Summary of Effluent System Management Needs and Suggested Modifications

CATEGORIES	MANAGEMENT NEEDS	SUGGESTED MODIFICATIONS
Absorption trench	Trenches rested if possible. Use shallow trenches to promote ET. More regular pump out of septic tanks. Replacing trenches. Use dual trenches and alternating.	Install filters in septic tanks to reduce solids.
Absorption/ET		Try pumped distribution to improve life and performance.
Alternating systems - general	Public education needed.	

Brick pit	Rarely installed now. Check not polluting groundwater.	Possible application in confined sites
Deeper trenches	Similar to shallower trenches and beds.	3 year trial being undertaken. Could be more effective in areas with shallow soil pans.
ECOMAX	Not specifically mentioned in survey returns.	Based on the survey feedback it could be worth trying in other areas.
ET bed + surface irrigation	More public education needed. Ongoing assessments.	Use of septic tank filters.
Infiltration bed	As mentioned in this table.	Larger bed areas required than previously specified. Try smaller distribution pipes by using tank filter to avoid blockages.
Infiltration bed/trench	More regular pump outs.	Use larger capacity septic tank with filters.
Infiltration trench - alternating	Use larger areas, in line with new joint standard.	Incorporate inspection openings. Deep ripping of disposal surfaces during construction. Gypsum treatment of soils.
Leach drain - nutrient retention	Not mentioned in survey returns.	Importing nutrient (P) retentive sandy soils. Use amended soils.
Leach drains	Ensure at least 1.2m above groundwater.	Use of dual drains and alternating. Use precast concrete and PVC drains with geotextile.
Modified leach drains	Not reported.	
Overland flow	Been operating 4 years in good soils. Owner education required. Site monitoring required.	
Pressure irrigation - mulch covered	Owner education needed.	Most suited to flatter slopes. Avoid long beds due to pressure distribution.

Recirculating sand filter	Owner education needed.	
Sand mounds	Cleaning of filter. Ensuring tanks pump out.	Use effluent filter in tanks. Use of smaller diameter pipes.
Shallow infiltration trench	More frequent desludging of septic tanks. Avoid tree root problems.	Distribution box beneficial for longer trench requirements. More frequent rotation of trenches.
Shallow subsurface (greywater)	Improved overall performance.	Flush out distribution lines annually (3 year trial). Use greywater tank filter.
Slow sand intermittent + wetland	Desludge pump pit annually. Check system 6 monthly.	Nil.
Subsurface irrigation - general	As for absorption trench	Using pipework designed for effluent disposal.
Surface irrigation	Better distribution of effluent. More care of irrigation field. Need for signage. Need to replace faulty sprinkler heads. Working well in some areas.	Use cultivated soil over irrigation area.
Surface irrigation/raised garden bed	Replacing diseased or dead species.	

The high ratings of 5 (excellent) are to be noted in Table 5.1 as being very viable alternatives. These include the following technology alternatives:

1. Deep trenches
2. Recirculating sand filters
3. Raised garden beds
4. Slow sand intermittent filters + wetland
5. Spray irrigation of HWTP effluent on soils with a K value of > 0.7 m/d

Ratings of 4 (very good) include the following:

1. Absorption/ET trenches
2. *ECOMAX* systems
3. Overland flow
4. ET beds + surface irrigation (greywater only)
5. Sand mounds
6. Shallow subsurface irrigation (greywater only)
7. Surface irrigation

There are an acceptable number and range of effluent disposal system on offer. The key effluent management problem in Table 5.2 is more owner education, which in the experience of the writer includes the need for a better understanding of how the system functions.

3.2 Environmental Aspects

Typical environmental concerns raised by councils included the following:

1. Effluent systems can be sources of groundwater contamination
2. The need to consider higher degrees of treatment, that is, aerobic type, with higher reductions in organic matter, solids, nutrients and pathogens
3. Known environmentally sensitive areas, requiring careful consideration by using higher degrees of treatment with more sustainable disposal systems.

It is interesting to note that the survey did not contain concerns about odours and the risk of direct human contact with wastewater or effluent.

3.3 Problems Highlighted

Typical design, operational and maintenance problems raised by the Councils included the following:

1. The need to desludge pump chambers

2. Solids carry-over from pump chambers and septic tanks
3. Lack of maintenance of surface irrigation systems
4. The need to regularly move irrigation sprinklers to prevent ponding
5. The need for more reliable pump and control systems
6. Lack of signage for surface irrigation systems
7. The need for public and owner education
8. The need to conserve water hence, requiring a smaller effluent disposal system.

3.4 Optional and Innovative Effluent Systems

Most on-site wastewater treatment and effluent disposal practitioners agree that there is a need for the application of innovation by developing alternative effluent disposal systems, rather than to continue to rely on the use of conventional trenches, beds and irrigation fields.

There is also potential for modifying existing effluent disposal systems, to enhance their performance or make them function in a more sustainable manner. This was the writer's experience in the mid to late 1970s.

The councils surveyed offered the following suggestions for improving existing systems or developing new concepts:

1. The intermittent dosed sand filter, receiving septic tank effluent, followed by a sub-surface wetland (reed/gravel bed) is producing higher performance than expected results in terms of BOD, suspended solids and coliform reduction
2. Pressure distributed surface irrigation beds, covered in mulch or bark
3. Pump applied effluent beds and trenches, facilitating the dosing when pumping and the resting or recovery between pumping sequences
4. Deep ripping and applying gypsum, at the time of constructing trenches and beds in problem soils.
5. Using shallower trenches and beds designed to enhance disposal by evaporation and transpiration and suitable media for enhancing capillary rise, as discussed in Portfolio 6
6. The use of filters in septic tanks to improve suspended solids reduction and minimise solids carry-over, as reported in this Portfolio
7. The use of designed sand mounds, for example the *ECOMAX* concept, for areas with site constraints

8. The importing of amended soils to enhance the performance of effluent systems
9. The use of composting plants with satisfactory greywater treatment and disposal systems
10. Precast concrete and PVC leach drains, using geotextiles, particularly in confined sites with lower groundwater levels
11. Improved designs for brick pits or porous concrete riser pipes, used in confined sites with lower groundwater levels
12. The use of effluent filters in septic tanks so smaller diameter pipes can be used to improve distribution.

4 Postal Survey Conclusions

Given the large coverage of the Australian continent and the wide range of geographic, demographic and climatic conditions, this mail survey has revealed a broad range of effluent disposal systems. It is apparent that in most instances systems have been or are being developed to suit particular site conditions, for example, using evapotranspiration/infiltration systems in suitable climatic areas, with less permeable soils.

It is possible that some effluent systems suit particular climates and they could be used in other States of Australia or in New Zealand, with similar climatic conditions. This is considered to be outcome of the survey.

It was expected that AS/NZS 1547:2000 would greatly assist in the site assessment, soil assessment, design and maintenance of new wastewater treatment and effluent disposal systems. In the writer's opinion this certainly was a step in the right direction but some sections needed to be reviewed and the document made more readable. This joint standard is currently under review.

The mail survey has identified some technical and management problems. It is evident that some improvements and modifications could be made to existing effluent systems. The survey has also identified some potential effluent disposal options for some councils and authorities to investigate. This is another key outcome of the survey.

In summary, the objectives of the Australia-wide mail survey of on-site effluent disposal systems have been achieved.

Part (B) Innovative Wastewater Treatment and Effluent Disposal Systems

1 Introduction, Aim and Objectives

There is an on-going demand for the development of more innovative systems for domestic wastewater treatment and effluent disposal. The development of new strategies and technologies for on-site wastewater treatment will revolutionise on-site management (Tchobanoglous and Leverenz, 2008).

The aim of this Part is to report on innovative wastewater treatment and effluent disposal systems which are not described as being conventional. The objectives to achieve the aim are to:

1. Review the technical literature
2. Report on systems either developed or encountered by the writer
3. Reflect on system changes that could enhance treatment and disposal.

2 Upflow Pebble Filters

2.1 Submerged Anaerobic Upflow Filter

In an anaerobic upflow pebble filter, the septic effluent enters at the base, flows upward through a layer of coarse aggregate about 500 mm deep and is discharged into a weir. Anaerobic bacteria grow on the surface of the aggregate and oxidise the effluent as it passes. The filter can be incorporated into the third chamber of the septic tank. Refer to Figure 5.1 for a typical section of an anaerobic upflow pebble filter, sourced from Raman and Chakladar (1972).

The writer first came across the concept of a submerged anaerobic upflow pebble filter, for treating septic effluent, which was being extensively used in India during university post-graduate studies in Auckland (Simpson, 1974). The reported advantages where as follows:

1. Capable of achieving acceptable BOD_5 and suspended solids reductions
2. Filters can operate for up to 18 months continuously without cleaning
3. Washing the sludge from the filter is simple
4. Filter treatment efficiency is not affected by intermittent inflows.

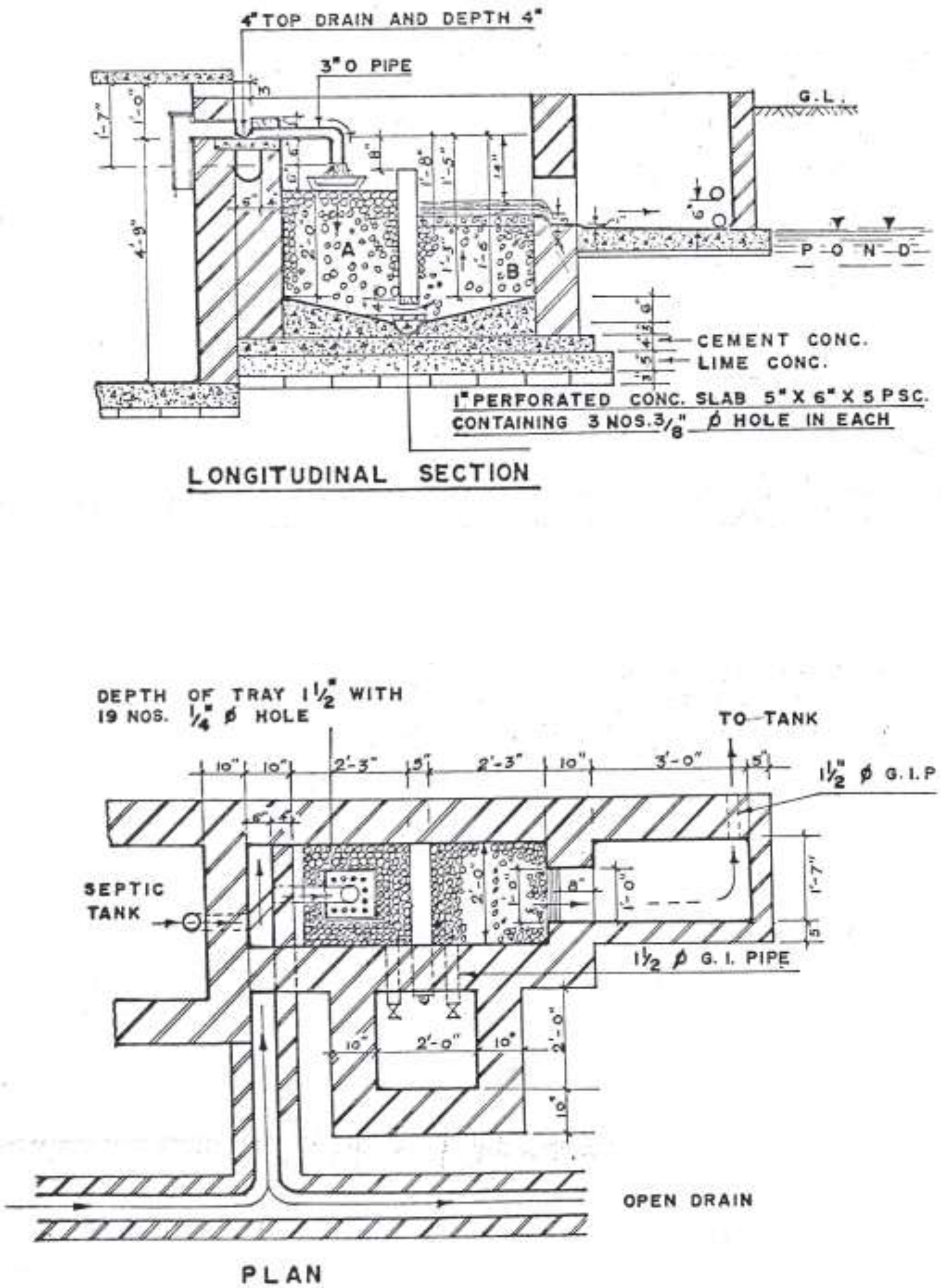


Figure 5.1: Typical Section of Upflow Pebble Filter (NTS)

The performances of upflow pebble filters in Mullickpur, Jalaghata and Apurbapur in India (Raman and Chakladar, 1972) are summarised in Table 5.3.

Table 5.3: Submerged Upflow Pebble Filter Percentage Removals - India

Location - India	**Effluent BOD_5 (mg/L)**	**Effluent COD (mg/L)**	**Effluent Suspended Solids (mg/L)**
Mullickpur	Mean removal 73.3 %	Mean removal 53 %	Mean removal 64 %
Jalaghata	Mean removal 71.4 %	Mean removal 60 %	Mean removal 64.5 %
Apurbapur	Mean removal 74.5 %	Mean removal 57 %	Mean removal 50.1 %

Comments on Table 5.3 are:

1. BOD_5 removals are high, given the limited retention time
2. Reasonably similar % removals of BOD_5, suspended solids and COD for the three locations. The BOD_5 removals of 71.4 to 74.5 % are very acceptable for a short retention time.

The very acceptable BOD_5 removal indicated that in addition to the physical actions of straining and settling of suspended solids, a biological process was occurring within the submerged filters, which resulted in adsorption and flocculation. This dual action was of particular interest to the writer at the time. In the early 1980s, the writer designed an upflow pebble filter for an Apostolic Church camp in South Auckland, New Zealand. It is understood that a New Zealand Standard submerged upflow pebble could have been developed, partially based on the South Auckland system.

The writer, based on observations and discussions with others, concurs with the researchers Raman and Chakladar (1972) that the submerged upflow pebble filters have the following merits:

1. They produce a relatively clear and odourless final effluent
2. They are capable of very acceptable BOD_5 reduction
3. They provide a simple and robust treatment system, suitable for individual homes or small institutions such as schools and camps
4. Annual cleaning will keep the filter in good operational order
5. Upflow filter performance is not affected by intermittent loadings
6. The upflow pebble filters operate successfully at hydraulic loading rates similar to low rate trickling filters.

A research proposal “Anaerobic upflow gravel bed filter for septic tanks” was proposed by the sponsoring authority Engineering and Water Supply Department (E&WS) and the research agency University Of South Australia in 1992. This proposal recognised the work reported in by Raman and Chakladar (1972). Research work undertaken by the E&WS during the period 1990-1992 resulted in a near 100% reduction in N and a 60% reduction in BOD_5 (UniSA, 1992). It is possible that the high Total N removal is due to the presence of aerobic and anoxic conditions for the conversion of ammonia to nitrate and nitrite to nitrogen gas respectively (Dr Vasantha Aravinthan, Senior Lecturer, University of Southern Queensland, pers. comm. 2012).

The research project was to focus on the following:

1. Phosphate reduction with gravel amended by iron oxides
2. Development of design criteria
3. Assessment of filter efficiency
4. Monitoring of bacterial reductions
5. Additional effluent disinfection
6. Development of guidelines for filter maintenance.

A recent follow up by the writer on whether this project went ahead yielded no response.

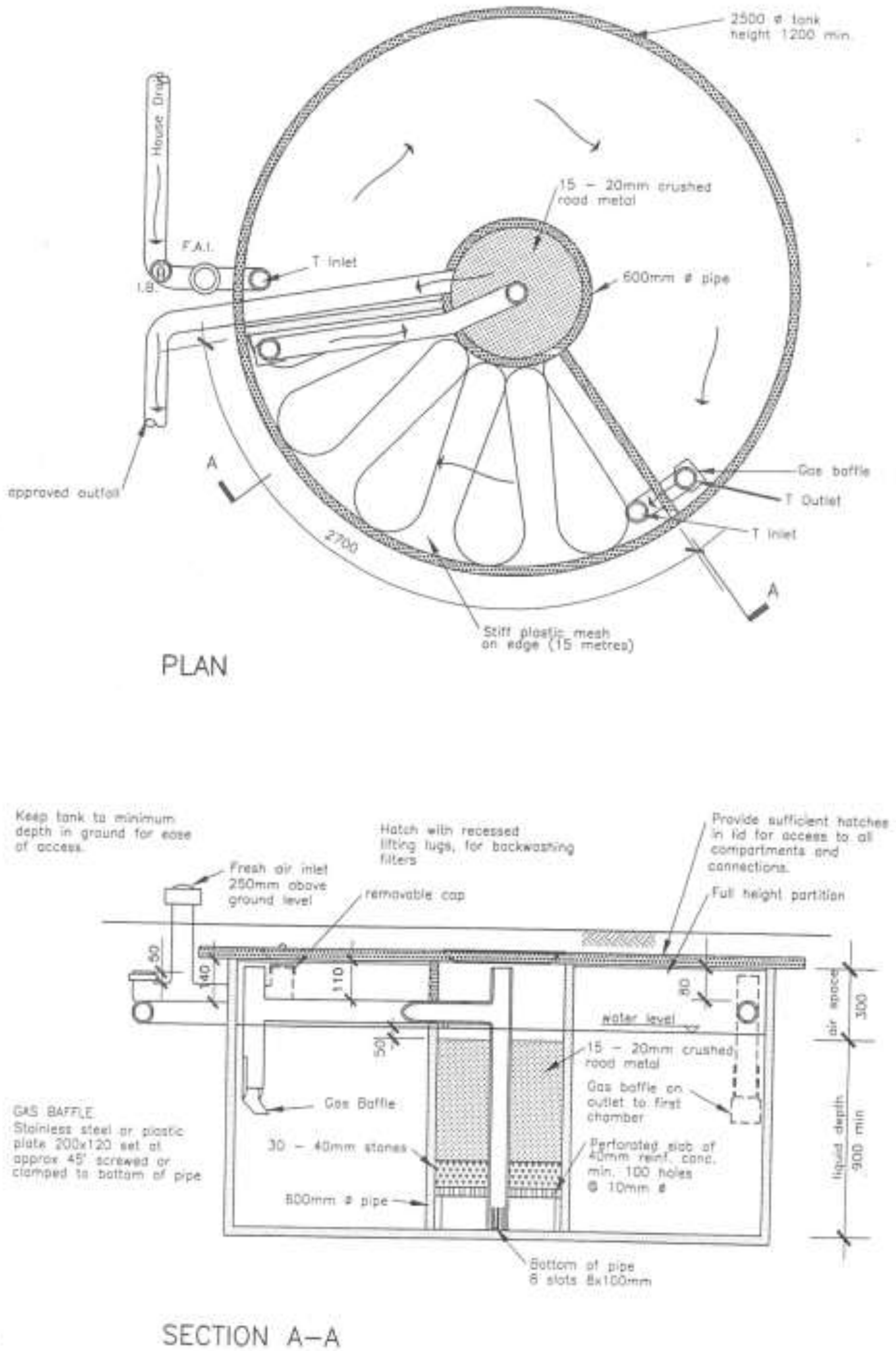
A septic tank/upflow pebble filter/alternating sand filter system was evaluated at the National University of Athens from 1979 and reported by Christoulas and Andreadakis (1989). The upflow pebble filter design was based largely on Raman and Chakladar (1972). The system produced an effluent characterised by BOD_5 of 20 mg/L and suspended solids of 15 mg/L. This effluent quality compared with the effluents produced by biological package treatment plants in the area.

A full scale study was conducted by the University of Science Malaysia to determine the treatment performance of a septic tank followed by an anaerobic upflow filter (Abllah and Lee, 1991). The filter contained synthetic porous plastic media.

The removal efficiencies of BOD_5 and suspended solids averaged 85.2% and 79.7% respectively. The removal efficiencies were found to depend on the value of the sludge age maintained in the filter. The correlation of the influent loading to the effluent quality, in terms of BOD_5 and suspended solids, indicated that better performance can be obtained at higher loading.

Others aspects noted included the following:

1. The pH was maintained in the range of 5.85 to 6.5 which is the optimum range for anaerobic treatment, in the opinion of Abllah and Lee (1991)
2. Upon reaching a steady state, both the BOD_5 and suspended solids reductions were very much enhanced, believed to be attributed to the increase in the thickness of the biomass on the filter media.



Reference: Development Services Dunedin City Council.

Figure 5.2: Section and Plan of ECOTANK

2.2 New Zealand Upflow Rock Filter

An upflow rock filter was originally introduced in Northland, New Zealand in the early 1990s as a means of controlling solids carry over from septic tanks (Gunn, 1994). Since then an integrated two-stage septic tank and upflow rock filter, known as an *ECOTANK*, was developed by the Dunedin On-site Management Team, New Zealand to improve the efficiency of the first upflow rock filter. This was jointly developed by Richard Davies, a Waste Consultant, Dunedin (Richard Davies, pers. comm. July 2010). Some performance testing was undertaken but the results are not available.

Any solids carry-over from the septic tank is entrapped in the upflow process. This actually caused blocking problems in the longer term as no effective outlet filters were available at the time (Robbie Ludlow, Dunedin City Council, pers. comm. 2010). The writer concurs with Richard Davies that installing effluent filters would maximise the life of the upflow rock filter units. This system provides a better alternative to two pre-cast place in series. The unit is seen as providing a factor of safety in prolonging the life of disposal fields. Refer to Figure 5.2 for a layout and section of the *ECOTANK*.

2.3 Aerobic Upflow Pebble Clarifiers

This system was developed by Banks (1965) and it is a means of clarifying effluent by upward flow through a shallow bed of pea gravel. For this reason the system is referred to as a Banks filter. Clarification is achieved by straining or filtration, but it is apparent that both flocculation and adsorption play their part. However, interesting observations of laboratory clarifiers suggest that they operate mainly by flocculation, since comparatively large flocs of humus may be seen in the interstices of the beds and falling away from them. Flocculated material also settles on the surface of the beds (Truesdale and Birbeck, 1966).

A typical section of the aerobic upflow pebble filter is shown in Figure 5.3 (Bartlett, 1971).

The method can be used with most existing settling or humus tanks or as a separate unit. Effluent is passed through a 150 mm layer of pea gravel (4-10 mm) supported on a perforated or wedge wire floor. Solid material which accumulates in the bed is removed from time to time by back washing, that is, by lowering the water level and washing with a high pressure hose of recycled effluent. Banks (1965) has stated that if the clarifier is deliberately over-sized, it may be possible to leave it unattended for several weeks.

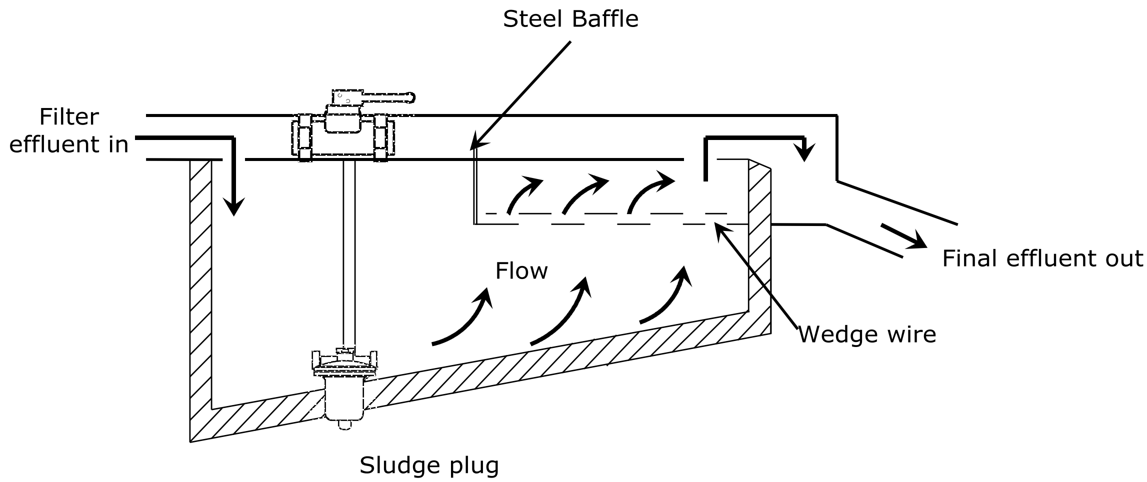


Figure 5.3: Section of Upflow Pebble Filter (NTS)

Sparkman (1970) reports the re-introduction of the pebble bed upflow filters. He maintained that they will achieve improvements in effluent quality of up to 90% on flow rates in the order of 1 m^3/m^2/hour.

The efficiency of an upflow pebble clarifier largely depends on the flow rate, the quality of the influent, gravel size, gravel depth and backwash frequency. An early upward flow pebble clarifier, at a laboratory scale, reduced the suspended solids by nearly 50% (Truesdale and Birbeck, 1966).

Banks (1965) reports that it is possible to achieve 63% BOD_5 and 93% suspended solids removal. Downing (1970) reports 30% BOD_5 and 50% suspended solids removals, at a flow rate of 23 m^3/m^2/day.

A further development was reported by Pullen (1972) using a floor of wedge wire panels, without the pebble media. He also reports that wedge wire and Banks pebble clarifiers can achieve similar performances for suspended solids.

Various hydraulic loading rates for upflow pebble clarifiers are given in Table 5.4 (DPI, 1992) **-** table compiled by the writer**.**

Table 5.4 - Hydraulic Loading rates of Aerobic Upflow Pebble Clarifiers

Method	**Rate at DWF** ($m^3/m^2/d$)	**Peak flow rate (3)** ($m^3/m^2/d$)	**Reference**	**Comments**
Gravel bed	14	23	Truesdale and Birbeck (1967)	50% suspended solids reduction
Gravel bed	14	23	Bartlett (1971)	For larger plants
Gravel bed	14	18	Bartlett (1971)	For smaller plants
Gravel bed	-	28	Bartlett (1971)	Maximum for activated sludge plants.
Gravel bed	20	Not given	Wilson (1981)	Or less, depending on degree of pre-treatment
Gravel bed	25-30 (1)	Not given	Pers. comm.	Reported as operating satisfactorily in NZ and Fiji (Simpson pers. Comm. with Harrison and Grierson Consultants, July 1983)
Gravel bed	23	Not given	Downing (1970) and Truesdale (1970)	
Gravel bed	23	Considered the maximum.	Bartlett (1971)	Work by Water Pollution Research Laboratory, UK. (3)

Notes:

1. Equivalent to conventional settling tank overflow rates
2. Means maximum desirable, at flows greater than DWF
3. Rates of 14 to 18 $m^3/m^3/d$ are the maximum if 50-60% suspended solids removal are desired

A vertical inlet baffle should be provided to achieve even distribution of flow beneath the pebble bed, and this should project at least 300 mm below the perforated floor (BS 6297:1983). The writer would concur with the installation of this baffle. The writer has overcome many short circuiting problems in the design and operation of wastewater treatment units with the use of baffles and weir plates.

2.4 Upflow Pebble Clarifier – Whangaroa Harbour, Northland, NZ

Having an interest in upflow pebble filters the writer studied the performance of a unit servicing a hotel and motels, some houses and a public toilet block. The design flow was 11,000 L/d and the peak flow was 34,000 L/d, a flow variation factor of 3. The pea gravel size was 6 mm to 9 mm. The filter was 3.6 m square giving a surface area of 13 m^2.

It was noted that there were no odours and complaints, which are definite attributes in harbour side communities. The final effluent discharged into the Whangaroa Harbour, which was used for recreational and commercial fishing and shell fishing. The discharged BOD_5 averaged 10 mg/L and the suspended solids 10 mg/L, which was very acceptable for such an environmental sensitive location.

This experience, the reports of others, and the experiences reported by *Harrison and Grierson, Consulting Engineers, Auckland and Suva* in 1983 gave the writer confidence in this relatively simple polishing treatment system. Consequently, when undertaking the review of the *Guidelines for the Planning and Design of Sewerage Schemes in Queensland* (1992-1992) the writer included this technology, as a secondary/tertiary treatment alternative in Volume 2 of the guidelines.

3 Multiple Chambered Septic Tanks and Treatment Fixtures

3.1 Multiple Chambered Septic Tanks

Cotteral et al (1969) reported on a double-compartment septic tank design, with an outlet baffle for improving performance. The first chamber was 2/3 and the second chamber 1/3 of the total capacity.

A British Standard (BSI, 1972) introduced double-chambered septic tanks for populations less than 100. Earlier the writer used BSI 1972 when designing larger than single household sized septic tanks.

Gunn (1974) adopted the 2/3 and 1/3 compartments and baffle approach and described it as the septic tank "rational design". The writer adopted this "rational design" in the mid-1970s. If the septic tank was not manufactured in a 1/3 and 2/3 form, brick and block baffle walls were constructed.

The writer was associated with the installation of a three chambered septic tank in 1976 with a co-researcher Alan Fielding in Whangarei, New Zealand. No sampling and testing on this single dwelling unit was undertaken but a visually improved effluent was obtained, in terms of organic content and suspended solids.

Laak (1980) studied multiple chambered septic tanks and concluded that they were superior to single-chambered tanks. The writer maintains the merits of multi-chambered septic tanks are as follows:

1. The buffering action undertaken within the first chamber, during high–rate short-term flows during laundry operations
2. The opportunity for progressive settleable BOD_5, settleable nutrients and solids removal
3. The final solids retention with the inclusion of an effluent filter at the system outlet, as discussed in this Portfolio.

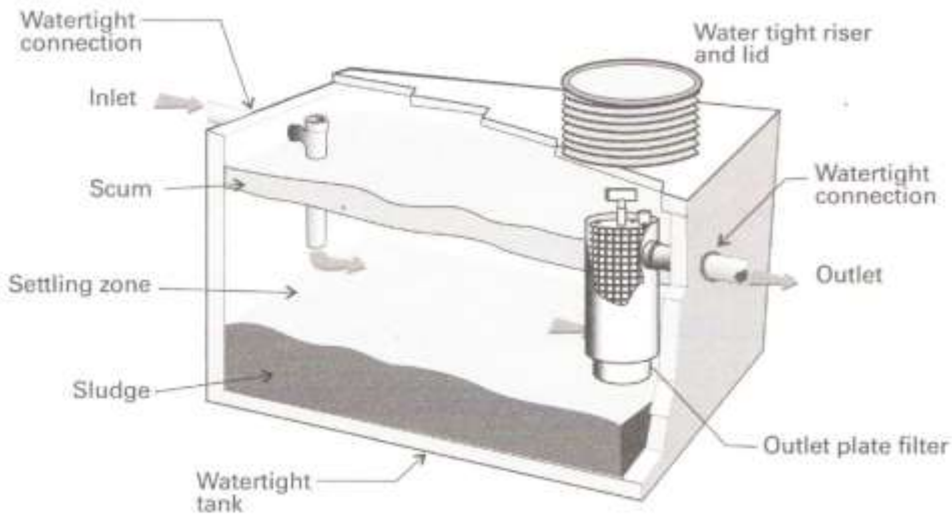
3.2 Septic Tank Outlet Filters

Septic tank filters are installed in the outlet tee of a septic tank and may be installed in new systems or retrofitted to existing systems. A review of septic tank outlet filters available in New Zealand and Australia was undertaken and reported by Stafford and Whitehead (2005). This study looked at designs, construction, installation and performance expectations.

Septic tank outlet filters can effectively reduce the discharge of gross solids (Crites and Tchobanoglous, 1998) and claims have been made by manufacturers that septic tank outlet filters are also effective in reducing BOD_5, suspended solids, fats, oils and greases (Lowhorn, 1999 and Zabel, 1999).

Currently, the fitting of septic tank outlet filters is not mandatory in many jurisdictions in Australia. However, there is compelling evidence in the USA and New Zealand that significant performance improvements can be gained and the life of the treatment/disposal system that follows can be extended considerably. The revision of the Auckland Regional Council TP58 (ARC, 2004) considers that septic tank outlet filters as "the norm" and requires them in all single, two stage or multiple chambered tanks (Stafford and Whitehead, 2005). Refer to Figure 5.4 for a septic tank outlet filter, also known as an effluent filter.

The assessment of the performance of septic tank outlet filters was updated and reported by Stafford and Whitehead (2007). The testing of four outlet filters for periods between 5 to 8 weeks demonstrated highly variable performances. Two filters at Martinsville, New South Wales with a low hydraulic loading and low pre-filter BOD_5 and suspended solids achieved minimal improvement to the post-filter effluent. However, on one occasion the BOD_5 was abnormally high and more than 50% reduction was achieved. The performance of two filters at Cardiff Heights, Newcastle with higher loadings and more frequent surges was again variable. Greater than 50% reductions in BOD_5 and suspended solids were achieved.



Reference: Auckland Regional Council TP58, 2004.

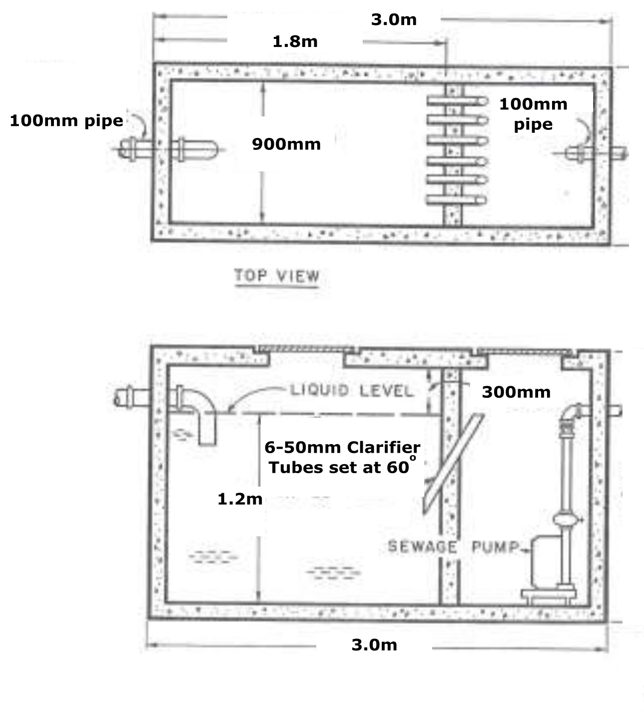
Figure 5.4: Detail of Effluent Filter (TP58)

Further work was undertaken at a concrete factory in Rockhampton. The study showed the following (Stafford and Whithead, 2007):

1. Outlet filters are more effective when BOD_5 and suspended solids are at high concentrations
2. In the case of lower levels of BOD_5 and suspended solids the performances are less significant
3. Outlet filters will prevent the passage of gross solids impacting on downstream disposal systems
4. The short term study does not present evidence that the outlet filters can reliably and consistently remove BOD_5 and suspended solids
5. There is clearly a need to thoroughly examine the performance of a wide range of filters and to undertake long term studies on their performances, loadings and maintenance characteristics.

3.3 Tanks with Tube Settlers or Clarifiers

A housing research project involving 31 homes, was undertaken by the Tuskegee Institute, the Alabama Agricultural Research Institute and the Farmers Home Administration (Willson et al, 1974). A modified septic tank was used for this project. Effluent was drawn off from the digestion chamber by a group of 50 mm diameter clarifier tubes sloped at 60 degrees to the horizontal, so that the particles settled out of the stream and slid back into the digestion chamber. The lower ends of the clarifier tubes were cut vertically to prevent gas bubbles from carrying solids up the tubes. Refer to Figure 5.5 for a typical detail of effluent tube settlers (Willson et al, 1974).



Reference: ASAE, 1974

Figure 5.5: Detail of Effluent Filter (TP58)

The writer adopted a dual septic tank/sand filter system in Northland, New Zealand incorporating 100 mm diameter PVC sloping tube clarifiers in 1977. The system was designed specifically for loadings higher than the normal single home, in situations where soil conditions tended to be marginal or sub-standard or groundwater levels were a potential site constraint. A section of this system is provided in Figure 5.6.

This was a consulting project rather than a research project, so no monitoring was undertaken by the writer. However, based on the fact it was a dual chambered septic tank, it contained 60 degree sloping clarifier tubes and it was followed by a sand filter. The resulting effluent was targeted to achieve a high quality.

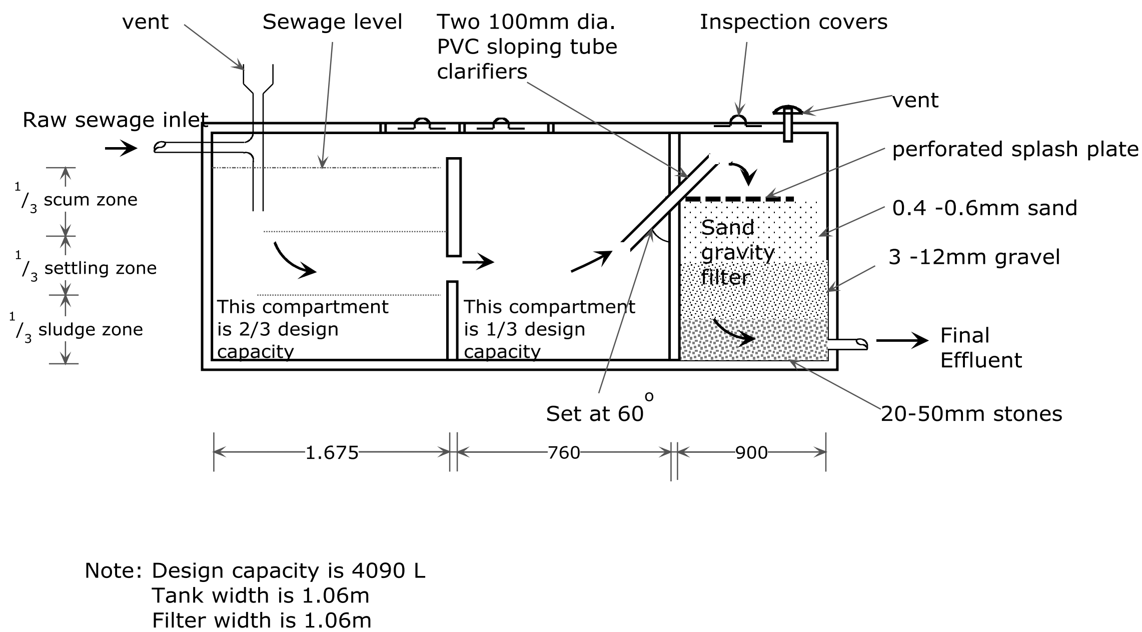


Figure 5.6: Section of Septic Tank/Sand Filter/Tube Clarifiers (NTS)

4 Optional Innovative Effluent Disposal Systems

4.1 Background

The writer developed a specific interest in site evaluation and optional effluent disposal systems from 1975. He recognised that there was very limited guidance on site evaluation and few effluent disposal alternatives were available.

The writer compiled a paper "The Evaluation of Sites and Alternatives for On-site Effluent Disposal" (Simpson, 1986). The alternative disposal systems reported included:

1. ET/adsorption systems
2. Total ET trenches
3. Deep shaft disposal (reported in Portfolio 7).
4. Raised mounds (*Nodak* type)
5. Shallow- narrow trenching reported later in this Portfolio.
6. Deep trenches reported later in this Portfolio.

The AS/NZS 1547:2000 offers a limited number of effluent disposal systems, that is, limited to the extent that the joint standards covers two countries and a large range of soil types and climatic regions. The different effluent disposal systems are listed in Table 5.5.

Table 5.5: Effluent Disposal Systems in AS/NZS 1547:2000.

Type of Effluent System	System Suitability (AS/NZS 1547:2000 Table 4.2B1)	Comments (by Simpson)
Conventional absorption trenches	More suitable for higher and medium permeability soil profiles. Requires benching in moderate and steeper slopes – this can introduce a degree of risk by downhill seepage.	Limited to 450 mm depth, hence must be within suitable upper soils. Depth to groundwater is more critical in more permeable soils, with primary effluent. Not suitable for slower permeability soils. Suited to larger blocks, say over 1,200 m^2 since 100% reserve area required by AS/NZS 1547:2000. Consider alternating trenches to enhance aerobic treatment and system longevity.
Conventional beds	Soils with reasonable permeability required.	Beds not considered being as efficient as trenches (Portfolio 1). The writer has seen several beds in the Caboolture area fail in slow permeability soils. Less versatile system since ET restricted hence, soils should be restricted to categories 2 to 4. Consider alternating trenches to enhance aerobic performance and improve system longevity.
Boxed trench		Very limited use in NZ and Queensland, in his experience
Mounds	Wisconsin mound was originally designed for higher water tables and shallow rock. Designed specifically for soil categories of 4 to 6. Can be suitable for soil categories 1 to 3.	Exposed sites more beneficial. There is a risk of failure, by toe seepage, if sited on sloping surfaces. There is a possible risk of failure if high rainfall coincides with low ET conditions.
Spray and drip irrigation	Suitable if land is available with separation areas.	Slope limitation, to avoid runoff. More suited to warmer climates with ET potential.
ETA (1)	Combined ET and seepage systems, suited for slower permeability soils categories 4 to 6.	To avoid possible failure these systems are more suited to lower rainfall and higher ET and good site exposure.

Category 1 trench	Designed for category 1 soils – gravels and sands	Offers up to 1.0 m depth. Has more potential for a broader range of applications since several soil horizons can operate under head conditions. Has an ET component on the top section.

Notes:

1. Denotes evapotranspiration/infiltration systems
2. This suitability can be questioned – higher soil categories would not necessarily need a mound and the infiltration rate would tend to dominate, causing possible cover vegetation stress, due to lack of moisture to sustain growth.

The soil categories and matching textures in AS/NZS 1547:2000 are described as follows:

1. Category 1 – Gravels and sands
2. Category 2 – Sandy loams
3. Category 3 – Loams
4. Category 4 – Clay loam
5. Category 5 – Light clays
6. Category 6 – Medium to heavy clays.

The fundamental needs or requirements of effluent disposal systems are as follows:

1. To undertake further treatment, particularly within trenches and beds and mounds (see Portfolio 2)
2. To suit the soil profiles and ground water levels
3. To contain the effluent, particularly with trenches, beds and mounds.

4.2 Shallow - Narrow Trenching

The writer developed a shallow- narrow trench system in the late 1970s whilst working on a number of greywater management projects. It was designed to suit sloping terrains covered in native vegetation. The writer reported this trench system in the New Zealand Journal of Environmental Health (Simpson, 1986). The writer found that a 200 mm wide and 350-450 mm deep trench, complete with overflow weirs at the end of each run of trenching, could be installed by hand excavation, to minimise site and vegetation disturbance.

The main advantages were:

1. Very minimal site and vegetation disturbances

2. The trench depth was generally located in more permeable upper soils
3. The remaining vegetation enhanced slope stability
4. The vegetation promoted disposal by ET
5. The trenches were laid level, along the contours, but they could be laid slightly out of line using flexible pipe, to avoid larger shrubs or trees.

The writer understands that the shallow-narrow trench system performed in an acceptable manner on the basis that the Health, Plumbing and Drainage Inspectors continued to approve them in many local authorities. Refer to the detail of a shallow-narrow trench in Figure 5.7.

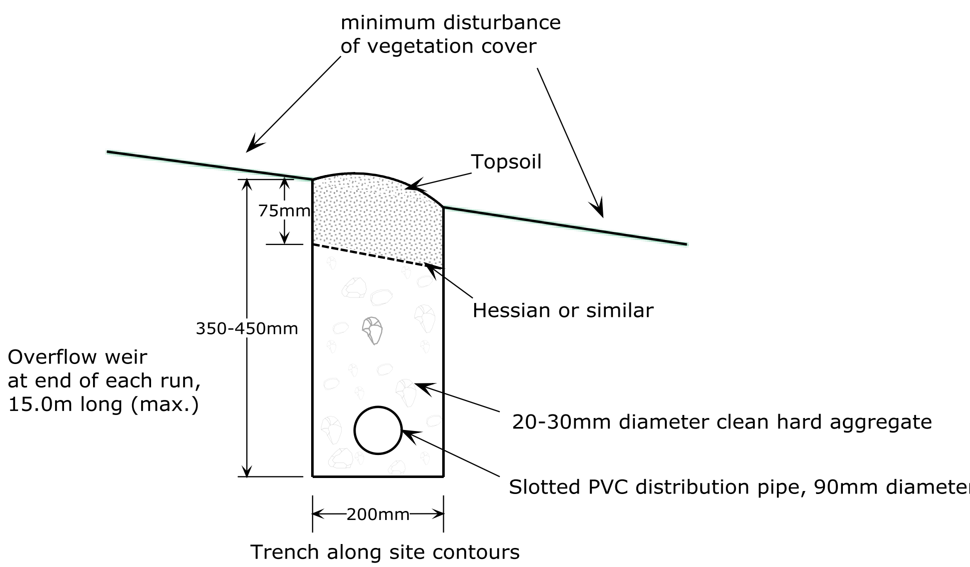


Figure 5.7: Section of Shallow Narrow Trench (NTS)

4.3 Deep Trenches

Some areas are characterised by clay or iron pans, as in the Northland Province and Auckland in New Zealand. These soil profiles offer more permeability with depth, via weathered rock horizons. When the deeper soil profiles and the groundwater levels have been identified, a deep trench can be designed. Two deep trenches can be used, in conjunction with a distribution box, on an alternating dose and rest basis.

As the side walls are unevenly loaded during the draining process, the "most conservative" design loading should be applied. The design surface is the total side wall area from both sides of the trench (Gunn, 1994). Refer to the detail of a deep trench system in Figure 5.8.

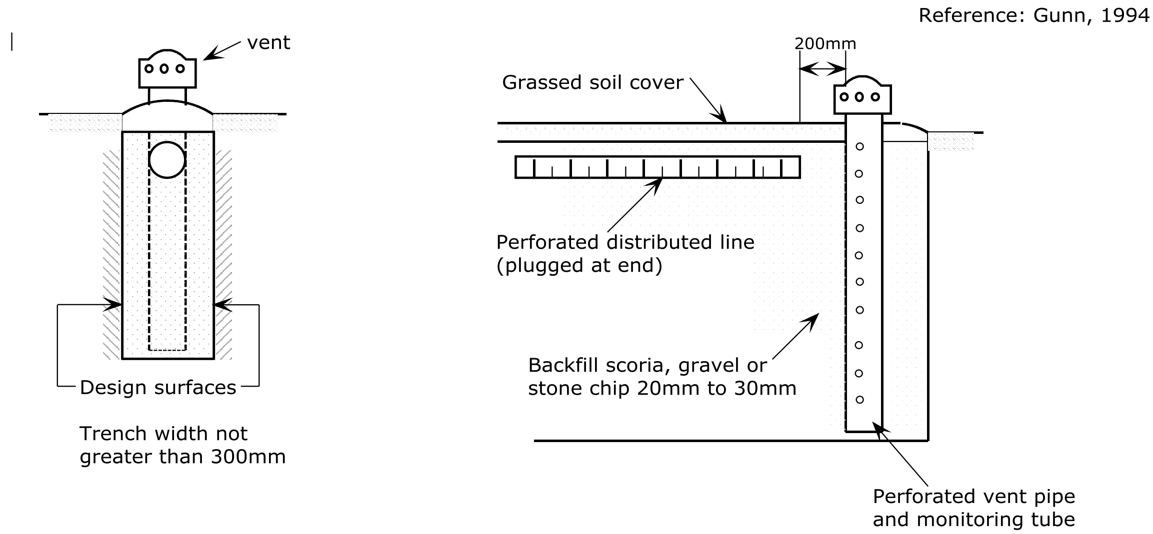


Figure 5.8: Section of Deep Trench Disposal (NTS)

The advantages of deep trenches include:

1. Varying soil profile, often with a more permeable stratum deeper down
2. System operates under head conditions, particularly under peak loads and adverse weather conditions
3. Ample storage is available; the trenches can have a larger diameter aggregate, with increased porosity
4. The dosing and resting sequence is very beneficial (reported in Portfolio 2)
5. PVC air vents can be readily installed to encourage aerobic conditions (reported in Portfolio 2)
6. Enables on-going effluent treatment, by trickling filtration, to take place (reported in Portfolio 7)
7. There can be some saving in land area requirements, due to the higher infiltration rates obtained
8. Some nutrient reduction can take place, with aerobic conditions nearer the surface and anoxic conditions lower down.

4.4 Seepage Pits

The writer was instrumental, in the mid to late 1970s in developing a seepage pit system for use in very sandy soils in estuarine areas of the east coast of Northland, New Zealand. It was very difficult to construct conventional trench systems since the wet sand caused cave-ins. Two solutions were used. Firstly precast concrete pipes and secondly, pipes made from porous concrete. The pipes were sunk gradually by the removal of the sand.

The precast pipes, placed vertically, had holes in the sidewalls. The pipes were surrounded with large rocks. A concrete base allowed the collection of solids, for periodic removal. Refer to Figure 5.9 for a typical section of a seepage pit.

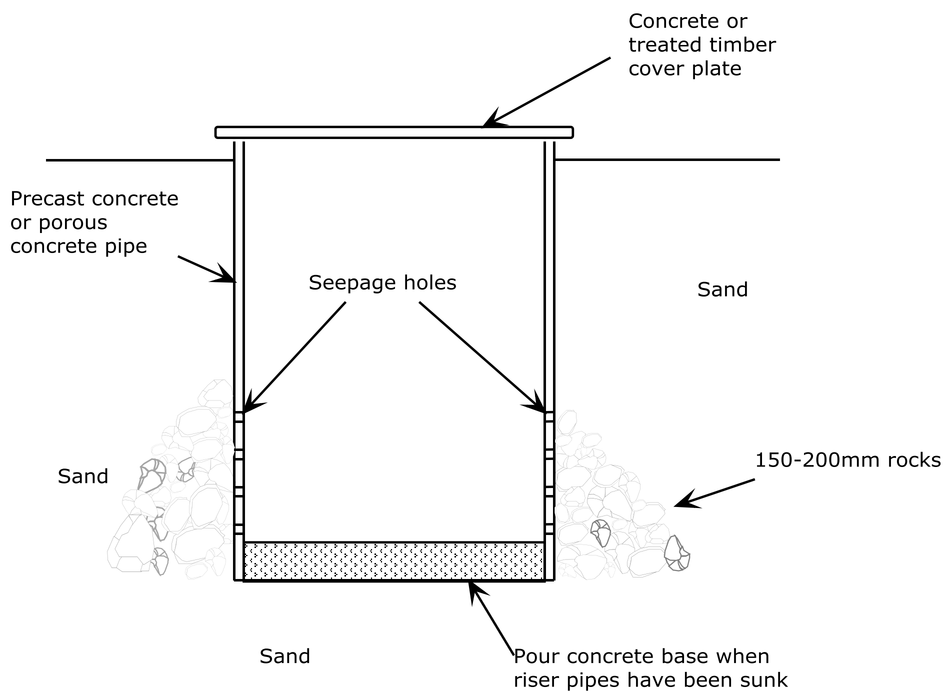


Figure 5.9: Section of Seepage Pit

These seepage pits were found to be most suitable for sandy and wet situations. The main advantages were that they could be readily de-silted, on a needs basis, and they were superior to conventional shallow trenches.

4.5 Pressure Distribution Trench for High Permeability Soils

In 2002 the writer designed a simple distribution trench for use when effluent was pumped and the soils possessed a high permeability. This consisted of the following components:

1. 25 mm diameter distribution pipe work, with 3 mm holes each 1.0 m run
2. The pipe surrounded with 20-40 mm clean hard aggregate
3. The aggregate wrapped in a geotextile or hessian.

4. The above system within a 1.0 m wide, 200 mm deep trench, backfilled with sandy loam or coarse sand.

The 20 to 40 mm aggregate allowed the distribution of the pressurised effluent. The geotextile or hessian prevented the ingress of sandy loam into the 20-40 mm aggregate. The effluent quality was secondary so there were no concerns of the geotextile becoming blinded or blockages within the coarse sand of sandy loam. This system was applied in cases where the effluent was treated to a secondary standard and spray irrigation was not the preferred effluent disposal option. Refer to Figure 5.10 for a typical section of a pressure distribution trench system.

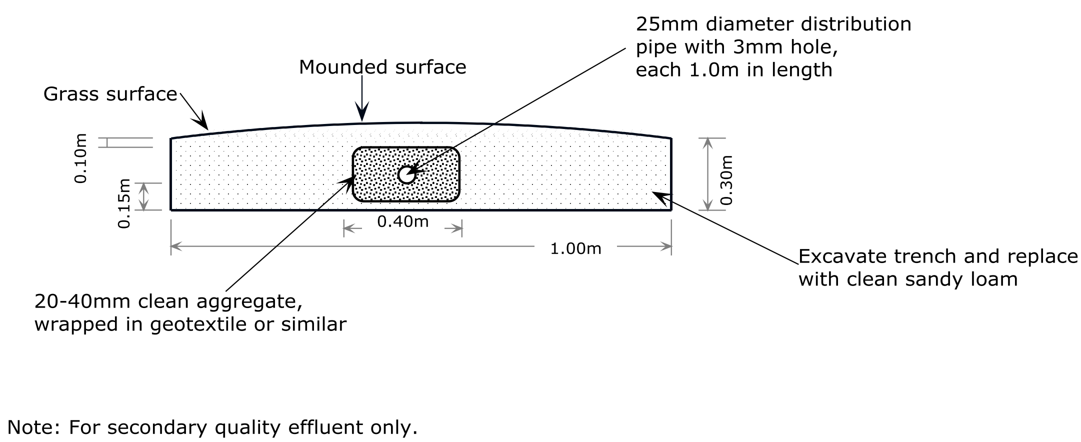


Figure 5.10: Section of Pressure Distribution Trench (NTS)

4.6 Dual ET/Adsorption Trenches

Occasions have arisen in the Caboolture area where the upper soil horizon (up to 450 mm depth) has not been entirely suitable for infiltration trenches. More permeable stratum has been identified at a depth of around 1.2 m. The ideal zone to dispose effluent would otherwise require a 1.2 m or deeper trench. This is not economic, with the cost of trench aggregate.

A 1.2 m wide, 400 mm deep upper trench has been constructed. The surface is planted and this section functions by largely ET disposal. A series of 200 mm diameter post hole bores are sunk down to the permeable zone, to allow the passage of effluent and disposal by infiltration. Refer to the section of a dual ETA trench in Figure 5.11.

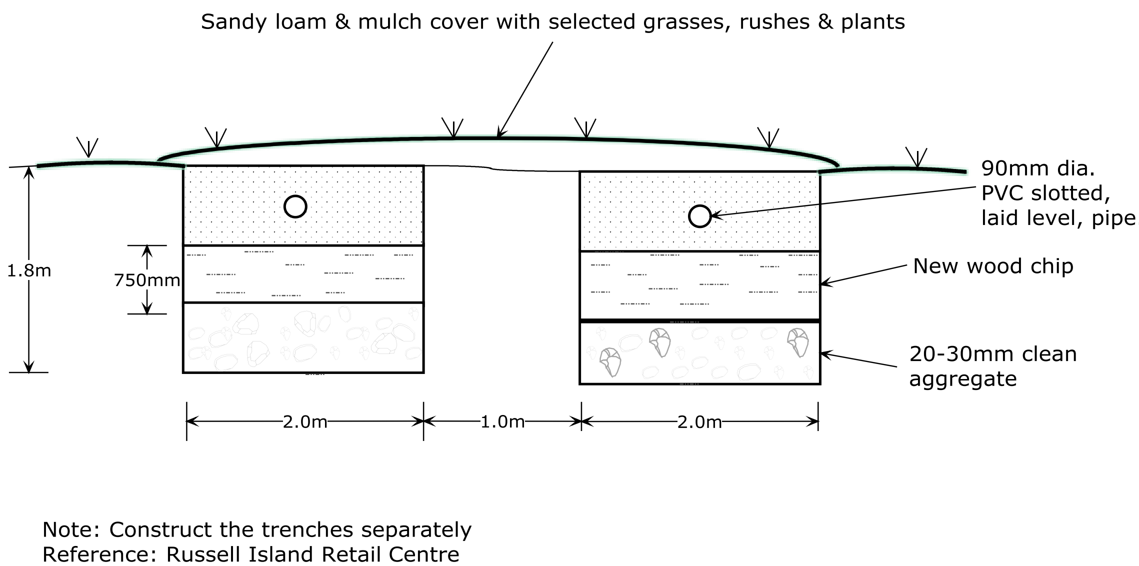


Figure 5.11: Section of ET/Adsorption Trench (NTS)

The design of a site specific wastewater treatment and effluent disposal system for a retail centre on Russell Island, Moreton Bay is listed in Table 1.2 of Chapter 2 as an innovative project undertaken in 2010. Refer to the section of this dual ET/Adsorption trench in Figure 5.11.

Since the property had an area of 3,300 m^2, the main site constraint was one of available land area since the lot was to contain 10 retail shops, a caretakers unit, an effluent disposal area, stormwater bio-remediation trenching, stormwater storage tanks, site landscaping and car parking for 32 vehicles.

The total design wastewater flow was 7.10 m^3 per day. The area of shallow trenching required was 560 m^2 with associated landscape gardens. This area was not available. The option of spray irrigation was not feasible due to the health risk to the public and the lack of available land area. Deep shafts disposal (to a depth of 6.0 m) was not an option due to the likely presence of groundwater.

The site constraints in this case study presented the writer the opportunity to highlight his broad and innovative experience. This project required a completely "off the shelf" system for effluent disposal. The site assessment and design process was as follows:

1. Assessment of the site and soil findings
2. Determination of the proposed building area, parking area and landscaping area, from the architect
3. Sourcing of the area required for stormwater treatment, from civil engineering consultants

4. Calculation of the area available for effluent disposal
5. Assessment of the suitability of conventional effluent disposal options
6. Development of the option of deep trenching, considering the reduction in land area requirement and the values of effluent wet weather storage, trickling filtration on-going treatment and bark treatment.

The key design aspects involved in the development of this "multi-functional" effluent disposal system included:

1. No signs of groundwater or semi-saturated soils within a 3.0 m pit located in the lowest point of the property
2. A very favourable soil profile consisting of sandy loam, sandy clay loam into light clay, into sandy clay
3. A definite site area constraint, hence an innovative deep trench system would be required
4. Wet weather storage would be available within the trench system
5. On-going treatment available by trickling filtration
6. Some potential nitrification by the upper aerobic zone and an in-situ wood chip zone
7. Some opportunity for de-nitrification within the anoxic bottom zone of the trenching
8. Distribution pipes positioned just below the trench surface, to optimise trickling filtration
9. Distribution pipes positioned just below the planted surface to also facilitate some effluent treatment and loss by evapotranspiration.

Due to the difficult economic situation the construction of this project has been deferred.

4.7 Deep Shaft Disposal

Deep shaft disposal is discussed in detail in Portfolio 6. This is an innovative effluent disposal alternative with potential in areas with low permeability upper soils and lower water tables.

4.8 Planted ET/Adsorption Raised Beds

With the prevalence of lower permeability and clay soils within the Caboolture region the writer developed, with the assistance of Council Plumbing Inspectors, a concept of effluent disposal using the following mechanisms:

1. Utilising the degree of soil permeability available, as determined by constant head percolation testing and soil K calculations
2. Constructing a raised garden bed, to achieve some exposure to wind and sun, as with a sand mound
3. Planting the bed with selected grasses, plants and smaller shrubs
4. Covering the surface with wood chips, bark or mulch, to suppress weed growth
5. Sloping the bed surface for rainfall runoff
6. Disposing effluent, preferable by pumping, within a distribution network.

Some of the former Caboolture Shire Council designed beds encountered the following problems, which were addressed by the planted ET/Adsorption bed disposal concept:

1. Installing these systems well before house construction led to weed infestation becoming a problem
2. Not having an appreciable surface slope for rainfall to run off
3. Not having a facility for draining off accumulated rainwater from the bed surfaces.

Refer to a section of the planted ET/Adsorption raised bed in Figure 5.12. Effluent is distributed over the surface by droplet type spray irrigation.

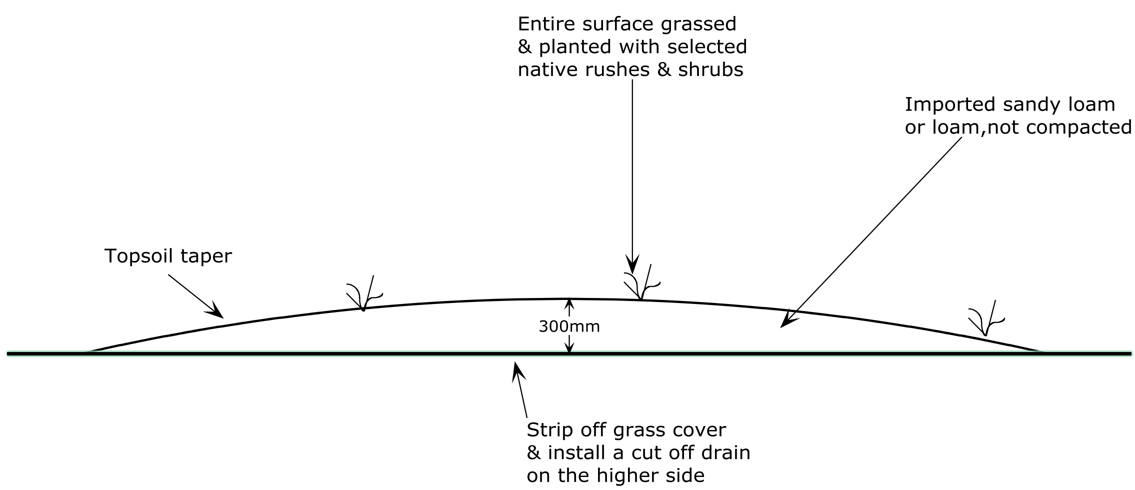


Figure 5.12: Section of Planted Raised ET/Adsorption Trench

5 Reflections and Recommendations

In the writer's experience, since the early 1970s, various councils have tended to select so called "standard'' effluent disposal systems such as shallow trenches. This has often been done on a personal preference basis, sometimes without any engineering, scientific and environmental input or experimentation. Reflections on several wastewater treatment alternatives that could be considered more often follow.

5.1 Banks Pebble Clarifier

It was reported by Bartlett (1971) that the Banks pebble clarifier was probably the simplest and most effective method of secondary and tertiary treatment available for smaller works. The writer had much confidence in the work of Bartlett during his earlier days in public health engineering. Reflecting back on the Banks pebble clarifier and the writer's own water quality engineering experience, he concurs with Bartlett.

The Banks system is robust, readily maintained by simple backwashing, and it produces very good performance results. The writer has noticed over the past 40 years that in wastewater engineering often simple, effective and robust treatment processes are overlooked, in preference for higher technology options. However, on the other hand these simple options can be recycled many years later. In the writer's opinion it is also a major benefit that the Banks pebble filter can be readily retrofitted to an existing humus or settling tank.

It was earlier reported that the main treatment mechanisms were filtration and straining, with some flocculation and adsorption. The writer feels that in the case of the wedge wire clarifiers, the main mechanism is physical straining and filtration. Minor flocculation may take place immediately above the wedge wire. In the case of the Banks pebble clarifier the writer can relate to straining and filtration being the initial mechanisms and flocculation and adsorption being the follow up and enhancing mechanisms.

Pullen (1971) reports that best results are obtained when a sludge blanket has formed on the pebbles or the wedge wire. On reflection, this indicates the value of flocculation and adsorption as treatment mechanisms. It is reasonable to suggest that backwashing is only undertaken when performance results or effluent clarity observations show there is a need to take filter cleaning action.

5.2 Wood Chip Trenches

Wood chips and good quality bark have the ability to treat wastewater and greywater, as reported in Portfolios 1 and 4 and the experience of Dr Chris Tanner (NIWA, NZ, pers.com July 2009).

Giving wood chip filters some more thought, a trench containing a distribution pipe, surrounded by clean aggregate, in a trench backfilled with wood chip would be an innovative alternative to effluent disposal. Refer to the detail of a wood chip trench in Figure 5.13.

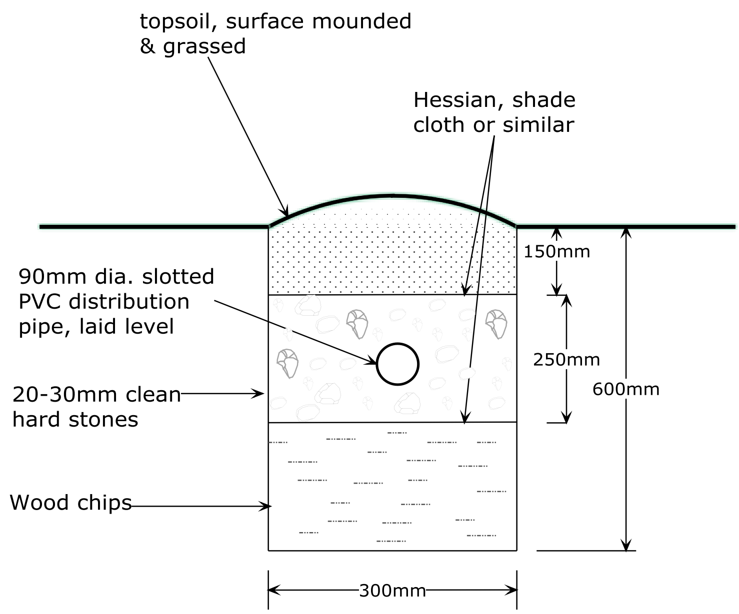


Figure 5.13: Section of Wood Chip Trench (NTS)

The aggregate surround will enable good initial effluent distribution. The wood chip will act to:

1. Breakdown organic matter
2. Undertake nitrification
3. Undertake de-nitrification within an anoxic zone in the lower section of the trenching, using the wood chip carbon source.

It is possible that some trench settlement, due to the degradation of wood chips, may take place. In this case the topsoil should be topped up.

5.3 Anaerobic Upflow Pebble Filter

It is the writer's opinion that the anaerobic upflow pebble filter for treating primary effluent has considerable potential for individual and larger institutional type applications.

Giving this technology option more thought it follows that with (say) a 15% increase in the design filter surface filter area to increase the detention time, an anaerobic upflow pebble filter operating in Queensland and NSW sub-tropical and tropical conditions would be capable of high overall contaminant reductions. The warmer all year weather conditions would enhance metabolism rates, as already discussed in this thesis in Portfolio 3.

Dr Vasantha Aravinthan (Senior Lecturer, University of Southern Queensland, pers. comm., 2012) feels that the key to the performance of anaerobic upflow filters could be due to the attached nature of the microbes working on different oxygen requirements. This is not subjected to wash out when flows are increased and is better able to handle shock or toxic loads. There is a capability for the provision of aerobic- anoxic and anaerobic conditions within the micro-areas in a single biofilm, promoting the growth of different microbes within this film. By reflecting back on the possible rationales for high BOD_5 and Total N treatment this technology is considered most worthy of further research.

6 Innovative Treatment and Disposal Conclusions

The AS/NZS 1547:2000, considering it covers the large combined area of New Zealand and Australia with a full range of climatic zones and varying soil types, does not offer a large range of effluent disposal alternatives. However, the Australian postal survey of effluent disposal systems (Simpson, 2001) shows that a full range of systems are in use. Some site constraints can demand the design of special or non-conventional disposal systems.

As a result of the writer's literature survey, past applied research, and an assessment of work by others have lead him to formulate the following conclusions on each technology alternative.

Anaerobic Upflow Pebble Filters

Anaerobic upflow pebble filters, following a septic tank, are effective, robust and an economic method of post-treatment. They are capable of very good BOD_5 and suspended solids reductions with the added benefit of being capable of reducing Total N. The Engineering and Water Supply Department in South Australia has reported they are capable of nearly 100% Total N reduction. The overall treatment capability of these simple units certainly warrants more research and development.

Upflow Rock Filters – New Zealand

It would appear that the two chambered septic tank and upflow rock filter, developed by the Dunedin group, has the potential for being another viable alternative. It is the writer's opinion that the problem of solids carry-over into the upflow rock filter can be overcome by the provision of a suitable outlet filter.

Banks Upflow Pebble Clarifier

There is merit in using Banks type upflow pebble filter units, on perforated plates and wedge wire, as a secondary treatment system. They are simple to operate, robust and readily maintained. In the writer's opinion the Banks pebble and the wedge wire clarifier has a definite application in small on-site units. Typical potential applications are for hotels, motels, schools, child care centres, camping grounds, caravan parks, churches, church camps, school camps and military camps. Such a unit would be easier to install in rectangular tanks. Such facilities in Queensland are most likely to require a Site-based Management Plan, so simple physical backwashing would not be a too onerous task.

Septic Tank Outlet Filters (Effluent Filters)

Septic tanks outlet filters have been in place in the USA and New Zealand for many years. These units are mandatory in South East Queensland. Since solids carry-over into effluent fields has been a major problem for many years, the writer supports their installation. However, the writer concurs with Stafford and Whitehead (2005) that there is a need to undertake more detailed performance studies.

Tube Clarifiers

This simple, robust and very economic system has much potential as a suspended solids trap. The efficiency of the tube clarifiers will mean that the septic tank compartments will need de-sludging on a more frequent basis.

Multiple Chambered Septic Tanks

Performance over the years has proven that multiple chambered septic tanks are superior to single-chambered tanks. There is merit in using the "rational design" approach proposed by Gunn (1974).

Shallow-Narrow Trenching

This special purpose trenching has much potential on sloping sites covered in existing vegetation. It can be laid along the contours and be used to avoid major trees as necessary.

Deep Trenches

Deep trenches are suitable if the upper soils have a low permeability and are not suitable for conventional trenches, but the permeability increases within deeper weathered zones.

Seepage Pits

Environmentally acceptable methods of effluent disposal in lower lying sandy areas are limited. The seepage pits facilitate this need.

Dual ET/Adsorption Trenches

Dual ET/Adsorption trenches are considered to be the ideal alternative if annual ET rates exceed the annual rainfall and if the upper soil horizons consist of low permeability soils, which are not acceptable for conventional infiltration trenches. These are covered in some detail in Portfolio 6.

Deep Shaft Disposal

This system has potential in soil profiles with poor permeability nearer the surface and more permeable stratum at depth. The ground water table must not be breached and slope stability must be verified. It is a very suitable alternative if land area is a constraint. This technique has been covered in more detail in Portfolio 7.

These technological alternatives are worthy of more focus in terms of practical application since in my experience they are considered to be robust, workable, simple and effective and readily maintained.

Trench Overflow Provision

There is considerable merit in having an effluent trenching system with overflow provision. The writer used such systems in Northland, New Zealand for many years, during the period 1975 to 1990. This avoided the need to duplicate trenching in the future due to increased water usage, house extensions and declining trenching performance. The overflow provisions included deep trenches (up to 1.5 m) and deep shafts (up to 6.0 m). Both overflow options also conserve land use requirements. This is particularly important on smaller lots.

Clarifier Tubes

The writer adopted the concept of fitting 60 degree clarifier tubes to draw off effluent from septic tanks, as reported by Willson et al (1974). This is such a simple and effective system for preventing solids carry-over into effluent trenches. The clarifier tubes must be well cast with extra plastering into the diaphragm wall and protected from possible damage when the septic tanks are de-sludged.

Septic Tank Outlet Filters (Effluent Filters)

The writer had some confidence in the performance of septic tank outlet filters, based on the experience of *ORENO* filters, in the USA and from the early 1990s in New Zealand.

The writer has had occasion to examine the construction of several brands of the filters in South East Queensland over the past 10 years. The writer's confidence in the performance of filters extended to the fact that he includes outlet filters in all his reports within northern NSW and Queensland.

The writer concurs with Stafford and Whitehead (2007) that long term research is needed to ascertain outlet filter efficiencies, and maintenance frequencies, under a range of loadings.

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Portfolio 6 – Evapotranspiration Systems and Nutrient Uptake

This Portfolio has been separated into Part (A) which covers evapotranspiration effluent disposal systems and Part (B) which covers nutrient assimilation within effluent disposal systems. They have a common link in that evapotranspiration systems rely on selected vegetation to transpire effluent. This vegetation also plays a major role in the assimilation of nutrients.

Part A

<u>Part B</u>

Part (A) Evapotranspiration Effluent Disposal Systems

1 Introduction, Aim and Technology Background

The aim for Part (A) of this Portfolio is to review and document the writer's experimental work on the method of effluent disposal by evapotranspiration (ET).

The objectives are to:

1. Review the technical literature on the history of ET system development and the mechanisms involved in ET
2. Document the writers work on ET in New Zealand over the period 1974 to 1986 and in the mid-1990s in South East Queensland, in association with others
3. Document the research by others on ET
4. Develop a model evapotranspiration/adsorption (ETA) design for Queensland, based on the findings of the above objectives
5. Reflect on ET and establish any needs for research and developments
6. Draw conclusions.

Publications by Simpson Appropriate to Portfolio 6

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2. Simpson, J S 1978a, Tikipunga Reserve (Rugby Club), Whangarei – Report on Evapotranspiration/Infiltration Bed System, Whangarei City Council
3. Simpson, J S 1978b, Report on Design of W Finn Evapotranspiration/Infiltration Bed System, Otamatea County Council
4. Simpson, J S 1978c, Report on Design of M Ashford, Evapotranspiration/Infiltration Trench System, Whangarei City Council
5. Simpson, J S 1981, Evapotranspiration – Infiltration Disposal Systems in Northland, *Alternative Sewerage Seminar*, University of Auckland, 24 - 25 November
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1.1 ET Related Terminologies

When designing ET systems, it is important that practitioners are aware of the various terms associated with soils, vegetation and evapotranspiration. Patterson (2002) discusses ET design and system functioning terminologies as follows:

1. Crop factor – is the coefficient expressing the proportion of open water evaporation transpired by vegetation under the same energy gradient, varying with the growth stage, plant type, plant density, sunlight, and wind and soil conditions. For vigorously growing, well-watered plants the coefficient of 0.7 to 0.9 may be used
2. Capillary water – exists as a continuous film around soil particles, held by surface tension that drives the water through very small pores. The movement of water laterally and upwards works like a wick. The writer feels the action is best described as capillary rise or wicking
3. Humidity gradient – or vapour pressure gradient, indicates the direction and intensity of evaporation, moving from a higher to a lower humidity. This gradient is influenced by temperature, solar radiation, wind and surface conditions
4. Void ratio – the ratio of volume of voids to the volume of a given mass of soil. The voids may be filled with air or water
5. Evaporation – the movement of water from the surface of a body into the atmosphere in response to a lower humidity gradient
6. Transpiration – the process of water vapour passed into the atmosphere through the tissues of living plants. More water passes from the leaves when water at the roots is not limiting
7. Soil water storage – the amount of water than can be stored in the pores spaces of a confined ET bed or trench
8. Porosity – the ratio of volume of voids to the volume of solids in a given mass of soil, sand or gravel.

Another term directly associated with ET is the “clothes-line” effect. Trees and shrubs with a large silhouette catch more advected heat, which is known as the “clothes-line” effect (Tanner and Bouma, 1975).

1.2 Types of ET Systems

Essentially , an evapotranspiration (ET) trench or bed is a constructed sponge of sand and gravel, sealed from the surrounding soil, with an inbuilt water storage capacity and a vegetated surface from which ET is maximised (Patterson, 2002). On-site evapotranspiration (ET) trenches and beds are designed to disperse effluent exclusively by the combined mechanisms of evaporation and vegetation transpiration.

An evapotranspiration/absorption (ETA) trench or bed is a sub-surface system designed to dispose of effluent by both ET and soil absorption. Both these systems are usually preceded by primary pre-treatment units, such as septic tanks, to remove settleable and floating solids. A total evapotranspiration (TET) trench or bed has a liner in the base and it relies on the sole disposal mechanisms of ET.

The surface of ET and ETA systems is planted with water-tolerant or (more correctly) hydrophyte vegetation. The ideal situation for ET purposes is plants and other forms of vegetation that pass water rather freely, depending on various other environmental factors. Therefore, they usually transpire freely and for ET purposes are reasonably tolerant of water-logged soil conditions for varying times (Alan Fielding, Whangarei, pers. com. June 2010). Effluent is drawn upward through fine medium by capillary action and then evaporated or transpired into the atmosphere. The size of the fine medium is very important to encourage capillary action. Media sizes are discussed later in this Portfolio.

The actions of ET in mounds, trenches and beds have been best described by one of the system pioneers, Bernhart. Bernhart (1973) describes them as follows:

1. Evaporation, measured as open water evaporation
2. Evaporation, due to microbial energy
3. Evaporation, due to increased activity of the open water surface
4. Transpirations by plants, usually via the stomata on leaves
5. Additional transpiration by luxuriant growth, which is stimulated by wastewater nutrients.

Evapotranspiration is not easily measured directly, but it can be calculated or estimated by the means of meteorological data. The accuracy of the estimate depends on the data available and the nature of the evaporating surface (NZ Met. Service, 1977). ET and ETA systems both rely on selected grasses, plants, shrubs and trees to transpire water. The selected vegetation also plays a role in the uptake of nitrogen and phosphorus. Refer to Part (B) of this Portfolio for reporting on nutrient uptake in on-site effluent fields.

1.3 Role of Leaf Stomata and Transpiration Rates

It is clear that stomata on the underside of leaves play a role in regulating water loss from terrestrial vegetation. The stomata occupy a central position in the pathways of both water losses from plants and the exchange of carbon dioxide. It is commonly assumed that they therefore provide the main short term control of transpiration and photosynthesis, though the detailed control criteria on which they are based are not well understood and they are likely to depend on each particular ecological situation (Jones, 1998). Incomplete stomatal closure during the night has been observed in a diverse range of plant species and can lead to substantial water loss (Caird et al, 2007). The magnitude of water loss during the night depends on vapour pressure difference between the leaves and the air, as well as canopy structure and atmospheric mixing. Night time transpiration rates are typically 5 to 30% of day time rates.

Maximum transpiration flow depends on the following botanical aspects (Fielding, 1977):

1. Extent of vegetative cover
2. Degree of active growth throughout the year
3. Leaf area
4. Density of the stomata
5. Ability of the stomata to close
6. Extent and depth of the root mass
7. Absorption ability of the roots
8. Climatic and soil/ water conditions.

Different plants transpire at different rates. The ratio between evapotranspiration/total evaporation is often called the crop factor (Floyd, 2001). Typical crop factors from Reid (1981) are as follows:

1. Lucerne has a crop factor of 0.95 in January and 0.55 in July
2. Pasture has a crop factor of 0.70 in January and 0.40 in July
3. A deciduous orchard has a crop factor of 0.75 in January and 0.15 in July.

Deciduous trees lose their leaves in winter and they have low ET rates, as low as 0.10 in June (Floyd, 2001). This aspect is discussed further in the Reflections in Section 4. Some common turf grasses can develop deep root systems. For example, kikuyu has roots up to 2.4 m, paspalum and couch up to 1.5 m and buffalo grass up to 1.0 m (Floyd, 2001). This would enhance moisture and nutrient uptake within ET systems.

1.4 California ET System Design

The California State Water Resources Control Board prepared pioneering Guidelines for ET Systems (Ingham, 1980). They maintained that ET/ETA systems should be considered only after determining that the site is unacceptable for sub-surface soil percolation alone. A typical Californian ET System cross section is in Figure 6.1. (Ingham, 1980)

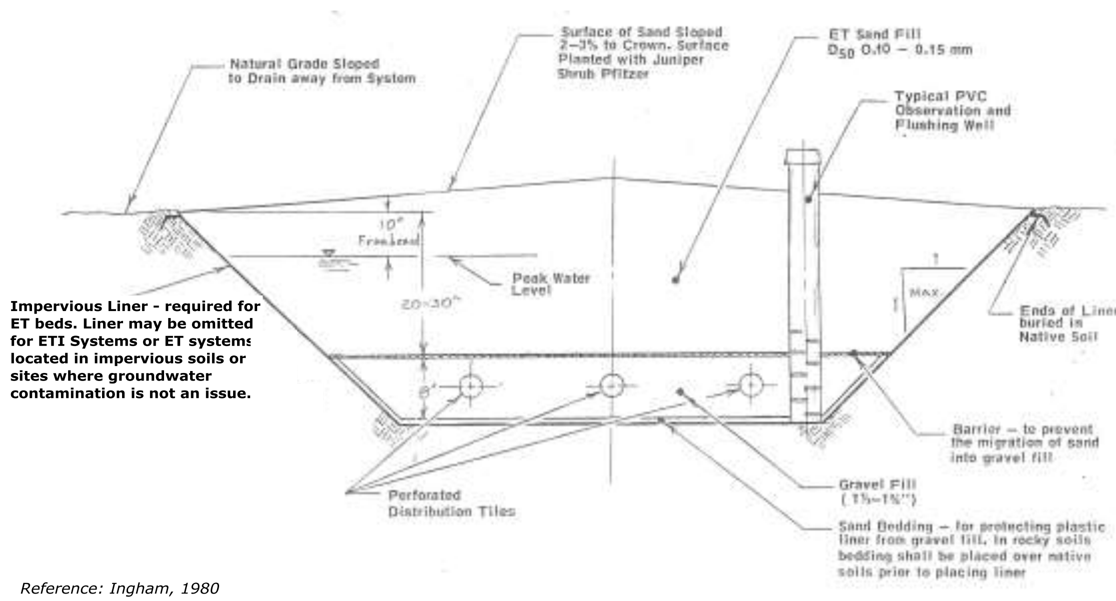


Figure 6.1: Section of ET System (NTS)

The main features of the Californian guidelines include:

1. The guidelines were based on criteria from the County of San Diego (San Diego, 1979) and research findings from the University of Colorado (Bennett, et al, 1978)
2. All ET systems in California (at the time) should be considered as experimental and should be monitored by local and state regulatory agencies
3. Exposure to sun and wind is desirable since it will enhance the transfer of effluent and soil moisture to the atmosphere
4. Surface runoff should be diverted from the ET system
5. The design should assume that 100% of the incident rainfall will soak into it
6. Every effort should be made to reduce water usage
7. The design ET rate shall be equivalent to the minimum monthly winter Class A pan evaporation rate
8. There should be a minimum depth of 1.0 m from the invert of the ET system to the highest known groundwater level
9. A 100% reserve area should be allocated
10. ET bed sand should be clean, uniform and a size range of 0.10 to 0.15 mm

11. The ET bed sand depth should be 500 to 750 mm
12. The ET bed should be crowned with a 2-3% slope from the centre
13. The gravel layer should be 30 to 40 mm diameter with a depth of no more than 200 mm
14. All ET/ETA systems must be constructed with a level base
15. Salt accumulation could be a problem in TET systems.

Since these were probably foundation ET guidelines in the USA they are worthy of comment in terms of the design criteria and based on the writer's findings how they compare today.

It was a plus for an experimental system to require monitoring. Very few of the earlier effluent disposal technologies during the 1960s and 1970s had this requirement. System performance capability can be determined from monitoring.

To assume that 100% of incident rainfall will soak into the ET system is conservative. This aspect is discussed later in this Portfolio. The writer feels that the grass cover and the upper soil cover will cause incident rainfall to run off if it has a crossfall. A surface crossfall of 2 to 3% from the centre is helpful but the writer tends to allow a steeper crossfall, as discussed later in this Portfolio.

The writer concurs that TET systems, since they are sealed or are based on slowly permeable soils, could experience salt accumulation problems. This will be less of a problem in areas with a well distributed rainfall, as in parts of northern New Zealand. Some soil salinity monitoring could be undertaken to confirm this problem. The solution in this instance is to periodically flush the ET system with fresh water.

In summary, the Californian State ET guidelines were soundly based and in the writer's opinion, as expected, some aspects tended to be conservative. He has considered the design components in these guidelines for the development of the model ET system for Queensland.

1.5 Advantages of ET Systems

Advantages of ET systems include (Trotta and Ramsey, 2000):

1. They can overcome site and soil constraints that prevent the use of conventional disposal trenches and beds
2. The risk of groundwater contamination is minimised if they are founded in very slow permeability soils or a liner is used
3. Reductions in horizontal and vertical setbacks are permitted
4. There is no need for a reserve area which is in conflict with the Californian State guidelines by Ingham (1980). The writer tends to

allow for a reserve ET trench system if the soil permeability is lower, for example, <0.15 m/d

5. They can be used for seasonal applications, especially for summer homes or recreational parks in areas with high evaporation and transpiration rates.

2 Research by Simpson and Associated Persons

This Section covers the following research topics and case studies:

1. Research based on Bernhart (1973) in Northland/Auckland, NZ
2. A schedule of selected trees and shrubs for ET systems in Northland/Auckland, NZ (Fielding 1977)
3. ETI projects in Whangarei Heads, Tikipunga Rugby Club and Whangarei City, Northland, NZ
4. Updated ETA work in Northland, NZ
5. Climatic similarity and differences and ETA work in North American, Australian and New Zealand
6. Suitable ET: rainfall ratios
7. TET systems
8. Retrofitting an ETA system, Morayfield, Queensland
9. Model ETA design for Queensland. It is to be noted that the work in Queensland does not include the wet tropics, due to its high rainfall.

It is to be noted that in the case of the project case studies no performance monitoring was undertaken. This was because either monitoring was not a specific requirement or funding was difficult to procure. Performance, particularly during low ET and persistent rainfall periods, was based on the fact that the TET or ETA system did not leak or overflow.

2.1 ET Research by Simpson Based on Bernhart (1973)

ET disposal by specifically designed mounds, beds, and trenches had been recognised in the early 1970s in Canada (Bernhart, 1973) and North America. The writer reviewed Bernhart's research findings extensively, at times in conjunction with Ian W Gunn, (Senior Lecturer in Civil Engineering, University of Auckland) and Alan Fielding, **(**environmental horticultural consultant and landscape architect, Whangarei, New Zealand) during 1975-1976, since it had potential in the Auckland and Northlands areas of New Zealand.

ET systems were considered to be applicable in areas with poorer quality and low permeability soils and higher ET rates. Particular highlights of Bernhart's research that were recognised included:

1. The function of capillary rise in the upper soils and fine media in ET systems to enhance ET
2. The selection of specific trench media and sand to enhance capillary rise. Very fine sand (0.05 mm) could lead to blocking problems whereas larger sized media (> 3 mm) could inhibit capillary rise action
3. The importance of maintaining aerobic environments within the mounds, trenches and beds. A rule of thumb was that aerobic systems required about 50% of the disposal area, when compared with anaerobic systems. This is due to increased temperatures as a result of aerobic activity, and the action of micro and macro organisms.

Based on the experience in New Zealand by Ian W Gunn, University of Auckland (pers. comm.) and the writer's experience since 1976, Bernhart's design approaches have been simplified for household ET systems as listed:

1. Pre-treatment in 3,900 L septic tank, preferably with double compartments
2. Dosed or pumped effluent loading
3. Diverted surface water and shallow sub-surface water from the ET system
4. Design of the disposal area based on an uptake rate of 10 mm/day, a conservative rate and 20 mm/day least conservative rate
5. Lower permeability soils to be used, less than 0.2 m/d
6. System to be sited to take maximum advantage of exposure to sun and wind
7. Use of two ET trenches or beds
8. Distribution lines to be vented to encourage aerobic conditions
9. Reserve area for future extension allowed
10. ET systems to be well raised and grassed to shed rainfall runoff
11. Selected shrubs to be established on the down slope side or over the ET system surfaces
12. Well crowned 1,200 mm wide trenches and 1,800 mm wide beds are preferred in New Zealand.

2.2 Northland/Auckland and Other New Zealand Based Research

Following on from the work of Bernhart (1973), the writer undertook a research program in conjunction with Alan Fielding; a New Zealand specialist in rare plants species. After a short period, it became very clear that the ET method of effluent disposal was a viable concept in milder temperate areas of parts of Northland and

Auckland for the following reasons (Simpson, 1978):

1. Under favourable conditions, some plants endemic to the region are capable of removing large amounts of soil water into the atmosphere
2. Vegetation generally has a quicker growth rate and a longer growing season in Auckland and Northland
3. Some types of endemic grasses, shrubs and trees respond to the uptake of additional moisture and the nutrients and mineral salts present in wastewater
4. Many of the more common species of native shrubs and trees were also suitable for ET disposal
5. Many of New Zealand's camping and resort areas are coastal areas where the peak occupancy is late spring, summer and early autumn. During this period the growing season is at its peak and ET rates are at their highest and the climatic and soil conditions are more favourable for effluent disposal.

The writer gained design and operational experience with a number of ET systems in Northland, New Zealand. Some of this work has been updated, based on current data. To examine the contrasts due to climatic differences, the writer also compared the Northland experience with cases in Western Australia and South East Queensland.

2.3 Selected ET Grass, Shrub and Tree Species – Northland/Auckland

Certain plants are responsible for the removal of large amounts of soil water into the atmosphere. The availability of soil water affects the rate of transpiration as does the atmospheric temperature, sunlight intensity, relative humidity, and wind velocity. Assuming that there is no limitation of soil water, with good climatic conditions, the maximum transpiration flow will depend on (Fielding, 1977):

1. Extent of vegetative cover
2. Good quality soil
3. Degree of active growth
4. Plant leaf area and density of leaf stomata
5. Ability of the leaf stomata to close
6. Extent and depth of root mass
7. Absorptive ability of the roots.

Based on the horticultural work of Fielding (1977), assisted by the writer and Ian W Gunn, University of Auckland, a list of plant species suitable for northern New Zealand (Northland, Auckland and Bay of Plenty) was developed. They were mostly evergreen, had a wide tolerance of soil water levels, require only a medium quality soil for good growth, had effective root systems for absorption and had a moderate to very high leaf density and area. This list of selected species suitable for ET systems was considered to be unique at the time, due to the joint effort and the detail of the assessment. For this reason it has been included in the text of this thesis as Table 6.1. A key to the codes given in the table is provided below.

Key:

1 = height around <1.0 m

2 = height around <2.0 m

3 = height around >3.0 m

A = very hardy (for Northland, Auckland or Bay of Plenty areas, with light and infrequent frosts and moderate rainfall)

B = moderately hardy

C = rather soft

D = deciduous

G = ground cover or grass

Table 6.1: List of Vegetation Species Suitable for ET Systems (Fielding, 1977)

Botanical Name	Common Name	Height	Hardiness	Deciduous	Ground cover/grass
Agapanthus spp.	African Lily	1	B		
Agrostis stolonifera	Creeping Bent	1	B		G
Alopecurus Pratensis	Meadow Foxtail	1	B		G
Aralias pp.	Angelica tree	3	B	D	
Aristotelia serrata	Maomako/wineberry	3	A		
Arrhenatherum elatius	Tall Oat Grass	1	A		G
Arthropodium cirrhatum	Renga/Rock Lily	1	B		
Azalea spp.	Azalea	1-3	B	Some D	

Bambusa spp.	Bamboo	1-3	B		
Brachyglottis repanda	Rangiora	3	A		
Bromus catharticus	Prairie Grass	1	B		G
Bromus mollis	Goose Grass	1	A		G
Cannas pp.	Indian Shoot	1	B		
Clianthus puniceus	Kaka beak	2	B		
Colocasia spp.	Taro	1	B		
Coprosma australis	Raurekau	*3*	A		
Coprosma luida	Karamu	*3*	A		
Coprosma propiqua		3	A		
Coprosma robusta	Karamu	3	A		
Cordyline australis	Ti/Cabbage Tree	3	A		
Cortaderia spp.	Toetoe/Pampas Grass	2-3	A		
Corynocarpus laevigatus	Karaka	3	A		
Cynosurus cristatus	Crested Dogs Tail Grass	1	A		G
Datura spp.	Trumpet Flower	1-3	A		G
Dichondra	Mercury Bay Grass	1	B		G
Dicksonia and Cyathea spp.	Tree Ferns	2-3	B		
Dysoxylum spectibile	Kohekohe	3	B		
Elaeocarpus dentatus	Hinau	3	A		
Elaeocarpus hookerianus	Pokaka	3	A		
Eragrostis elongata	Bay-Grass	1	B		G

Fatsia japonica		3	B		
Festuca spp.	Fescues	1	A		G
Fuchia spp.	Fuchia	1-3	B		
Geniostoma ligusttrifolium	Hangehange	3	A		
Geranium spp.	Geranium	1	A		Some G
Glyceria spp.	Sweet Grasses	1	A		G
Gunnera spp.	Prickly Rhubarb	1-2	B		Some G
Hedychium spp.	Butter fly Lily/Ginger	1	B		
Heimerliodendron brunonianum	Parapara	3	C		
Hibiscus spp.	Hibiscus	1-3	B		
Hoheria spp.	Lacebark	3	A		
Holcus spp.	Creeping Fog	1	A		G
Hosta spp.	Funkia/Plantain Lily	1	A		G
Hydragea spp.	Hydrangia	1-3	A	D	
Iris spp.	Iris	1	A		
Laurelia novae	Pukatea	3	A		
Lobelia spp.	Lobelia	1	B		G
Lolium perenne	Perennial Ryegrass	1	A		G
Lophomyrtus bullata	Ramarama	3	A		
Macropiper excelsum	Kawakawa	3	A		
Meryta sinclairi	Puka	3	B		
Neopanax arboreum	Five-Finger	3	A		
Paspallum dilatatum	Paspalum	1	A		G

Pelargonium spp.	Geranium	1	B		
Pennisetum clandestinium	Kikuyu Grass	1	B		G
Phleum pratense	Timothy grass	1	B		G
Philodendron spp.		1-3	C		
Phormium tenax	Flax	2	A		
Pittosporium spp.	Kohuhu	3	A		
Poa annua	Annual Meadow Grass	1	A		G
Podocarpus dacrydioides	Kahikatea	3	A		
Primula spp.	Primula	1	B		G
Prunus	Cherry	3	A	D	
Quercus spp.	Oaks	3	A	Mostly D	
Rheum spp.	Rhubarb	1-2	A		
Rhopalostylis sapida	Nikau Palm	3	A		
Ricinus communis	Castor Oil Plant	2	B		
Salix spp.	Willows	3	A	D	
Senecio petasites	Cineraria	2	B		
Solanum aviculare	Poroporo	3	A		
Trifolium spp.	Yellow Clovers	1	A		G
Ulmus spp.	Elms	3	A	D	
Viburnum spp.					
Vitex lucens	Puripuri	3	A		
Zantedeschia aethiopica	Arum Lily	1	A		

The plant species list was published in a New Zealand manual by Gunn (1994). The process of selecting suitable species required careful consideration of the following factors:

1. All climatic factors of the region
2. Micro-climatic factors
3. Plants known to be appropriate to the area
4. Effective plant species
5. Topography
6. Soil type, quality and depth
7. Aesthetic factors
8. Scent, which could be desirable.

In the mid-1970s there was a lack of ET information. A desk top study by Fielding (1977) sourced the performance of the listed five tree species, all deciduous. The ET information is per tree.

American Elm *(Ulmus fulva)*	130 mL /24 hours
Black Oak *(Quercus velutina)*	135 mL/24 hours
Cherry *(Primus avium)*	91 mL/24 hours
Tulip tree *(Linodendron tulipifera)*	43.5 mL/24 hours
Maple *(Acer sp.)*	8 mL /24 hours

The above listed transpiration flow rates were based on the following:

1. Light - taken under relatively high intensity
2. Night flows estimated at about 5% of maximum
3. Temperature range of 20 to 30 degrees C.
4. Relative humidity range of 64 to 96%
5. Soil water – maximum.
6. Tree species about 1.0 m high and bushy
7. Typical leaf area about 2.2 m^2

This information was sourced in the absence of similar findings. Note that the low transpiration amounts were applied to only 1.0 m high tree species. Larger tree species would yield higher amounts of moisture. The writer prefers shrubs that are at least 2.0-2.5 m high at maturity. Note also that transpiration still takes place at night, to a lesser extent.

More work by Fielding (1977) was undertaken on the transpiration rates of some deciduous species. The following aspects were included:

1. Evergreen species were more suitable for ET systems
2. Evergreen transpiration rates were generally higher than deciduous species
3. Flow rates could be expected to increase considerably for larger trees, as mentioned above

Some of the above ET information was considered in the development of the model ET design for Queensland.

2.4 ETA Projects in Northland, New Zealand

2.4.1 ETA Disposal – Golden Bay Cement Co, Whangarei Heads

Aspects of ET that the writer identified during a study of a land development project, owned by the *Golden Bay Cement Company*, Whangarei Heads, New Zealand, are as follows (Simpson, 1978):

1. Water balance determinations are required for the design of larger than individual home ETA systems. The main concern being that there is sufficient storage for effluent and rainfall during wet and cold weather conditions. Winter and early spring is usually wet
2. At the time, climatic data over a reasonable period was difficult to obtain in some parts of New Zealand. The writer was able to extrapolate data, from nearby stations, as supplied by Government Departments. He had local knowledge of weather patterns and micro-climates in Northland and was able to apply this to the data extrapolated
3. This residential development was to be largely composed of holiday type lots with about 10% intended for full-time residential. Typically, Northland holiday patterns are restricted to Christmas/New Year, Easter, School holidays (May and August), Queens Birthday (June) and Labour weekend (October).

Graphs of rainfall, ET and sunshine hours are plotted against estimated holiday influxes in Figure 6.2. From Figure 6.2 it can be seen that, during holiday influxes with the exception of the May and August school holidays, the other holiday periods coincide with more favourable weather. This includes low rainfall, high sunshine hours and high ET rates. This confirmed the cross-year suitability of ETA effluent disposal systems for Whangarei Heads, Northland, New Zealand.

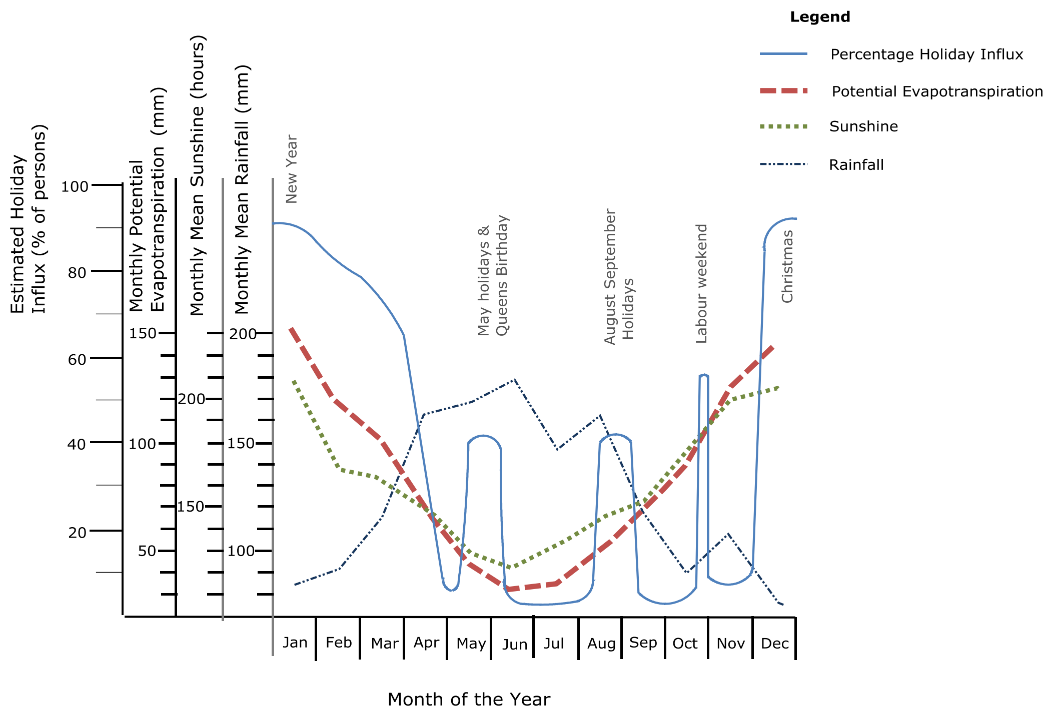


Figure 6.2: Plots of Whangarei Heads Rainfall/ET/Sunshine Data

2.4.2 ETA Disposal Bed System – Tikipunga, Northland Case Study

The writer undertook a case study for the former Whangarei City Council reserve (Rugby Club) toilet facility and presented this to a seminar at the University of Auckland (Simpson, 1981). Site and design parameters included the following:

1. Wastewater treatment - in an 18,000 L double chambered septic tank
2. Soil profile – loam on yellow clay, into basalt
3. Groundwater level – about 6 m from the surface
4. Field soil permeability value (by percolation test and K calculation) was 7 cm/day
5. Effluent loading was 150 persons at 60 L/head
6. Peak loading factor was 1.5.

The design ET for this project considered the following aspects:

1. Eight years of open pan evaporation records by former Whangarei City Council – selecting the lowest average daily evaporation for the lowest winter month. In Northland, New Zealand experience, the open pan evaporation figures compare well with potential evaporation (Penman) figures

2. Average daily lowest winter evaporation – 0.79 mm of ET times a factor of 3 (Bernhart, 1973) for dense evergreen planting and the "clothes - line" effect. The increased ET factor is discussed further in reflections. Design ET rate – 3.1 mm/d. The effluent disposal bed required, based on Bernhart (1973), was 460 m^2 which amounts to about 115 m^2 per EP.

Other ETA bed physical requirements included:

1. A cross-fall on the bed surface for some rainfall runoff
2. Some bed storage for adverse weather and site conditions
3. Water balance determination over one year (considering the problem with loading was seasonal and not being constant). The ETA facility catered for a rugby clubroom which was used during the rugby season, typically from April to September, and when the clubrooms were hired for private functions.

After seven months of operation, under variable load and weather conditions, the writer made the following observations:

1. The surface water cut off drain was effective
2. Final shaping of the bed surface was important, to avoid localised ponding areas
3. The bed became hydraulically overloaded during prolonged wet weather however no adverse effects such as effluent overflow were noted
4. Quick isolated surface ponding recovery took place very soon after heavy rainfall had ceased
5. No odours were experienced
6. Most of the vegetation had established well
7. Initial staking of taller trees was necessary, since the site was exposed to wind
8. Venting of the bed system, to encourage aerobic conditions, was considered useful.

The writer undertook project inspections were undertaken at least two weekly but he focused on checking the site after or during rainfall events.

2.4.3 ETA Disposal Trenches Case Study – Ashford, Whangarei City.

This was a case study that the writer undertook for domestic ETA trenching in Whangarei City and presented at a University of Auckland seminar (Simpson, 1981). Site and design aspects for this project included:

1. Wastewater treatment - using a 2,700 L septic tank with double chambers

2. Soil profile – red/brown loam (relatively free draining) into rock.
3. Calculated soil K of 40 cm/day
4. Design infiltration rate of 265 L/m^2/day
5. Peak flow was not determined
6. ET losses were considered as a factor of safety. The design was based on soil infiltration only. The assumption was that ET uptake would take place when site/soil conditions and weather conditions were unfavourable.

ET/infiltration trenching features included:

1. Two 8.0 m long, 600 mm deep, 900 mm wide trenches to suit the site contours
2. Due to shallow rock, the side walls of the trench were considered as the only disposal surface
3. A distribution box for effluent dosing and resting, to encourage aerobic conditions
4. 250 mm depth of site topsoil, planted in selected vegetation
5. Surface water diversion drain
6. Planting blended in with site landscaping

The system was completed and operational by August 1978. No problems were reported by the owner following commissioning.

2.4.4 Early ET Experience – Northland

The writer examined average annual ET and rainfall for five stations along the east coast in Northland, New Zealand supplied to him in 1977. Based on project and local experience, the writer considered that the ratio of ET: rainfall would vary, due mainly to changes in coastal rainfall patterns. Annual ET and rainfall figures for some stations along the east coast of Northland are listed in Table 6.2.

Table 6.2: Early Potential Evapotranspiration: Rainfall Ratio Determinations - Northland, NZ

Stations	Average Annual Penman Potential Evapotranspiration (mm) (1)	Average Annual Rainfall (mm) (2)	Evapotranspiration: Rainfall ratio
Kaitaia Aerodrome	1051	1,332	0.79

Puketurua	829	1,314	0.63
Kerikeri	907	1,557	0.58
Whangarei Airport	1038	1,572	0.66
Leigh	1066	1,133	0.94

Notes:

1. Sourced from N Z Meteorological Service, 1977
2. Since no average annual rainfall figures were supplied by the NZ Meteorological Service in 1977 the writer has sourced rainfall data recently from National Institute of Water and Atmospheric Research Ltd, NZ (NIWA), July 2010.

From this mix in time of ET and rainfall data the following observations from Table 6.2 can be made:

1. Kerikeri (Bay of Islands) has the least favourable ET: rainfall ratio of 0.58. This situation demands more detailed design and some allowance for storage within the system and an overflow facility
2. Leigh has the most favourable ET: rainfall ratio of 0.94 - when the average annual ET is just less than the average annual rainfall so there will be some reliance on soil infiltration
3. Most of the writer's ET design work, in the mid to late 1970s and early 1980s was undertaken in the Kerikeri and Whangarei areas, which have less favourable ET: rainfall ratios. However, each system was not sealed and the writer took into account the available soil permeability, as a factor of safety.

Around the 1970s the New Zealand Meteorological Service made broad scale evaluations of the evaporation climate of the country, and to this end it calculated potential ET values according to Penman, Thornthwaite and Priestley-Taylor formulae (N Z Met. Service, 1977). The ET results refer to a uniform short vegetation cover (pasture, in generally flat or low relief terrain). The New Zealand Meteorological Service had not attempted to apply theoretical approaches to formulate the effects of different vegetation, slopes, degrees of exposure and it had not undertaken experimental work in that area.

In the writer's opinion, actual ET within areas with taller grasses, shrubs and trees can be expected to increase due to more vegetation exposure and the "clothes – line" effect. In general, tree cover can be expected to increase ET somewhat, partially because of greater absorption of solar radiation and greater aerodynamic roughness. This effect is probably more significant in isolated patches of trees or bushes than for extensive cover (NZ Met. Service, 1977).

2.4.5 Updated ET Experience – Northland

The writer considered that the average annual ET and rainfall data should be updated. More recent average annual and potential ET, using Penman and Priestley-Taylor determinations, for four weather stations along the east coast of Northland have been listed in Table 6.3.

Table 6.3: Updated Potential ET: Rainfall Ratios – Northland, NZ

Station	Average Annual Rainfall (mm)	Penman Potential ET (mm)	Penman Potential PE: rainfall	Priestley - Taylor Potential ET (mm)	Priestley - Taylor Potential ET : rainfall
Kaitaia Aerodrome	1,338	1,048	0.78	911	0.68
Kerikeri	1,614	869	0.54	859	0.53
Whangarei Airport	1,405	1,012	0.72	861	0.61
Leigh 2	985	1,129	1.15	885	0.90

The following observations from Table 6.3 can be made:

1. The average annual ET: rainfall ratios, from Table 6.5 and Table 6.6 did not vary to any great degree. Some variations could be expected since the 1977 data was over a longer period, whereas the 2010 data were over the period 2006-2009
2. Comparing the potential ET figures, Priestley-Taylor is consistently lower than Penman
3. The Kerikeri station gives virtually the same potential ET:rainfall ratio for both methods since the two estimates are almost identical, whereas the ratios for the other three stations vary more widely
4. When considering all the stations, the ratios for the Leigh 2 station were the highest. The Penman ratio is higher and the Priestley-Taylor ratio is similar to the ratio obtained earlier.

2.4.6 Application of Climatic Similarity to ET Design – Transferability to Australasian Climates

Over the period 1976 to 1980 the writer considered climatic similarity when designing ET systems in Northland/Auckland, New Zealand, based on data from Bernhart (1973).

It seemed reasonable to select to transfer this approach to a range of Australasian climates.

The purpose of this sub-section is to demonstrate the contrasting climatic differences between Northland, South East Queensland and Western Australia. AS/NZS 1547:2000 gives design parameters which are intended to cover New Zealand and Australia. It is not feasible to have standard design ET rates for Australasia.

The writer has established that earlier and updated more recent ET: rainfall ratios in the temperate climate of Northland, New Zealand showed little variation in time.

Pan evaporation and rainfall information for the arid west climate of Western Australia had been reported by Anda et al (1991). From these data pan evaporation: rainfall ratios were calculated for each site (see Table 6.7).

Since two different contrasting climatic areas in Australasia had been reviewed; Northland (temperate) and Pilbara, Western Australia (dry tropics), the writer decided to include South East Queensland (sub-tropical) for comparative purposes. This comparison of three different climatic areas provided an opportunity to draw conclusions and assess the risks of installing ET systems in these three varying climatic areas. Refer to Table 6.4 for the comparison of these three climatic areas and comments on risk assessment of installing ET systems in these areas.

Table 6.4: Comparison and Risk Assessment Comments on ET: Rainfall Ratios in Australasia

Locations	Pan or Potential ET:Rainfall Ratios	Comments on Risk Assessment
Northland, NZ	0.53 to 0.90	ET: rainfall within the lower to middle range will require specific design otherwise there is a risk of system overflow.
Pilbara, Western Australia	4.4 to 14.3	The very high ET: rainfall ratios show that strong ET action will take place. The risks in this situation are that it could dewater the ET trench/bed and plant die-off takes place, particularly during periods of no flow.
SE Queensland	0.89 to 2.4	ET: rainfall in this range requires site specific design. For example, a ratio of 0.89 should require storage and overflow provision. There should be no apparent risks associated with a ratio of 2.4 since the ET action will be strong.

When applied to Queensland, with the exception of the wet tropics, for a larger scale project, it is advisable to determine the ET: rainfall ratio. As discussed earlier an ET: rainfall ratio of 1.2 is considered to be suitable for the model Queensland/Northern NSW ET design.

The pan evaporation: rainfall ratios for several projects in Western Australia have been presented in Table 6.5 (Anda et al, 1991).

Table 6.5: Pan Evaporation: Rainfall Ratios – Western Australia

Location	Rainfall (mm)	Pan Evaporation (mm)	Pan Evaporation: rainfall ratios
Ningamia	260	2,600	10
Kawarre	680	3,000	4.4
Mardiwah Loop	530	3,200	6
Piawadjari	210	3,000	14.3
Wurrenranginy	680	3,000	4.4
Tjuntjuntjarra	190	2,400	12.6
Wongatha Wonganarra	234	3,500	14.9

It is understood that pan evaporation figures, which are normally readily available, are slightly lower than ET rates, largely due to the lack of vegetation influence.

To complete the comparison of three different climatic areas, rainfall and ET data for four stations in South East Queensland, sourced from the Bureau of Meteorology Queensland Climate Services Centre, are summarised in Table 6.6.

Table 6.6: Average Annual Evapotranspiration and Rainfall Ratios – SE Queensland

Station	Average Annual Rainfall (mm)	Average Annual Evapotranspiration (mm)	ET: rainfall Ratios
Sunshine Coast Airport	1,510	1,494	0.98
Beerburrum Forest	1,448	1,287	0.89
Brisbane Airport	919	1,672	1.77
Gatton	674	1,613	2.4
Coolangatta Airport	1,533	1,520	0.99

Comments on Table 6.6 are as follows:

1. High evapotranspiration (ET) has been recorded at the Brisbane Airport and Gatton
2. The Sunshine Coast Airport ET compares very well with the Coolangatta Airport
3. Beerburrum Forest has the lowest ET
4. The most favourable ET: rainfall ratio is at Gatton, which is closely followed by the Brisbane Airport
5. From this range of ET: rainfall ratios the writer has ‘selected a ratio of 1.2 since the ET exceeds rainfall, which can be applied to the model ET system design for Queensland.

2.5 Total ET (TET) Systems

There are situations whereby land area is restricted, soils have low permeabilities, water tables are shallow and bedrock can be a constraint. In the writers experience in Northland/Auckland, New Zealand and Queensland the following factors must be applied or cautions taken:

1. Climatic data reviewed carefully. With TET systems in particular, the existence of a micro-climate in the area must be checked for, which could be of some design assistance
2. A water balance is undertaken in these more sensitive situations and for larger projects. In the case of TET, the writer considers that a water balance is a good design aid

3. Generally speaking, evergreen plants with the best known ability to transpire must be sourced (Alan Fielding, pers. comm. June 2010)
4. These plants must be suitable for the local likely climatic conditions
5. It is preferable that the trench or bed is installed and the plants are established, before the effluent is applied. This is more important for TET systems where soil infiltration does not take place
6. Depending on the effluent loading, effluent standard and climatic conditions, a factor of safety should be applied when determining the area. This factor of safety, which is probably more applicable to TET systems, may vary from 1.2 to 1.45
7. The assumption was that ET uptake would take place when site/soil conditions and weather conditions were unfavourable.

2.6 Retrofitted ETA Trenches – Case Study by Simpson, Morayfield, Queensland

This is a case study of standard infiltration trenches being converted into an ETA effluent disposal system. The writer purchased a 3000 m^2 rural/residential property in Morayfield in 2002. Greywater is collected and irrigated separately. Blackwater is directed to two shallow infiltration trenches. The blackwater disposal area was near level and surface runoff is diverted via a planted stone channel (constructed by the writer). Conventional infiltration trenches were installed for the blackwater in about 1984. The trenches are 10 m long, 600 mm wide and 450 mm deep.

To assess the capability of the existing trenches (without selected planting) a soil profile was undertaken on 1 November 2010. The soil profile consisted of an upper clay loam for 300 mm depth, into silty clay. A constant head percolation test was undertaken. Using the percolation test results the calculated soil K value was 0.05 m/d. This is a low figure for such a profile since the soil was saturated. Some 327 mm of rainfall had been recorded for October 2010 in the property rain gauge which was 4.1 times the long term monthly average.

The expected effective life for conventional infiltration trenches in clay loam and silty loam is 15-20 years, as estimated by drainage inspectors from the Moreton Bay Regional Council. Since the original trenches were installed in 1984, to enhance the effective life of the trenching, the writer converted them some years ago into an ETA system by:

1. Establishing selected shrubs largely around the perimeter, to enhance effluent uptake by ET and facilitate nutrient uptake
2. Raising and mounding the surface with sand and loam, to shed off incident rainfall.

The layout of the trenching system and the position of the selected shrubs are shown in Figure 6.3. A typical section of the Simpson retrofitted ETA system is in Figure 6.4.

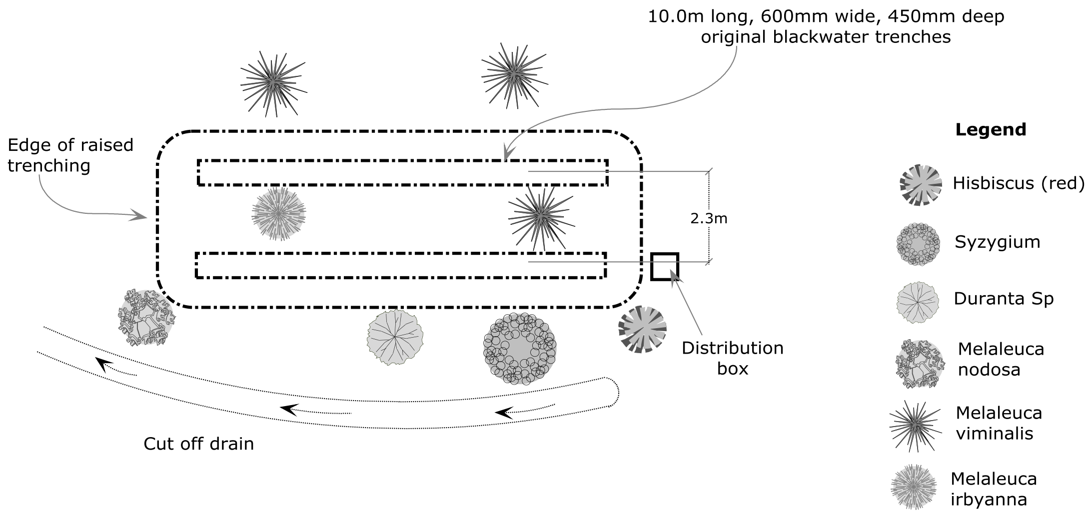
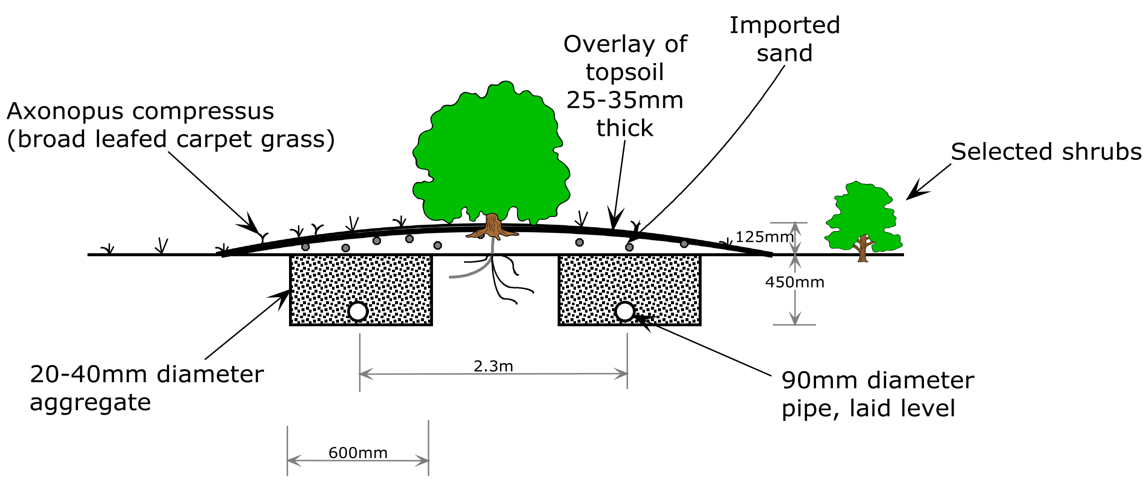


Figure 6.3: Layout of Simpson Retrofitted ET/Infiltration System (NTS)



Note: Raised sand & topsoil surface installed to enhance capillary rise of effluent and shed off some incident rainfall.

Figure 6.4: Typical Section of Simpson Retrofitted ET/Adsorption System

The estimated daily flow (based on 2.5 persons at 75 L per day) is 175 L which is based on standard water saving devices and meter readings. Considering 2 trenches at 10 m long each, the soil K value of 0.05 m/d and an assessed DLR of 5 mm/d, the length of trenching required is 18 m (as per AS/NZS 1547:2000). It is doubtful that the original trenches without selected planting would effectively store and dispose effluent, particularly during peak field and weather conditions. The selected shrubs and grass cover established, identified with the assistance of Leiper et al (2008), are listed in Table 6.7.

The modified ETA system has worked well for at least the past 6 years, since the vegetation has become more established. The system has not overflowed. A recent test of the system occurred with a rainfall event over the period from 8 to 16 October 2010 whereby 292 mm of rainfall was recorded by the rain gauge on the site.

Table 6.7: Selected Shrubs and Grass Cover – Simpson Retrofitted ETA Trench System.

Common Name	Botanical Name	Comments
Duranta	-	Also within greywater irrigation area. Height 2.5m
Hibiscus (red)	-	Also within greywater irrigation area. Height 2.5m
Prickly – leaved paper bark	*Melaleuca nodosa*	Matured to a height of 2.2 m
Powder puff Lilly Pilly	*Syzygium moorei*	Matured to a height of 3.5m
Bottlebrush – two (1)	*Melaleuca viminalis*	Height of 2.2m
Swamp tea tree	*Melaleuca irbyania*	Height of 2.3m
Broad-leaved carpet grass	*Axonopus compressus*	Provides a very good cover and a good transpirer

Note:

1. Bottlebrush or Callistemon are now known as *Melaleuca spp*. (Leiper et al, 2008)

This retrofitting of a standard effluent trench into an ETA system has value, particularly in low soil K situations (less than 0.08 m/d). The writer can confirm that all the above shrub species and grass cover are suitable for ETA effluent disposal, based on their consistent growth and survival.

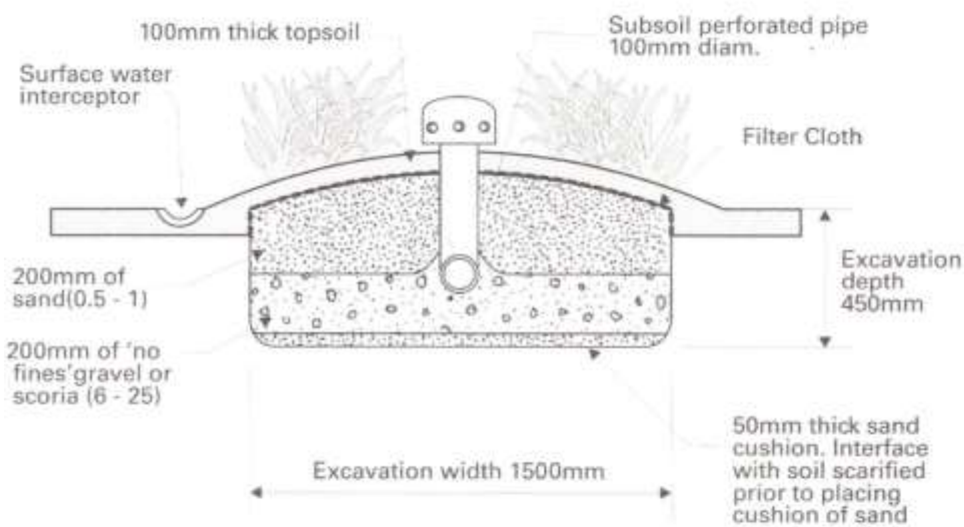
2.7 Model ETA Trench Design for Queensland/Northern NSW

ETA trenches are covered in AS/NZS 1547:2000 but little rationale and detail is given.

It is to be noted that ETA trenches can be classified in two types:

1. Shallow raised trenches surface planted systems, as referred to in AS/NZS 1547:2000 and Bernhart (1973)
2. Raised trenches with peripheral planting of shrubs, as reported by Tanner and Bouma (1975). The Simpson retrofitted ETA trench system in Morayfield is this type.

By utilising the writer's past ET, ETA and TET experience in New Zealand and ongoing ETA experience in Australia, he has developed a model ETA trench with surface planting, suitable for application in Queensland (except the wet tropics) and Northern New South Wales. This model design has considered suitable vegetation species, enhancing capillary rise, the distribution of effluent, the effluent treatment standard, selection of surface slopes and the location and depth of the distribution pipes. A section of the model ETA trench, which is a modified version in AS/NZS 1547:2000, is given in Figure 6.5.



Source: AS/NZS 1547:2000

Figure 6.5: Section of Model ET/Absorption Trench

The features and components of the model ETA Design for Queensland are listed in Table 6.8 and these are compared and discussed in relation to other ET systems reported in this Portfolio. The rationales for the ET components selections for the model design are provided in Table 6.9.

Table 6.8: Model ETA Trench Features/Components

Feature or component	Nominated Details	References / Comments
Lining	No lining for ETI systems	
Gravel layer: Depth: Gravel Size:		 Depth – 200 mm (AS/NZS 1547:2000) Depth – 150 – 200 mm (Bernhart,1973) Depth – 150 mm (Patterson, 2002) Depth – 300 mm (EPA 635 and EPA TFS-31) Depth – 400-700 mm (Anda et al, 1991) Depth – 200-300 mm (Sampson, 1981) Depth – 200 mm (Ingham, 1980) Depth – 300 mm (Austin City,1995) Gravel size – 5 - 20 mm (Bernhart,1973) Gravel size – 6 - 25 mm Gravel size - < 20 mm (Patterson, 2002) Gravel Size – 20 – 65 mm (EPA 625 and EPA TFS-31)
Trench/bed Gradient	Level	Level (Ingham, 1980)
Location of Distribution Pipes	Top of gravel layer	Top of gravel layer (AS/NZS 1547:2000) Mid way within gravel layer (Ingham, 1980) Top of gravel layer (Sampson, 1981) Top of gravel layer (Austin City, 1995)
Trench/Bed: Type: Width:	 Trench system 1.2 m	 Trench and bed Width – 1.0m (Anda et al, 1991) 0.6 to 1.5 m (AS/NZS 1547:2000)
Capillary Rise Sand Layer:		

Depth:	250 mm depth	Depth – 500 – 700 mm (Ingham, 1980) Depth – 300 mm (Patterson, 2002) Depth – 600–700 mm (EPA TFS-31 and EPA 625) Depth – 150 mm (Salvato et al, 2003) Depth – 200 mm (AS/NZS 1547:2000) Depth – 300 mm (Sampson, 1981) Depth – 300-600 mm (Anda et al, 1991) Depth – 600-750 mm (Crites and Tchobanoglous, 1998) Depth – 450 mm (Bernhart, 1973)
Sand size:	0.1 – 0.5 mm sand	Size – 0.1 mm (Crites and Tchobanoglous, 1998) Size – 0.1– 0.15 mm (Ingham, 1980) Size – 0.1 mm (Patterson, 2002) Size – 0.1mm (EPA TFS-31 and EPA 625) Size – 0.1 mm (Salvato et al, 2003) Size – 0.5 -1.0 mm (AS/NZS 1547:2000) Size – 0.1 mm (Anda et al, 1991)
Soil Cover:		
Type:	Loam/sandy loam	Topsoil
Depth:		Depth - 100mm (AS/NZS 1547:2000) Depth – 150-300mm (Salvato et al, 2003) Depth – 50mm (Austin City, 1995) Depth – 100mm (min) (Gunn, 1989)
Planting System	Surface or peripheral	Surface planting (Anda et al, 1991) Surface planting (Ingham, 1980)
Surface Slope	5%	2-3% crossfall (Bernhart, 1973) 2-3% crossfall (Ingham, 1980) 13mm per 1.0m (Patterson, 2002) 60 mm per 1.0 m (used by the writer for retrofitted ETI

		system in Morayfield.) 1 in 12 slope (Salvato et al, 2003) 2 - 4% slope (Austin City, 1995)
Surface Runoff	50%	50% (Sampson, 1981)
Area Increase	15%	15-20 % (Patterson, 2002) 15-20% (Crites and Tchobanoglous, 1998)
Distribution System		2 trenches (min), alternating (EPA, 1980) 2 trenches (min), (Bernhart,1973) 2 trenches or beds (min) (Gunn,1989)
ET:rainfall ratio	1.2 or higher	Sourced from ET:rainfall table in this Portfolio

Table 6.9: Rationale for Selecting Model ETA Trench/Bed Features

Feature or component	Nominated Details	Rationale for Selection
Lining		Linings for TET systems only
Gravel layer	6-25 mm gravel 200mm depth	To allow some storage
Gradient	Level	To allow positive distribution of effluent
Location of distribution pipes	Top of gravel	To be closer to the surface to optimise capillary rise
Trench/bed	Trench	Trench system preferred
Capillary rise sand	0.5 mm sand 250 mm depth	Depth and sand size to optimize capillary rise
Soil cover	100 mm of loam or sandy loam	100 mm minimum
Planting system	Surface planting using grasses/rushes or peripheral planting using selected shrubs	Optional systems. On the surface planting should be restricted to native grasses/rushes/sedges or

		flowers such as Canna lilies and agapanthus, with smaller root systems. Peripheral planting can be selected shrubs with heights <2.5 m at maturity.
Surface slope	5%	To create a positive crossfall and allow for some settlement.
Surface Runoff	50 %	Design assumption
Area increase	15%	To allow some reserve storage to compensate for peak flows and rainfall variations.
Distribution system	Twin trenches	To allow intermittent effluent dosing and resting
ET:rainfall ratio	1.2	Sourced from ET:rainfall ratio table in this Portfolio

With the current AS/NZS 1547:2000 and its review it is unlikely that the Standards Committee would accept the writer's proposal of a model ET design. Councils will abide by the AS/NZS 1547:2000 and its review so the above exercise has been of academic interest only.

3 Literature Review on ET Systems

The topics covered in this Section include the following:

1. ET conditions and mechanisms
2. ET research by others – New Zealand based
3. USA based work on capillary rise
4. Australian ET experience
5. North American ET experience.

3.1 ET Conditions and Mechanisms

An understanding of ET requires expertise from several disciplines. When studying the meteorological aspects of ET systems, consideration must be given to the following:

1. Local soil/vegetation conditions
2. The availability of energy
3. The transfer of water vapour

4. Potential ET.

The mechanisms involved in ET are examined in terms of evaporation from the soil surface, evaporation of intercepted water and water loss by transpiration (Sharma, 1985).

Evapotranspiration is the net water loss caused by the evaporation of moisture from the soil surface and transpiration by vegetation. For continuous evaporation the following three conditions must be met (Trotta and Ramsey, 2000):

1. There is a latent heat requirement of approximately 590 cal/g of water evaporated at 15 degrees C
2. A vapour pressure gradient is needed between the evaporative surface and the atmosphere to remove vapour by diffusion, convection or a combination of the two
3. There must be a continuous supply of water to the evaporative surface.

Evapotranspiration depends on the supply of liquid at the evaporation surfaces and on the weather as heat (solar radiation) is required to change liquid into vapour. ET is also influenced by the vegetation over the disposal field. Trees and shrubs with a large silhouette catch more advected heat. This is known as the "clothes-line" effect (Tanner et al, 1975).

Advection can double the ET from a small irrigation area (230 m^2) when the surrounds are dry. This is the extreme for arid areas. In humid areas advection seldom exceeds 25% and it can be infrequent (Tanner et al, 1975).

By 1979, accurate methods were available for determining actual evapotranspiration at a given location. However, most methods could only be applied at sites where special instrumentation had been set up. Brutsaert and Stricker (1979) developed a novel and simple way of determining actual evapotranspiration. This method was:

1. Based on a conceptual model
2. Hypothetical non-advective evaporative power of the air used as a reference
3. Both excess and deficit provide an index of the aridity of the atmosphere and they are related to the regional advection, hence, the approach was known as the "advection-aridity" approach.

A comparison with data obtained about the same time during the drought year in Western Europe in 1976 produced good agreement, which was promising for wider application of the ET method.

ETI systems can fail if the ET contribution is over-estimated. Over-estimation can be overcome by placing the effluent distribution pipes higher in the trench or bed profile to enhance capillary rise. The writer feels that this is a logical approach and used this technique in New Zealand (1976-1982) and in Queensland over the past decade.

3.2 ET Research by Others – Northland/Auckland, NZ based

3.2.1 Lake Taupo, NZ - Experimental ET Case by Sampson (1981)

Water pollution problems have occurred along the margins of Lake Taupo as a result of continuing urban settlement. On-site treatment and disposal has been by septic tank and shallow infiltration trenching in the free draining pumice type soils, releasing organic matter and nutrients into the water resources. The Taupo County Council reviewed the ET work of Bernhart (1973). Other overseas and local research information indicated that less optimistic ET rates could be expected. Consequently, council decided to set up a research program by building a full scale system, servicing a county staff house, to provide information under local conditions (Sampson, 1981).

Three ET beds of 20 m^2 each were installed. A splitter box divided the flow into three equal streams; two streams were from the septic tank and the third stream from an aeration unit. Two of the beds were of identical construction. The third bed was deeper with an additional layer of fine compacted pumice within the gravel zone. The purpose of this additional layer was not described. A chart-recording rain gauge was installed near the centre of the site. Sampling of nitrogen, phosphorus and sodium was undertaken.

A water balance was carried out and this revealed the following:

1. The observed ET rates were 80% of those predicted by Bernhart (1973)
2. The observed ET rates were considerably higher than those calculated by the New Zealand Meteorological Office.

A sodium balance was carried out to check leakage from the beds, which was found not to be a problem. Nutrient mass balances showed that considerable amounts of nitrogen and phosphorus were being lost passing through the beds. The nitrogen loss was considered to be partly due to plant growth and partly due to de-nitrification.

The TET areal requirements for effluent from a typical house were high when compared with ETA requirements. A compromise suggestion was a partial ET system where the bed was not fully sealed to the surface. A sub-surface overflow could be provided to operate during times of stress (Sampson, 1981).

They also assumed a 50% surface runoff co-efficient. Runoff from the surfaces of ET system is a debatable issue. In the writer's opinion it depends on the crossfall and the surface cover. In his observations, during appreciable rainfall, a mulched surface gives a higher percentage runoff than a bare soil or grassed surface. He feels that 50% is a sound basis and has consequently adopted this figure for the model ET system for

Queensland and Northern New South Wales.

3.2.2 Northland, NZ - ET Survey

A survey of 20 ET systems in Northland, New Zealand was undertaken and reported by Crampton (1984). Those classed as being successful amounted to 13 while 7 were considered unsuccessful (as evidenced by leaking at the end of the winter). The systems were a mix of ETA and TET, with and without overflows. It was clear that the unsuccessful beds were poorly sited and not protected from groundwater and/or surface water influence. This survey highlights the importance of siting, with exposure to sun and wind, and having effective clearances from groundwater, as well as a surface runoff diversion system.

3.2.3 Leigh, NZ - ET System

The most successful large ETA system in New Zealand to date was reported by Gunn (1988). It is located at the University of Auckland's Leigh Marine Laboratory wastewater disposal system which was designed to cater for 35 people. The ETA system is comprised of 20 beds, 15 m long and 1.5 m wide which operate on a dosing cycle of one week operational and three weeks resting period. The interesting aspects of this ETA system are as follows:

1. It relies on the dual disposal mechanisms of ET and soil infiltration
2. It is both elevated and exposed, enhancing ET disposal
3. The beds are 1.5 m wide, which the writer feels is very suitable since it provides more surface area per metre run
4. The effluent dosing period of 1 week and the 3 week resting sequence helps to maintain the benefit of an aerobic soil environment.

3.2.4 Lincoln University, NZ - ET Experience

A non-discharging ET bed system was installed and studied at the Lincoln University farm in New Zealand (Balley and Dakers, 2004). This evaluated the actual ET rate and it was used to validate a water balance model. This sealed ET system was found to be questionable due to the fact that the precipitation rate exceeded the ET during critical winter months. However, two possibilities were suggested for future studies:

1. Design the system with the surface adequately crowned to shed larger amounts of incident rainfall
2. Design a bed with a removable transparent plastic roof to shed the total amount of rain water and act as a "green house".

This project demonstrates the need for suitable climatic conditions. Ideally, the ET should exceed the rainfall. The cold winter at Christchurch is not conducive to plant growth.

3.2.5 General New Zealand Experience

In New Zealand's wet climate one cannot avoid the disposal system being flooded at times. The consequential extra retention time provides nutrient uptake and bacterial die-off. The New Zealand Department of Scientific and Industrial Research (DSIR) investigations in the early 1990s showed very high nutrient removal was being achieved due to nitrification and anaerobic de-nitrification which protects ground water.

The northern New Zealand climate and growing season is far more favourable to the use of ET systems than the North American climate in which Bernhart (1973) carried out his work. The writer supports this statement based on the work of Ian W Gunn and the experience he has gained by several study trips to North America over the past 30 years.

Gunn (1989) made the following comments on ETA design for New Zealand:

1. Reasonable topsoil depth is required – 100 mm or more
2. Effluent dosing is to be evenly distributed over the total system coverage
3. A minimum of two systems to be installed
4. Allow a 50-100% of reserve area for future extensions
5. Bed and trench sand is not to be too fine or have a high percentage of fines
6. TET systems to be lined and provided with an overflow for peak loads and winter conditions.

As a result of an extensive study tour of the USA, Gunn (1986) found that ET systems were generally acceptable where the annual ET exceeded the annual rainfall by an amount which equated to the total annual effluent flow. The writer feels that this is probably the ideal scenario but it does not take into account the following:

1. The effect of micro-climates which can favour ET systems
2. The available soil infiltration.

Gunn (1990) presented an ET disposal paper at a symposium in Auckland. A range of alternative designs are now available to facilitate effective on-site effluent disposal using ET in conjunction with soil infiltration. The ETA systems were initially designed with a completely sealed bed and plastic liner. However, in New Zealand's rainy climates you cannot avoid the system filling up with liquid so TET systems were used, having 2 weeks detention in the pore spaces within the gravel. The following aspects were answered by Ian W Gunn at a discussion session following the paper presentation:

1. The TET systems with the 2 week detention facilitated nutrient uptake by the vegetation and bacterial die-off
2. Another option is to use 600-700 mm sand depth above the natural ground. Topsoil is place over the surface to assist ET disposal.

3.3 USA Based Work on Capillary Rise

USEPA (undated) mentions the process of capillary rise which is responsible for drawing effluent from an ET system allowing it to be evaporated or transpired into the atmosphere. This publication reports that some designs can be suspect because they store effluent so deep that the capillary rise action can be inhibited. It is also claimed that that some ETA systems have been less than impressive, for example, over estimating the ET potential of shrubs and trees and of the overall potential of ET.

Capillary rise is most important to the successful functioning of an ET system. Capillarity is the upward movement of water and effluent in the soil, into the vegetation root zone. Molecules within the liquid are attracted to each other and on a surface which creates surface tension. Water balances have greater affinity for a solid they come in contact with than for other molecules. The result is that liquid will climb up the surface of a solid to an extent dependent on the diameter of the soil particle pore space or tube. Salvato et al (2003) report typical capillary rises as listed in Table 6.10.

Table 6.10: Typical Capillary Rises for Different Particle Sizes

Soil Type	Particle Sizes	Typical Capillary Rises
Coarse sand	0.5 - 1.0 mm	12.5 cm
Medium sand	0.25 – 0.5 mm	25.0 cm
Fine sand	0.1 – 0.25 mm	40.0 cm
Silt	0.002 – 0.25 mm	100.0 cm

It can be seen from Table 6.10 that the capillary rise efficiency increases with finer sand or silt particle sizes.

The writer understands that ETA systems have generally been working well, but no scientific studies have been undertaken to verify this outcome. Many times the placement of the effluent distribution pipe higher in the trench and bed offsets the problem of poor capillary rise. The writer has tried this concept in conjunction with the careful selection of the capillary sand layer and the depth, resulting in improved functioning.

3.4 Australian ET Experience

3.4.1 Pilbara, WA - ET Experience

In some regions of the Pilbara, Western Australia, the soil is extremely impermeable, with high clay content. Conventional infiltration trenches frequently fail. ET disposal systems have a potential in these areas. The first prototype ET trenches were to be installed in several remote Aboriginal settlements in 1990 (Anda et al, 1991).

The ET systems are restricted to regions with low annual rainfall (700 mm) and high evaporation. The ET system option is seen as having the additional merits of:

1. Being childproof and safe
2. Relatively low cost
3. Capable of accepting largely fluctuating loads within the botanical tolerances of the species used (Alan Fielding, pers. comm. June 2010)
4. Minimal maintenance requirements.

The use of salt tolerant plant species is very important to prevent premature system failure arising from plant death. It must be appreciated however that in some areas many halophytic plants are rather poor transpirers, except under ideal conditions, including situations where water is abundant (Alan Fielding pers. comm. June 2010).

The Pilbara ET systems offer the unique experience of working in very high ET and low rainfall environment, probably unparalleled in Australia and New Zealand. The only problem the writer can foresee is when ET rates are at a peak, rainfall is zero or minimal and the wastewater loadings are zero or minimal, in which case the survival of the planting is critical.

3.4.2 Western Australia – Ongoing ET Experience

An introduction to the Western Australian ET experience has been presented in Sections 2.4.6 and 3.4.1. Continuing experience using ET for on-site effluent disposal for remote Aboriginal communities is reported by Anda, et al (1999). A summary of these systems is in Table 6.11.

Table 6.11: ET Systems in Western Australia

Year Installed and Community	Annual Rainfall (mm)	Pan Evaporation (mm)	Soil Type	Influent Type
1990 Ninga Mia	260	2,600	Rocky clay	Greywater
1990 Kawarre	680	3,000	Black soil	Ablutions block
1991 Irrungadji	330	n.a	Clay loam	Domestic wastewater
1991-93 Mardiwah Loop	530	3,200	Rocky clay	Ablutions bloc
1992 Parngurr school	310	-	Clay loam	n.a
19994 Parngurr house	310	-	Clay loam	Domestic wastewater
1996 Pia Wadjari	210	3,000	Sandy clay	Kitchen + laundry
1996 Wurrenranginy	680	3,000	Rocky clay	2 house ablutions + school canteen
1997 Tjuntjuntjarra	190	2,400	Sandy clay loam	Domestic wastewater
1998 Jigalong	270	n.a	Sandy clay loam	Greywater
1998 Irrungadji	330	n.a	Clay loam	Domestic wastewater
1999 Tjalku Wara	-	-	n.a	Greywater
1999 Wongatha Wonganarra	234	3,500	n.a	Greywater

The following conclusions can be drawn from Table 6.11:

1. Pan evaporation rates are high and vary from 2,400 mm to 3,200 mm, considerably higher than other areas in Australasia
2. Annual rainfall is relatively low and varies from 190 mm to 680 mm, with an average of 361 mm, on the lower side of other areas in Australasia
3. The ratio of pan evaporation: rainfall varies from 4.4 to 14.9, due to the high pan evaporation and low rainfall

4. With the exception of sandy clay loam at Jigalong, the soils typically have a slow permeability hence, the main effluent disposal mechanism is ET.

In summary, the above Western Australian communities rank as the higher extremes in pan evaporation: rainfall ratios in Australasia.

3.4.3 Effluent Disposal Selection - Victoria

The NRE in Victoria (NRE, 1997) used an on-site disposal selection matrix for conventional infiltration trenches and ET trenches which is presented in Table 6.12.

Table 6.12: Comparison of Conventional Trenches with ET Trenches

Factor	Standard Infiltration Trenches	ETA Trenching
Lot size		
>0.4 ha	s	s
0.1 to 0.4 ha	ms	ms
<0.1 ha	us	us
Land slope		
< 25 %	us	us
>25%	ms	ms
Soil percolation rate		
>1200mm/h	us	ms
50 – 1200 mm/h	s	ms
<50 mm/h	ms	s
Depth to water table		
>2m	s	s
1 to 2 m	ms	us
<1m	us	ms
Depth to rock		
>1.5m	s	s
0.75 to 1.5m	ms	ms
<0.75m	us	ms
Annual rainfall		
>1000 mm	ms	us
<1000 mm	us	s
Frequency of flooding		
>1 ARI	s	s
1 to 5 ARI	ms	ms
<5 ARI	us	us

Density		
>10 dwellings/ha	us	s
2.5 to 10 dwellings/ha	ms	ms
<2.5 dwellings/ha	us	us

Abbreviations:
s = generally suitable
ms = may be suitable
us = unsuitable
ARI = Average Recurrence Interval (years)

The following observations can be made from Table 6.12:

1. ET systems are more suitable for lower percolation soils
2. ET systems are more suitable for higher density developments, since trench/beds areas are less than when compared with standard infiltration trenches
3. ET systems are more suitable for lower annual rainfall, <1000 mm/yr, only if the ET rates are higher than the rainfall
4. The two disposal methods have similar application qualities for slope, depth to the water table and depth to rock.

The assessment information formulated by NRE (1997) has been considered for the model ET system for Queensland.

3.4.4 Maroochy, Qld - ET Experience by Others

Given the high rainfall in the former Maroochy Shire Council, ET beds are not permitted where the soil permeability is less than 0.04 m/day (Code, 1994).

The Council's code has probably been revised since the introduction of AS/NZS 1547:2000. However, the ET bed criteria were as follows:

1. ET bed area for mixed wastewater – 300 m^2 minimum
2. ET bed area for greywater only – 200 m^2 minimum
3. Prepared soil depth – 300 mm minimum
4. Surface to be constructed with greater than 1 in 100 crossfall to stop surface ponding
5. ET bed floor to be level.

These ET design requirements have no doubt been superseded by AS/NZS 1547:2000 but they would have served their purpose. The writer would however, query the 1 in 100 crossfall since it is flatter than others reported. It is for this reason that it has not come into consideration for the model ET system for Queensland.

3.5 North American ET Experience by Others

3.5.1 Allowance for Climatic Conditions

It is inevitable that some months will have higher than average rainfall, a water balance for each month over a year will often provide adequate in-bed storage of water for those higher events into the drier months. A 15-20% increase in area to account for variations in rainfall and effluent flows is suggested by Crites and Tchobanoglous (1998).

According to EPA (1980) some ET and TET designs have not performed satisfactorily. The problem in some cases is the over estimation of the ET potential of shrubs and trees planted on the surface and the overall potential of ET itself. Poor system design has been somewhat offset by leaking liners that give the appearance that the system is performing adequately. Most of the systems are based on the following aspects:

1. Distribution systems placed in 300 mm of gravel at the bottom of the bed or trench
2. Spacing of distribution pipes 1.2 m to 4.0 m, with the lower value preferred for better distribution
3. Wicking is accomplished by a 600 mm layer of 0.1 mm sand and a soil-sand mix to raise the water to the surface.

Most system sizing has been based on the formula:

$$A = \frac{nQ}{ET} - P \qquad \textit{Equation (3)}$$

Where: A = surface area, n = coefficient, which varies from 1 to 1.6, Q = annual flow volume, ET = annual ET rate and P = annual precipitation rate

The contribution of plants remained a matter of controversy at the time (EPA, 1980).

In areas where net ET exceeds net precipitation, total ET systems are utilised (Martens and Associates, 1995). Plant transpiration, nutrient uptake and evaporation are higher in the warmer months. Lower ET rates in the winter months, which result from lower temperatures, and lower daylight hours in humid and temperate climates (Bureau of Meteorology 1991), result in lower rates of disposal which are seasonally dependant.

Many ET systems are in use in Central Texas and other parts in the USA, particularly in the semi-arid regions of the south west. Austin City (1995) feels that ET systems are suitable for higher net ET areas, such as arid and semi-arid climates. Liners are not required if the K value is < 10^{-4} cm/sec.

3.5.2 ET System Locations

Salvato et al (2003) report that ET systems can be installed in the following situations:

1. Where the available soil absorptive capacity is limited
2. Where no topsoil exists over clay
3. Where the bedrock is shallow
4. When a water balance shows the ET + runoff exceeds rainfall + infiltration + inflow. In the case of the Northland and Auckland provinces in New Zealand, based on experience, this is not necessarily the situation (agreed by the writer and Alan Fielding, pers. comm. June 2010).

The rate of ET depends mostly on temperature, relative humidity, barometric pressure, wind speed, soil moisture, and the type of soil and depth of moisture below the soil surface.

3.5.3 Updated USA ET Experience

Mihelic et al (2009) reports on updated ET information as follows:

1. Gravel bed depths 150-300 mm – within the range listed in Table 6.8
2. Sand bed mean diameter of 0.10 mm promotes capillary rise – within the sand size listed in Table 6.8 and Table 6.10
3. A sloping surface can impede at least 30% of the rainfall – less than the 50% suggested in Table 6.9
4. ET can be 70-80% of Class A pan evaporation data.

4 Reflections on Evapotranspiration Systems

4.1 New Zealand ET Projects

4.1.1 Earlier ET Experience in Northland, NZ

The writer has reflected on his earlier (1976 to 1982) ET system investigations and designs within Northland, New Zealand. Based on annual rainfall and Potential ET figures such work could have been on the ambitious side, particularly when compared with the ET work in Western Australia. More recent ET: rainfall ratios differ marginally.

4.1.2 Earlier ET Determinations – New Zealand

It was reported earlier by New Zealand Met. Service (1977) that the calculated and estimated ET values using Penman, Thornthwaite and Priestley-Taylor were based on uniform closed short vegetation cover. The writer was aware of this aspect at the time but also appreciated that rates would increase with higher degrees of site exposure, higher vegetation and the enhancement of the "clothes-line" effect. The writer did consider during the design of ET systems, during the period 1976 to 1982, increasing the ET rates supplied by weather stations by a factor of 10-15%. However, the writer chose to adopt the New Zealand Met. Service data and considered the likely increases in ET as a factor of safety.

4.1.3 Comments on Dr Bernhart's (1973) ET Work

Thinking back the writer found that the work by Dr Alfred Bernhart in Toronto, Canada was most interesting. This experience was applied in New Zealand by the writer and Ian W Gunn, University of Auckland. On reflection however, the writer feels that Bernhart tended to over-estimate the ability of vegetation to evaporate and transpire. This aspect has also been raised earlier. Possibly the main impact would have been on TET systems.

Also, Bernhart (1973) estimated that well mounded ET trenches and beds could be responsible for shedding up to 75% of incident rainfall. In the writer's opinion this estimate is excessive and a more conservative figure of 50% seems more realistic, particularly during higher intensity events. The writer noted that the Taupo County Council experimental ET bed assumed a figure of 50% (Sampson and Bartley 1981). The writer has applied a 50% figure for incident rainfall runoff to the model ETA system for Queensland.

4.1.4 Large ET System, Leigh, NZ

The large and successful ET system for the University of Auckland Leigh Marine Laboratory has been mentioned previously.

The writer conducted a comprehensive water balance for determining moisture inputs and losses from the Rosedale Landfill site in Auckland (Simpson, 1990), which is on the coast south of Leigh.

On reflection, the writer feels that the climatic data for his research area on the North Shore of Auckland could be compared with the Leigh Marine Laboratory site. The writer found that ET increases substantially between October and March of each year. He also found that a correlation existed between sunshine (hours) and ET (mm). The average monthly sunshine hours were 160 or 5.4 hrs per day (Simpson, 1990). The writer feels that these climatic characteristics would be similar to Leigh.

The writer has attended the Leigh Marine Laboratory for University of Auckland field trips in 1974 and over the period 1988-1990. In his opinion it is not surprising that the Leigh Marine laboratory ET system is successful, as reported by Gunn (1988) for the following reasons:

1. The climate is suitable – the site is elevated and exposed to sun and wind
2. The dosing cycle of 1 week operation and 3 weeks resting is conducive to maintaining aerobic bed conditions which enhance disposal mechanisms
3. There is reserve storage within the ET beds to cater for adverse climatic and higher than design loading conditions – all beds could be operated for shorter periods.

4.2 ET Rates and ET Rainfall Ratios

4.2.1 ET Rates in South East Queensland

In South East Queensland typical ET rates for grasses and other vegetation are (Lindsay Furness, Hydrogeologist, pers. comm. 2000):

1. Summer – 12 mm/day
2. Winter – 2 mm/day.

The worst case of winter conditions should be considered. Hence, 8 mm/d would appear to be reasonable if good site exposure, use of evergreen plants, and the "clothes line" effect are available.

4.2.2 ET Rates in USA

The design ET rate in the USA shall be equivalent to the minimum monthly pan evaporation rate, taken from the previous 10 years (Ingham, 1980). In the writer's opinion this is a conservative approach for the following reasons:

1. The benefit of transpiration is not taken into account
2. The benefit of advection is not taken into account, which at times can further enhance ET
3. There is no mention of reserve ET systems, for adverse climatic and flow conditions
4. There is no mention of an overflow pit or trench for adverse conditions

The conservative approach by Ingham (1980) was not applied to the model ET system for South East Queensland.

4.2.3 Australasian ET:Rainfall Ratios

Overall, the different ET: rainfall ratios are a reflection of the climate of each area. For example, Western Australia has very low rainfall but very high pan evaporation. This unique situation, when compared with other climates in Australasia, poses a risk of lack of moisture present at times in the ET systems. On reflection, the Western Australia situation could present the problem of system sustainability, during low rainfall and intermittent and low effluent loading periods. The vegetative cover could come under stress due to a lack of moisture, unless specific species were selected to cover this ephemeral type situation.

The other extreme is the situation in parts of the east coast of Northland, New Zealand where the average annual ET: rainfall ratio is around 0.6 which would normally require careful design as well as the use of ETI systems. The Leigh site gives an ET: rainfall ratio of 0.9. As explained above in this Portfolio the Leigh ET system has been functioning very well, partially due to the specific design approach by Gunn (1988).

The ET: rainfall ratios in South East Queensland show that the Beerburrum area (ET: rainfall of 0.89) is the only area that requires specific design, by ensuring that bed storage and some overflow provision are incorporated. The Lockyer Valley, with a high ET: rainfall ratio of 2.4 could be described as being a classic area for ET systems since this is a lower risk of effluent buildup and overflow.

4.3 Effluent Disposal Water Balance

4.3.1 Water Balance of New Zealand Systems

The writer has a detailed knowledge on climatic characteristics in the Northland Province, Auckland and most parts of the North Island of New Zealand. The writer has undertaken several water balances for ET systems in Northland, New Zealand. Based on this experience, for the purpose of designing individual household ET systems within the areas mentioned above, the writer feels that there is no need to undertake water balances for smaller systems, as for individual homes, for the following reasons:

1. If local climatic data is sought it is usually clear which months have higher and lower rainfall and higher and lower ET
2. The ET system design should take into account the higher rainfalls and lower ET periods, and design for the worst case scenarios
3. The ET systems should have in-built storage to cater for adverse climatic and loading conditions. (This is a good precaution, Alan Fielding pers. com. June 2010)
4. It is imperative to select plant species that are suited to enhance liquid and nutrient uptake, that are also known to grow in the area

5. The writer found it was sound practice to incorporate an overflow system as a contingency plan. This could be a deep trench or a bore hole that would function when the ET bed was about 70% full.

The project the writer undertook for the Golden Bay Cement Company land at Whangarei Heads, Northland show how plots of ET, rainfall, sunshine hours against occupancy periods can be effective in terms of confirming ET effluent disposal suitability. In the case of the experimental ET beds in Taupo County Council the writer feels that it was sound practice to conduct a water balance to provide future design information and so chemical balances could also be undertaken.

The writer concurs with Gunn (1988) that because of the indeterminate relationship between ET and seepage or infiltration into the natural soil, full water balances are now rarely carried out as part of the design process.

4.3.2 Water Balance of Queensland Systems

Having conducted constructed wetland experimental work within Queensland and northern NSW the writer also has a detailed knowledge of climatic characteristics. In areas of Queensland, other than the wet tropics, if the ET system is over-sized the vegetative cover can be stressed during periods of no or low rainfall. That is, the vegetation becomes over efficient at transpiring liquid and plants can die due to the lack of soil moisture. Another scenario is during monsoonal seasons when the ET systems become overloaded. For this reason the writer would question the option of using ET beds in Northern Queensland, unless the particular area is semi-arid or if a detailed water balance was prepared, in conjunction with an overflow facility.

4.4 ET System Suitability

Ingham (1980) reported that the use of ETA systems should be considered only after determining that the site is unacceptable for sub-surface soil percolation alone. The writer does not necessarily agree with this for the following reasons:

1. ETA systems are normally a viable alternative disposal system, in a range of soil types. However, they are not so applicable in high soil K situations, since soil infiltration is likely to play the major role
2. ETA systems are more sustainable, due to the dual disposal mechanisms that are available.

The northern New Zealand climate and growing season is more favourable to the use of ET systems than that of Canada and parts of the USA where Bernhart carried out his work (Gunn, 1994). In the writer's experience the same would apply to Australia, particularly in Northern NSW, Queensland, Northern WA and the NT where climatic conditions greatly favour ET systems.

The ET systems are designed to create favourable conditions for effluent uptake by consideration of the following (Bernhart, 1973, Simpson, 1976 and Patterson, 2002):

1. Utilisation of aerobic microbial conditions
2. Focusing on capillary of effluent, by selected sand and media sizes
3. Planting broad leaf evergreen plants
4. Locating systems to maximise solar radiation and wind eddies over the bed
5. Using shallow designs, so that plants can reach the bottom of beds and trenches
6. Shaping the surface of trenches and beds, by crowing, to divert rainfall runoff during larger events. It has been estimated that about 75% of rainfall is diverted from crowned surfaces (Bernhart, 1973).

Based on the writer’s knowledge of the climatic conditions, the application of TET beds for individual houses in the Lake Taupo, New Zealand area could be questioned for the following reasons:

1. Evaporation rates are lower
2. Transpiration rates are correspondingly low
3. Winters can be very cold, particularly when the winds come off the snow covered mountains
4. The plant growing season is restricted, when compared with Northland and Auckland, New Zealand, which inhibits water loss. Many of the useful species capable of ET would not survive the winters in the Taupo area (Alan Fielding pers. comm. June 2010).

The writer concurs with the outcome of the experimental work (Sampson, 1981) and the suggestion of partially sealed ET beds with an overflow facility.

4.5 ET System Design Aspects

4.5.1 AS/NZS 1547:2000

Clause 4.2A5.7 states that “planting on a site to encourage evapotranspiration shall always be considered in land application systems”. In the experience of the writer this is a sound concept, particularly in lower K value soils, since it provides an added factor of safety. Table 4.2A2 provides ET/infiltration design loading rates (DLR) which should be justified by water balance calculations. The writer agrees that ET/adsorption not normally used for situations with soil categories of 1 to 3, largely due to the higher soil K values. To compare a category 4 soil, consisting of clay loam with a weak structure and an assessed K of 0.35 m/day, the DLR tabled is 8 mm/d.

The writer understands that since extensive work on smaller ET systems has not been undertaken, it would be interesting to compare ET/adsorption designs, as reported by Crites and Tchobanoglous (1998):

$$A = \frac{Q}{(ET - Pr + P)} \qquad \textit{Equation (4)}$$

Where: A = bed area (m^2), Q = annual flow (m^3/yr), ET = annual potential ET rate (m/yr), Pr = annual precipitation (m/yr) and P = annual percolation rate (m/yr)

For ET/infiltration systems the percolation rate should be based on longer term saturated flow conditions.

4.5.2 California State ET Guidelines

Since the climates of sub-tropical and semi-arid Queensland are similar to the Californian climate, the writer's reflections on the California State ET Guidelines by Ingham (1980) are in Table 6.13.

Table 6.13: Comments on the California State ET Guidelines

Factor	California State (Ingham, 1980)	Comments by Simpson
Site exposure	Desirable for sun and wind exposure	Very important that site is exposed to both sun and wind, to enhance ET processes
Runoff control	Diversion required	Diversion of runoff essential to avoid bed hydraulic overloading
Incident rainfall	Nearly all goes into the bed	It is possible to shed some incident rainfall, by increasing the surface cross fall and covering with at least 30 mm of mulch.
Reduced water usage	A sound requirement in 1980.	*Waterwise* program and Federal Government initiatives in Australia have enhanced water reduction practices.
ET rate	Equivalent to minimum monthly winter Class A pan evaporation rate.	The Californian State ET rate tends to be conservative. With ETA beds the soil K can be taken into account, particularly during lower ET periods. For S E Queensland the design rates used are winter 2 mm /d and summer 12 mm/d (Lindsey Furness, Hydrogeologist, Brisbane, pers. comm. 2000)

Depth to groundwater	1.0 m minimum to highest groundwater level	The depth depends on the effluent quality, usage of the groundwater and soil types. Consider 1.0 m to be a minimum in slower K soils.
Sand diameter and depth	0.10 to 0.15 mm sand diameter at 500-750 mm depth	AS/NZS 1547:2000 – 0.5 to 1.0 mm diameter and a depth of 250 mm. Bernhart, 1973 – 0.5 to 1.0 mm diameter and a depth of 300 to 380 mm. The writer uses clean hard coarse sand, within the range of AS/NZS 1547: 200 and Bernhart (1973) and a depth of 300 mm. The key issue is to avoid possible sand blockages and encourage capillary rise.
Surface crowning	2 to 3%	The writer has observed in NZ that higher crowns in trenches are capable of shedding relatively high amounts of incident rainfall – up to 5% slopes.
Gravel diameter and depth	30-40 mm gravel diameter at 200mm depth maximum	AS/NZS 1547:2000 – 6 to 25 mm diameter and depth of 200 mm. Bernhart (1973) – 5 to 20 mm diameter and a 150 mm depth. The writer uses 10 to 30 mm diameter for a depth of 250 mm. On reflection, he should reduce the depth of gravel to 200 mm, since this may impact on capillary rise.
Bottom configuration	Level	Agree with level bottom, to avoid un-uniform effluent distribution
Reserve areas	100%	Agree with 100% future bed area being allocated
Horizontal setbacks	Setbacks for ET and ETI systems are tabulated by Ingham (1980)	The writer considers that horizontal and vertical setbacks are not so relevant for ET systems, since they are often sealed and the effluent is eventually evaporated and transpired.

4.5.3 Benefit of Aerobic Conditions

Aerobic mounds, trenches and beds need a near constant supply of oxygen. This can be sourced from the presence of aerobic micro-organisms such as protozoa, rotifer and nematode. These micro-organisms undertake the beneficial roles of:

1. Keeping the soil pores open, to improve infiltration
2. Enhancing capillary rises for disposal by ET (Bernhart, 1973).

Oxygen is available via more porous mound, trench and bed media and with the introduction of simple vents. The desirable dissolved oxygen content is 1.5 mg/L (Bernhart, 1973). The installation of air vents in trenches and beds was practised by the writer in Northland, New Zealand and in Auckland by Ian W Gunn, University of Auckland. In the case of anaerobic trenches and beds the vents did emit odours on a regular or nuisance basis.

4.5.4 Effluent Chlorination

The chlorination of effluent, to be disposed of by sub-surface means, has been an issue for many years. The writer agrees that the process of chlorination can actually kill useful soil bacteria, which is detrimental for the disposal mechanisms in trenches and beds (Bernhart, 1973). This soil bacteria population is very important for the functioning of ET systems.

A study by Emmanuel et al (2004) showed that the addition of NaOCl to effluent can reduce bacterial pollution but also give rise to toxicity effects of soil/water and aquatic organisms.

5 Conclusions on ET Disposal

5.1 New Zealand ET Projects

Dr Bernhard Contribution

The late Dr Alfred Bernhard made a major contribution to ET technology that has been applied by the writer on New Zealand projects. The University of Toronto investigations were compared and correlated to other research findings in North America, Europe and Japan, published in about 260 papers, reports and personal communications (Bernhart, 1973). The writer would have thought that Ingham (1980) would have reviewed Bernhart (1973) or at least consulted him. The writer rates the late Dr Alfred Bernhart, formerly of the University of Toronto, as a pioneer and guru on evapotranspiration effluent disposal systems. Bernhart (1973) encouraged aerobic action within effluent trenches and beds. The writer advocates that this will improve the performance and sustainability of effluent adsorption and ETA systems.

Taupo County ETA Study

The Taupo County ETA study (Sampson, 1981) found that the observed ET rates were only 80% of those predicted by Bernhart (1973). Much of Bernhart's work was centred on Toronto, Canada. The climatic characteristics in Toronto are similar to the Taupo, New Zealand so the writer cannot find an explanation for the differences in ET rates, other than as mentioned earlier, Bernhard's tendency to overestimate ET.

5.2 ET Rates and ET: Rainfall Ratios

ET: rainfall ratios give an indication of the suitability of ETA and Total ET systems. For example, an ET: rainfall ratio of 0.7, where the ET is lower than the rainfall, shows that specific effluent storage and overflow provision must be made. An ET: rainfall ratio of 4 shows that the ET rate is substantially above the rainfall recorded. Potential ET: rainfall ratios in Northland, New Zealand vary considerably with those in Western Australia. The difference is that rainfall in Northland is relatively high and the ET is seasonal whereas in Western Australia the rainfall is low and the ET is very high. ET: rainfall ratios in South East Queensland show that the region is suitable for ET systems. An exception is the Beerburrum area where the high rainfall could inhibit ET systems unless provisions are made for extra bed storage and the installation of an overflow system. An ET: rainfall ratio of 1.2 has been adopted for South East Queensland, for the purpose of the model ET system for Queensland.

The estimated ET rates from systems with dense vegetative cover dosed with aerobic effluent can exceed measured lake or open water evaporation values by 3 to 4 times (Bernhart, 1973). Patterson (2002) reports, based in work done in 1979, that when water was high in the bed, the ET rate increased and in some instances can be up to 10 times pan evaporation. However, this has not been proved to occur in Australia, but rates up to 2.0 times are common in reed/gravel bed systems.

5.3 Effluent Disposal Water Balance

ET systems are more suitable for areas where ET exceeds rainfall, unless specific design has addressed loadings and climatic conditions, by the means of a water balance. It is sound practice to undertake water balances for larger ET systems. For individual homes a water balance may not be required if the designer has a local climatic knowledge or has worked in the area on previous occasions.

5.4 ET System Selection and Design Aspects

Different ET Systems

There are two different categories of ET systems. Evapotranspiration/Adsorption (ETA) is a dual function unit which draws on ET and soil adsorption as the disposal mechanisms. Total Evapotranspiration (TET) relies solely on ET as the disposal mechanism since the bed or trench is lined. This type is used in more sensitive situations with high groundwater and shallow bedrock. TET systems are preferred in areas where ET exceeds rainfall, unless provision has been made for storing peak flows and providing an overflow, during adverse climatic/soil conditions.

Model ET Design – South East Queensland

The model ET trench for South East Queensland has considered information gained from the early experimental work of Bernhart (1973), the compilation of the Californian State ET Guidelines (Ingham, 1980) and on the design and performance and efficiency of similar systems in New Zealand and Australia.

ET System Surface Slopes

The former Maroochy Shire Council Code (1994) for ET beds stipulated a bed surface of >1 in 100 slopes to prevent ponding. In the writer's experience this figure is not suitable for the following reasons:

1. This does not consider likely bed surface settlement, enhanced by rainfall impact. In time the bed surface would be near level or low spots would develop
2. The writer suggests a bed surface slope is 5% which would allow for some settlement and effectively shed larger amounts of rainfall. Bernhart (1973) recommends 2-3% slope and suggests that surface settlement could be about 25 mm. The shedding of rainfall from the surfaces of ET systems is not to be under estimated and it is imperative on sealed ET systems
3. A bed crossfall of at least 13 mm/m from the centre is cited by Patterson (2002).
4. The bed crossfall of the writer's ETA retrofitted disposal system in Morayfield is 60 mm/m. This is above the minimum suggested by Patterson (2002) but it is not excessive in that it could result in some surface scouring under intensive rainfall events.

Vegetation Selection

The leaves of grasses, plants, shrubs and trees have stomata which release water to the atmosphere. Vegetation can transpire, at a lesser extent, at nights and during rain. The silhouette of larger trees and shrubs ("clothes-line" effect) catch more advected heat which enhances water loss rates.

The development of a comprehensive list of suitable ET vegetation species by A Fielding was an innovative approach at the time and a major contribution to ET disposal technology.

Maximising Surface Runoff and Increasing Storage

The provision of sufficient crossfall on the surface of an ET system is very important, to maximise runoff. The writer favours the crossfall of 60 mm/m of his retrofitted ETA system in Morayfield. In situations where the ET does not exceed the rainfall, on an annual basis, it would be sound practice to increase the design bed/trench area by 15-20%, as suggested by Crites and Tchobanoglous (1998). The provision of an overflow system is another contingency measure in such situations.

ET Capillary Rise

The action of capillary rise or wicking is most important in ET systems. Fine sand and silt improves the functioning of capillary rise, as shown in Table 6.2. Capillary action is enhanced when the effluent distribution pipes are near the surface.

ET System Retrofitting

A standard infiltration trench system can be retrofitted as an ET system, to enhance effluent disposal in low soil K situations (<0.08 m/d) and nutrient uptake. The Simpson ET system has a mixture of planting between the trenches and around the perimeter.

Part (B) Nutrient Uptake by Vegetation and Soil

1 Introduction and Background

The aim of Part (B) is to examine the mechanisms and potential for the uptake or assimilation of nitrogen and phosphorus from effluent, by the soil and selected vegetation, using the three techniques of the writer's version of OSRAS (2001), a *SEQWater* model and the *MUSIC V4* model.

The objectives to achieve this aim are to:

1. Discuss the use of vetiver grass in three case studies within South East Queensland
2. Discuss the use of kikuyu grass using a case study, based on a methodology the writer developed, on a 3,000 m^2 lot within a *SEQ Water* catchment in the Caboolture area.
3. Undertake a case study using the *SEQ Water* and *MUSIC Model Version 4* methods of determining assimilation areas for effluent, and compare this with the result with the case study in (2) above.
4. Discuss phosphorus buffering capacity, using a case study in the Caboolture area
5. Reflect on information gained and case study outcomes
6. Draw conclusions.

Nutrients such as nitrogen, sulfur, phosphorus and carbon are essential for the growth of living things. The discharge of excessive nutrients can result in high concentrations at the point of discharge and this gradually falls due to dilution by incoming surface and groundwaters (Connell, 1993). Nutrient discharge can lead to the stimulation of grasses and aquatic plants, which may be beneficial in some cases, but detrimental in others.

Nutrient enrichment can lead to algal blooms, which eventually die and decompose. Their decomposition removes oxygen from the water and potentially leads to levels of dissolved oxygen that are insufficient to sustain life forms.

Soils are relatively effective for purifying degradable wastes as nutrients. Nitrogen and phosphorus are essential for vegetation growth. As a rule of thumb, phosphorus uptake by plants is 8 to 10 times less than nitrogen uptake (Gardner et al 1997).

In freshwater ecosystems and sensitive water supply catchments it is important that the nitrogen and phosphorus, generated in effluent, are contained within the soil and grasses and other vegetation. Nitrogen pollution of some groundwaters and surface waters has become a problem in some densely urbanised areas and some agricultural areas.

This section examines methods for the uptake of nitrogen and phosphorus via vetiver grass, kikuyu grass and other selected vegetation.

2 Vetiver Grass Systems

Vetiver grass *(Vetiveria zizanioides)* is native to South and South East Asia, where it has been used for centuries. It has been used extensively for slope stability and the rehabilitation of degraded and disturbed land (NRM, 2001). This very robust perennial grass, due to its extensive and deep root system, can tolerate drought, extreme heat and frost. It tolerates acidic soil conditions with very high levels of aluminium, manganese and a range of heavy metals. Since 1989 this grass has undergone extensive field trials for soil conservation, sediment filtration and effluent disposal in Queensland. It is propagated using rooted clumps.

2.1 Esk Shire Council Experience

The town of Toogoolawah, on the Brisbane Valley Road, with a population of approximately 1,300 people, had a treatment system consisting of an Imhoff tank and three wastewater lagoons. The high nutrient content of the final effluent was a concern. A vetiver grass effluent treatment system had earlier been developed by the Queensland Department of Natural Resources (Granzien and King, 2003). Vetiver was found to have a very rapid and high capacity for absorbing nitrogen and phosphorus.

Some 21 floating pontoons containing vetiver grass were constructed and trialed. The lagoon margins were also planted with vetiver grass. Several overland flow plots, planted with vetiver grass, were also established to function as a tertiary treatment system.

The success of the Toogoolawah vetiver grass system is demonstrated by the nutrient reductions results from Granzien and King (2003) in Table 6.14. It is to be noted that these were interim results only, since the grass was not fully developed at the time.

Table 6.14: Effect of Vetiver Grass on Effluent N and P Concentrations

Date	**Source**	**Ammonia - N (mg/L)**	**Total N (mg/L)**	**Total P (mg/L)**
15/10/2002	Previous effluent	9.1	20	6.3
8/4/2003	Lagoon influent	49	58	6.6
	Lagoon effluent	0.65	15	3.3

	Wetland effluent	0.57	6.7	1.2
20/5/2003	Lagoon influent	34	41	9.2
	Lagoon effluent	2.9	14	4.4
	Wetland effluent	0.07	7.3	2.1

It can be seen from Table 6.14**,** by comparing the 20/5/2003 lagoon effluent results with the matching wetland effluent results, that very short term reductions of nutrients had been achieved by this natural biological process.

The above nutrient stripping qualities of vetiver grass attracted the writer at the time, since the nutrient reduction potential exceeded all other grasses, including kikuyu grass. The writer was invited to inspect the project, particularly for the vetiver grass overland flow component. It was agreed at the time (between the writer, council wastewater engineer and plant operator) that the main potential was based around the overland flow plots of vetiver grass. During much of the year zero effluent discharge was being achieved, which is considered to be an ideal scenario, since the town is within the water supply catchment of Somerset Dam, Queensland.

2.2 Vetiver Grass Effluent Treatment at Davis Gelatine, Sunny Hills, Queensland

This industry discharges 1.3ML/day of effluent. Some 120 ha of land was licensed by the Queensland EPA for the disposal of gelatine manufacture effluent. This effluent is characteristically high in nitrogen, TDS and sulphate. Most pasture and annual cropping species have limited salt tolerances. The accumulation of salts in surface soils is a serious problem so flushing of the soils is required. The treatment required nitrogen removal to protect the groundwater. The vetiver grass system was identified as having the potential to meet all the treatment criteria (Truong and Smeal, 2003).

The interesting findings from work undertaken between 2001 and 2003 (Truong and Smeal, 2003) are as follows:

1. Vetiver grass has the potential to export up to 1,920 kg/ha/yr of N and 198 kg/ha/yr of P, if regularly harvested and removed
2. Kikuyu grass has the potential to export 687 kg/ha/yr of N and 77 kg/ha/yr of P
3. Rhodes grass has the potential to export 399 kg/ha/yr of N and 26 kg/ha/yr of P

4. *MEDLI* model simulations (described later) on kikuyu grass, rhodes grass and vetiver grass indicate that only vetiver grass was able to dispose of the effluent output
5. After a vetiver grass calibration, the *MEDLI* model re-run showed that vetiver grass required the least land for sustainable irrigation for both nitrogen and effluent volume. Refer to Table 6.15 for the results.

Table 6.15: Land Requirements for Irrigation and N Disposal with Three Grasses

Plants	Land Needed for Irrigation (ha)	Land Needed for Nitrogen Disposal (ha)
Vetiver	80	70
Kikuyu	114	83
Rhodes	130	130

Table 6.15 shows that vetiver grass is more efficient for nitrogen assimilation and disposal. It is interesting to note that kikuyu has a higher nitrogen uptake potential in relation to other grasses, as shown by the lower land requirement.

Under shallow overland flow conditions, vetiver grass had the highest water use rate, as compared with wetland species such as *Typha spp*, *Schoenoplectus validus* and *Phragmites australis*. This is due to the very deep and high root mass.

The writer has noted that vetiver grass does not function under a body of water, as with wetland emergents. The grass is likely to drown if planted in a body of water. The experience of the former Esk Shire Council together with the writer's Watson Park experience showed that with an overland flow action, a depth of effluent < 10 mm was more suitable.

2.3 Watson Park Convention Centre and School Vetiver Grass Trials

Based on the potential demonstrated by the Toolgoolawah system, trials were undertaken at Watson Park, Dakabin, Pine Rivers Shire under the writer's supervision.

The treatment system consisted of septic tanks as primary settling units, three wastewater lagoons and an effluent holding pond. Effluent was irrigated over a designated field with trees. The effluent quality concerns included:

1. Occasional exceedance of BOD_5 (limit 80 percentile 30 mg/L and maximum 90 mg/L)

2. Occasional exceedance of suspended solids. (limit 80 percentile 20 mg/L and maximum 60 mg/L)
3. Total nitrogen periodic exceedance (no licence limit but accepted limit 10 mg/L)
4. CFU frequent exceedances (limits median 1,000/100 mL and 4 out of 5 <4,000 /100 mL).

Based on a suggestion of the Queensland EPA, the writer and the camp manager established a system of vetiver grass pontoons and edge planting in lagoon 3. The effluent monitoring results are in Table 6.16.

Table 6.16: Effluent Monitoring Results of Vetiver Grass Pontoons – Watson Park.

Parameter	**Results (1)**					**Comments**
	16/03/04	**1/06/04**	**20/07/04**	**3 /11/04**	**23/02/05**	
pH	6.88	7.58	9.25	8.98	7.2	Large variation. High figures indicative of algal action.
BOD_5	7	17	24	20	10	Variation - one result over the licence
Suspended Solids	28	49	88	118	47	Large variation, only one result within the licence. Indicative of algal presence.
Ammonia - N	1.48	<0.02	<0.02	0.72	-	Variation
NO_X - N	<0.02	<0.02	<0.02	<0.02	<0.02	Consistently low results
Total N	3.4	4.5	6.0	11.1	4.6	Variation, higher results of some concern.
Total P	1.66	1.7	1.6	5.58	4.3	Variation, higher results of some concern.

Note:

1. pH in units, all other results in mg/L

Some nutrient reduction was being achieved but performance results were mixed and tended to be non-conclusive. It was agreed that a much higher density of vetiver grass pontoons was required to obtain more consistent nutrient reductions.

Work undertaken in Bangkok (Boonsong and Chansiri, 2008) using the platform technique gave reductions of 90-91% BOD_5, 61-62% Total N and 17-36% Total P with retentions times varying from 3 to 7 days.

The alternative at Watson Park was to install a small overland flow plot, planted with vetiver grass, which was done. A photograph of the vetiver grass overland flow plot is in Plate 6.1. This photograph was taken in early October 2011, some weeks after the vetiver grass plot had been replanted in conjunction with kikuyu grass.

Plate 6.1: Watson Park, Dakabin – Vetiver Grass Overland Flow Plot

To check the efficiency of the vetiver grass plot, three effluent samples have been taken. The vetiver grass had been planted for several weeks before the grass plot was commissioned with lagoon effluent. After being operational for about a week, a single grab sample of the final effluent was collected from the final (storage) lagoon and tested 5 May 2008. Follow up samples were tested on 29 April 2009 and 27 October 2010. The results are presented in Table 6.17.

Table 6.17: Final Effluent Results – Watson Park

Parameter	Final Effluent – 5 May 2008	Final Effluent – 29 April 2009	Final Effluent - 27 October 2010	Final Effluent - 8 December 2010 (1)	Comments
BOD_5	<3 mg/L	3 mg/L	5 mg/L	15 mg/L	Consistent results with the exception of 8 December 2010. All results well within secondary standard.
Suspended Solids	27 mg/L	2 mg/L	11 mg/L	37 mg/L	All under secondary standard with the exception of 8 December 2010
Ammonia -N	0.9 mg/L	<0.5 mg/L	0.17 mg/L	<0.02 mg/L	Reasonable consistent results with the exception of very low result 8 December 2010
TKN	3.4 mg/l	1.3 mg/L	1.2 mg/L	<0.02 mg/L	Reasonable consistent results with the exception of very low result 8 December 2010
Organic N	2.5 mg/L	1.3 mg/L	0.19 Mg/L	3.4 mg/L	Reasonably consistent results
Total P	1.74 mg/L	1.7 mg/L	0.3 mg/L	0.63 mg/L	Reasonably consistent results

Note:

1. Sampling undertaken after heavy rainfall period of 149 mm over 5 days

It is fully appreciated that the sampling is very limited but the above results indicate that it is feasible that a high standard of final effluent can be achieved, particularly when the vetiver grass plot has fully established itself. Wastewater lagoons traditionally produce high suspended solids and elevated BOD_5 results largely due to the action of algae. This has been the past experience with the Watson Park treatment system. Apart from the suspended solids result, in the writer's experience, the above results were amongst the best or lowest recorded for this project. Taking into consideration that this sampling has been limited to three occasions the above results show the grass plot is a most worthwhile unit process, for nutrient stripping and polishing wastewater lagoon effluent. It must be appreciated however, that the soil could have removed some of the nitrogen and phosphorus. The vetiver grass plot is also responsible for high evapotranspiration (Camp Manager pers. obs).

Ideally, a testing program should be devised and the system monitored at least every 3 months for at least two years, to establish the full potential of this overland flow grass plot. This raises an environmental engineering problem in that management often does not see the need or allocate the funds for performance monitoring in more depth.

3 Kikuyu Grass Potential for Nutrient Uptake

Kikuyu grass *(Pennisetum clandestinum)* occurs naturally on the highland plateau of east and central Africa. It is now well distributed throughout the sub-tropics of Australia (Meares, 1970) and, in the writer's experience, throughout Auckland and Northland, New Zealand.

The potential of kikuyu grass for nitrogen uptake has been compared with vetiver grass in the earlier section. As a commonly used grass, it ranks highly.

The writer has used kikuyu grass for the assimilation of nutrients, in particular nitrogen, for on-site effluent irrigation areas in more sensitive areas or within water catchments in South East Queensland. The nitrogen and phosphorus are taken up by the soil, vegetation and liberated in the form of nitrous oxide, into the atmosphere (James Pulsford, Agricultural Scientist and Agronomist, Caboolture, pers. com.).

3.1 Barlow, Stanmore – Nutrient Assimilation Zone Case Study

One of the writer's design reports within South East Queensland water supply catchments included a case study (the writers report was for a client named Barlow, February, 2006) for nutrient assimilation. The writer based his nutrient assimilation methodology on consideration of OSRAS (2001), the research findings of soil scientists/chemists (mentioned below) and his own experience in water quality and effluent disposal.

In sensitive water supply catchments managed by *SEQ Water*, it is important that the main nutrients generated from the treatment of wastewater, being Total N and Total P, are contained within the site soils, grasses and other vegetation.

Experience has shown that in the design of on-site effluent disposal systems generally, in other than sands, the focus nutrient is nitrogen. Total P appears to be limiting in some very sandy soils or where effluent treatment produces low Total N concentrations, based on the experience of research scientists from the Qld DNR and W, as mentioned below.

Since the focus on the assimilation of Total N for soils containing organic matter and clays had been the subject of much scientific discussion in the early 2000s the writer approached very experienced scientists in South East Queensland on their views. These included Dr Phil Moody (soil scientist and phosphorus researcher), Glen Barry (soil scientist), Dr Heather Hunter (soil scientist) and Ann Woolley (wastewater chemist), all from the Department of Natural Resources and Water at the time of consultation. All these specialists concurred that Total N is the nutrient to focus on, except in very sandy soils.

Total P is of less concern since it is combined with organic matter and clays whereas ammonia nitrogen and nitrate is mobile in soils, so phosphorus is of less concern. Total N assimilation capacity is therefore used to determine the required assimilative area (OSRAS, April 2001)

The potential for Total P uptake by kikuyu grass is rated as high. Dr Phil Moody estimated that kikuyu was capable of assimilating 40-60 kg of phosphorus /ha/year. Mr James Pulsford (consulting soil and agricultural scientist) confirmed that 40 kg of phosphorus /ha/year was suitable for design purposes. He advised that that the nitrogen uptake by kikuyu grass was at least 5 times that of phosphorus.

Land slopes were required to be gradual over the effluent field, perhaps <5% (James Pulsford, pers. comm.)

The features of the site are:

1. The upper soil was sandy loam for a depth of 200-800 mm
2. Site exposure enhanced ET disposal
3. Runoff control and diversion was not required
4. Groundwater was not located. The soil profiles were limited to a depth of 2.0 m
5. Kikuyu grass with selected shrubs was recommended since they have nutrient uptake potential.

Assimilation Area Determination

The average nitrogen uptake rate was approximately 140 kg/ha/annum (OSRAS, 2001). The secondary effluent had an assumed 32 gm/m^3 of nitrogen (OSRAS, 2001 - nitrogen being the nutrient to focus on). The Total N loading over the coverage of the nutrient assimilation zone was calculated to be 7.6 kg/annum (32 gm/m^3. 0.65 m^3. 365 days/ 1,000).

Since 140 kg of nitrogen per hectare can be assimilated per annum, for 7.6 kg of nitrogen the required nutrient assimilation zone was 543 m^2. The nutrient assimilation zone included the effluent irrigation area and a 100% reserve irrigation area; a *SEQ Water* requirement.

The Barlow, Stanmore case study nutrient assimilation zone requirement, using the above method, is an area of 543 m^2.

The nutrient assimilation zone management practices that the writer has developed include the following:

1. The nutrient assimilation zone is not to be fertilized
2. The planting of more mature selected shrubs, to optimise nutrient and effluent uptake, is encouraged in the initial stages particularly
3. Failed kikuyu grass cover and selected shrubs are to be replaced by turf and new plantings
4. The spacing of selected shrubs needs to consider sufficient room for mowing, pruning, effluent application and sun and wind contact
5. Some mulching around the selected shrubs is advisable, during the establishment stage
6. There is no real need to catch grass clippings where this material was not expected to readily migrate off - site, for example, to waterways (a point of issue with some authorities)
7. The effluent sprinklers should be shifted periodically, to enhance effluent distribution
8. The location of the nutrient assimilation zone must consider surface stormwater paths, to reduce the likelihood of contamination of streams and dams.

3.2 SEQ Water Model

SEQ Water has developed a water balance model calculator for the purpose of determining the land areas required for rural/residential blocks for assimilating nitrogen and phosphorus within the catchments of South East Queensland. This model calculator was developed in conjunction with the *SEQ Water* Development Guidelines (2007) and it enables the user to evaluate water saving devices and land characteristics to determine how much area is required for irrigation and to assimilate the nutrient content. The "model water balance calculator" is available on-line on the *SEQ Water* website found at <www.waterbalancemodel.seqwater.com.au/Index.aspx>.

Table 6.18 lists different scenarios for an assumed effluent loading of 700 L/day (the assumed daily flow for model comparative purposes) at a design irrigation rate (DIR) of 5.0 mm/d.

Table 6.18: Scenarios for Nutrient Assimilation Areas – *SEQ Water* Model

Soil Type	**Soil Texture (1)**	**Grass Cover**	**Nitrogen** (m^2)	**Phosphorus** (m^2)
Sandy loam	weak	Kikuyu	360	980
Sandy loam	massive	Kikuyu	360	980
Sandy loam	weak	Couch	780	1,260
Clay loam	moderate/strong	Kikuyu	360	980
Clay loam	moderate/strong	couch	630	1,260

Note:

1. Soil texture, as per AS/NZS 1547:2000 Table 4.1D4. For example, sandy loam can be weak or moderate and cohesive clay is strong.

The results in Table 6.18 show the following:

1. Land areas required for nitrogen are substantially lower than the areas required for phosphorus assimilation
2. Kikuyu grass performs better than couch, as shown by the smaller land area requirements.

It would appear that the *SEQ Water* model has not considered the uptake role of the soil. A later discussion on phosphorus buffer capacity shows, in the case of organic and a clay soil, that phosphorus is immobilized in the soil thus requiring a minimal land requirement. The focus should be on catering for nitrogen assimilation only.

For the Barlow case study at Stanmore, the upper soil type is sandy loam and moderately structured. Considering that nitrogen is the target nutrient and kikuyu grass is to be established, the nutrient assimilation zone required is 360 m^2. If phosphorus is the target nutrient, then 980 m^2 is required.

3.3 MUSIC V4 Model

The *MUSIC* model was original developed by Monash University in the form of V1 in May 2002. The latest Version 4 was released in September 2010. *MUSIC* is a conceptual design tool and an aid to decision making. It is to be appreciated that it is not a detailed design tool.

The writer commissioned *Hendriks/House (Caboolture) Pty Ltd* to explore the use of *MUSIC* V3 and V4 for determining the nutrient assimilation area for the Barlow case study at Stanmore (Hendriks/House, 2010). A series of scenarios were modelled to explore the sensitivity of the model versions, which included the following:

1. Hydraulic conductivity sensitivity test
2. Soil type sensitivity test
3. Treatment node sensitivity test.

It was soon found that the *MUSIC* V4 superseded V3 but difficulties were encountered with the application of new V4. These difficulties arose as the current objectives were designed for V3 and not modified to suit V4. Since the time this work was undertaken, a new guideline has been produced. Nutrient assimilation areas using MUSIC V3 and V4 are listed in Table 6.19.

Table 6.19: Nutrient Assimilation Areas – *MUSIC* V3 and V4

Scenario	**Hydraulic Conductivity (mm/hr)**	**Soil Type**	**Treatment Node Type (4)**	***MUSIC* Version**	**Treatment Area Required (m^2)**
Hydraulic Conductivity Test					
1	30 (1)	Silty loam	C	V4	515
2	37.5 (1) (3)	Silty loam	C	V4	516
3	50 (1)	Silty loam	C	V4	519
Soil Type Sensitivity Test					
4	37.5 (2) (3)	Sandy loam	C	V4	523
5	37.5 (2) (3)	Loamy sand	C	V4	523
6	37.5 (2) (3)	Silty loam	C	V4	515
Treatment Node Sensitivity Test					
7	37.5 (3)	Silty loam	A	V3	1,000
8	37.5 (3)	Silty loam	B	V4	1,050
9	37.5 (3)	Silty loam	C	V4	516
10	37.5 (3)	Silty loam	D	V4	75
11	37.5 (3)	Silty loam	E	V4	43

Notes:

1. These figures were used for testing the sensitivity only
2. Sandy loam, loamy sand and silty loam are all similar soil types with similar hydraulic conductivities
3. The model scenario
4. These denote media and bio-filtration nodes. The V4 bio-filtration node introduced new science into the algorithm giving the options of un-vegetated, or vegetated, with effective or ineffective nitrogen removal properties.

Taking a treatment node of C and a hydraulic conductivity of 37.5 mm/hr, referring to Table 6.19, then the scenario 4 (since the site has sandy loam) requires 523 m^2.

Adopt a nutrient assimilation zone area of 523m^2 using *MUSIC V4.*

It can be seen that the writer's method based on OSRAS (2001) and using soil science and water engineering principles compares very well with the *MUSIC V4* model outcome. The *SEQ Water* model gives considerable larger total nutrient assimilation area of <u>980 m^2</u>.

4 Phosphorus Buffer Capacity

The phosphorus buffering capacity is a measure of the ability of a soil type to retain or uptake phosphorus. The writer worked for a period with Dr PW Moody, Principal Soil Scientist, Queensland Department of Natural Resources on the potential for phosphorus uptake in wetlands and wetland soils. To determine the movement of phosphorus, if any, within effluent irrigation areas, the writer undertook a case study of the phosphorus buffering potential of clay loam soils. Clay loam upper soils are common to much of the area where the writer carried out on-site effluent disposal consultancy work in the Caboolture area.

The phosphorus buffer index (PBI) was determined according to the methodology detailed in Burkitt et al (2000). The Australian Soil and Plant Analysis Council have specified this method as the preferred one for characterising soil P buffer capacity (Dr PW Moody, pers. comm., 2003).

Two clay loam soil samples from one of the writer's land development projects at Caboolture River Road, at depths of 0-280 mm and 300-500mm, were analysed by the DNR for phosphorus buffer index. The results are in Table 6.20.

Table 6.20: Phosphorus Buffer Index Results

Date Registered	4/11/2003
Requesting Officer	Dr P Moody
No. of Samples	2 moist soils
Comments	For John Simpson
	Batch 426 Lab Nos 11950, 11951
Batch No/Lab. No.	

Phosphorus Buffer Index (1000 mg P/kg)

Soil ID	**Depth (mm)**	**P left in solution (mg/L)**	**Colwell P (mg/kg)**	**P Buffer Index**
Caboolture River Road soil	0-280	60.35	13	76
Caboolture River Road soil	300-500	30.07	8	175

Comments on the two PBI results in Table 6.20 are as follows (Dr P Moody, DNR):

1. The PBI of 76 is described as low in which case phosphorus was highly unlikely to leach
2. The PBI of 175 is described as moderate and also highly unlikely to leach
3. Any off-site movement of phosphorus from these sample clay loam soils will only occur through erosion.

Both the above soil samples indicate that phosphorus is immobilised within the soil environment. The focus is then on how the nitrogen is assimilated.

The adsorption of phosphate in soils is both concentration and time dependent. Some interesting work on phosphorus travel was undertaken at an effluent disposal site in Albany, Western Australia. The estimated minimum mobility times for phosphorus in 3.5 m of top soil ranges from 300 to 700 years. Theoretically >95% of phosphorus from wastewater could be stored in the soil profile as hydroxyapatite and fluoropatite. (Gerritse, 1993).

5 Reflections on Nutrient Assimilation

5.1 MEDLI Model

The use of the *MEDLI* model for small scale effluent disposal projects has caused the writer some concern for some time. Many South East Queensland councils still specify that the *MEDLI* model be used for smaller scale water quality situations. The writer gathered from interviewing a Chartered Professional Engineer candidate who had used the model extensively, that *MEDLI* is characterised by the following:

1. It caters more for the use of larger numbers, for example BOD and Nitrogen, for large agricultural crop areas rather than smaller urban or rural/residential lots

It focuses more on farm wastes rather than treated domestic effluent

It is felt that it is not suitable for single rural/residential lots.

Giving this model more thought it would appear to be unsuitable for small lots for individual homes.

5.2 MUSIC Model

On reflection, it is interesting to note that nutrient assimilation zone areas, as determined by *MUSIC* modelling and other means, with similar climatic, site and soil characteristics, do not necessarily produce answers within the same order.

6 Conclusions on Nutrient Assimilation

The aim of examining the mechanisms and potential of nitrogen and phosphorus assimilation from effluent, by soils and selected vegetation has been achieved by the application of projects and case studies in South East Queensland.

Use of PBI Buffer Index

In other than sand, phosphorus is readily immobilised and nitrogen must be assimilated by the soils and a vegetative cover. The phosphorus buffer index (PBI) can be used for determining whether phosphorus is likely to leach from soils in effluent disposal areas.

Vetiver and Kikuyu Grass

Vetiver grass is very robust, it has an extensive rooting system, and it can tolerate heat, drought and frosts. The high potential of vetiver grass for reducing nitrogen and phosphorus has been demonstrated at the Davis Gelatine plant, Toogoolawah and at Watson Park. Both sites are in South East Queensland. At the Davis Gelatine plant it showed the potential for exporting up to 1,920 kg/ha/yr of nitrogen and 198 kg/ha/yr of phosphorus if harvested.

Vetiver grass trials at Toogoolawah found that this grass species had a very rapid and high capacity for absorbing phosphorus and nitrogen.

A vetiver grass overland flow plot at Watson Park, Dakabin is stripping nutrients from lagoon effluent. This method of treatment is recommended for treating domestic effluent.

Kikuyu grass has the potential for exporting up to 400 kg/ha/yr of nitrogen and 26 kg/ha/yr of phosphorus. Kikuyu grows well in the subtropics and apart from vetiver grass it has the highest capability for up taking nitrogen and phosphorus. It is recommended for small and large scale effluent irrigation fields.

Barlow Nutrient Assimilation Case Study

The Barlow case study of a rural/residential property within the Somerset Dam water supply catchment showed that a total nutrient assimilation area of $\underline{543m^2}$ is required, using the OSRAS (2001) basis and soil science and water engineering principles. This figure compares well with Gardner et al (1997) who reported that on nutrient and hydraulic loading criteria, an effluent irrigation area of at least 500 m^2 is required.

Using the *MUSICV4* model a total nutrient assimilation area of $\underline{523m^2}$ is required. This compares well with the above method.

When applying the *SEQ Water* model a total assimilation area of $\underline{980m^2}$ is required, which is considerably more than the above two methods. It would appear that the ability of the soil to take up nutrients was not taken into consideration. This could be an explanation for the high assimilation area requirement, when compared with the other methods, particularly for phosphorus.

It would appear that the *MEDLI* model is less suitable for the determination of nutrient assimilation on small scale or individual home projects.

The writer has concluded that the first principles determination, based on OSRAS (2001) and using sound soil science and water engineering principles, is worthy of adopting as a design approach for sizing domestic scale nutrient assimilation zones. It is interesting that the outcome using the above mentioned methodology compared very favourably with the *MUSIC V4* model.

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Portfolio 7 – Deep Shaft Disposal and Sand Mounds

This Portfolio consists of Part (A) which covers a survey of deep shaft disposal and Part (B) which is on sand mounds. Deep shaft disposal is not a conventional system and sand mounds are not widespread. Both methods of effluent disposal are responsible for the ongoing treatment of wastewater.

Part A

<u>Part B</u>

Part (A) Deep Shaft Disposal

1 Technology Background

The use of deep shafts, from 2.0 to 6.0 m, for domestic effluent is considered an alternative or innovative technology option, and it is not a common practice in Australia or New Zealand.

Deep shafts have been defined by Barnett and Ormiston (2010) as “a form of deep infiltration system, typically around 6 m deep, used on sites where low permeability soils, such as poorly drained clays, are underlain by more permeable subsoils at depth”.

The writer undertook a study of domestic effluent disposal using deep shafts in the upper North Island of New Zealand for rotary drilling contractors, *Fenwick Drilling.* This company had been involved in drilling deep effluent disposal shafts in the Provinces of Northland, South Auckland, Waikato, Bay of Plenty and King Country, for many years (Simpson, 1976).

The aim of the study was to investigate the performance of existing deep shafts within the areas listed above, and the suitability of a range of site conditions.

The study objectives, for the purpose of the survey and report in 1976 and for technology update, are as follows:

1. Undertake a literature review, at the same time of the survey and also appreciating that this technology was not conventional
2. Interview the rotary drilling operators (the writer’s client), prior to the study
3. Conduct a postal survey on deep shaft disposal of various local authorities within the upper North Island of New Zealand
4. Analyse the responses and compile the survey results
5. More recently, updating the technical literature findings
6. Updating the technical aspects of deep shafts and Simpson’s contribution
7. Review the challenges with deep shafts, as a result of the survey and a more recent update
8. Deep shaft construction aspects
9. Deep bore case studies, undertaken after the survey and report
10. Reflections and recommendations
11. Conclusions.

Publications by Simpson Appropriate to Portfolio 7

1. Simpson, JS 1976, Report on Deep Shaft Effluent Disposal in the North Island of New Zealand, prepared for *Fenwick Drilling Contractors*
2. BCHF 1976, report on Effluent Disposal during Natural Ground Stability Titirangi Park Estate Residential Subdivision, *Beca Carter Hollings and Ferner Ltd*, Auckland, November (Simpson report as an appendix)
3. Simpson, J S 1983, The Feasibility of Domestic Effluent Disposal by Ten Metre Vertical Shafts, Summary Statement for Auckland Consultants, 14 February
4. Simpson, J S 1984, Feasibility Report on Sewage Collection, Treatment and Disposal Alternatives for Waiheke Island, New Zealand

1.1 Suitability of Deep Shafts

Deep shafts have an application in areas where the upper soils contain pans, making it unsuitable for trenching and beds, and where more permeable stratum exists at depth. This technology has been in use in the North Island of New Zealand since the early 1970s (survey by Simpson, 1976). Deep shafts have been in use, to a lesser extent and in more recent years, in some local government areas within Australia.

It has been concluded by several researchers that lower permeability soils provide a higher retention time for microbial activity. Based on research in the 1970s (and prior to the survey) the following aspects relating to deep shaft disposal had been established:

1. Mulcock (undated) had been engaged in a research project on the contamination of groundwater by effluent in the Rolleston area, near Christchurch. From work in very porous soils it was found that indicator organisms did in fact pass through the layers of soils and gravel into the groundwater
2. Studies by the New Zealand Department of Scientific and Industrial Research (DSIR) Freshwater Research Team in Taupo had confirmed that nutrients from effluent leached through the very porous pumice type soils to the groundwater and then into Lake Taupo
3. Hall (1970) had established that very permeable soils were far less effective in removing phosphorus compared with those which were more obstructive to the infiltration of effluent
4. Reneau (1975-76) found that lower permeability soils offered more effective barriers to the vertical movement of phosphorus
5. Healy (1974) pointed out that single deep shaft systems for individual homes are often flooded continuously
6. Cole (1974) advocates that seepage pits should be terminated at least 1.2 m above the water table.

The US EPA (1980) presents a useful comparison of effluent disposal systems in Table 7.1. For the purpose of this table, deep shafts are shown as pits, since very similar principles are involved.

Table 7.1: Deep Shafts compared with Other Disposal Options

Method	Soil Permeability	Depth to bed rock	Depth to groundwater	Slope	Lot size
Trenches	Rapid / moderate	Deep	Deep	0 – 15 degrees	Medium
Beds	Rapid/moderate	Deep	Deep	0-5 degrees	Medium
Mounds	Very slow /rapid	Shallow to deep	Shallow to deep	0 -15 degrees	Small
Pits	Moderate to rapid	Deep	Shallow to deep	0 – 15 degrees	Small
ET trenches	Very slow to rapid	Shallow to deep	Shallow to deep	0-15 degrees[1]	Medium
ETI trenches	Moderate to rapid	Deep	Deep	0-15 degrees	Medium

Note:

1. High evaporation required over 5 degrees slope

The following conclusions can be made from Table 7.1:

1. Pits or deep shafts and mounds are suitable for small lots or blocks, for example 1,000 m^2
2. Trenches and beds required greater depths to bed rock and groundwater
3. Pits or deep shafts require soil with some degree of permeability
4. Pits or deep shafts are considered satisfactory on slopes up to 15 degrees. In the writer’s opinion this is a generalised suitability rating only. The other important factors that need to be checked are slip circle potential, groundwater movements and ground surface cover type.

A literature review undertaken since the deep shaft survey has revealed more advantages and potential constraints of his technology. Whilst there are few Australasian case studies on deep disposal systems and groundwater quality, the early stage of an ongoing study at Dodges Creek, Tasmania has shown a connection between shallow groundwater quality, the density of residences and domestic on-site disposal systems (Whitehead and Geary, 2000).

The lower soil horizons and the unsaturated zone are very active environments for contaminant attenuation. Pathogenic protozoa and bacteria are normally eliminated but viruses are more resistant. Some but not all inorganic compounds are reduced but the removal of nitrogen compounds varies considerably. The organic carbon content is reduced considerably. If sited well, deep effluent disposal is simple and robust and it can effectively recharge groundwater resources. A sound understanding of the local hydrogeology is required to design effective deep effluent disposal systems (Foster et al, 1994).

Deep shafts have an application where permeable sub-soils exist, under poorly draining clayey upper soils (Gunn, 1989). The presence of weathered rock at depth results in economical effluent disposal (Gunn, 1994). As shown by the deep shaft survey in New Zealand this type of geology exists in many parts of the upper North Island.

Because of clogging problems in effluent trenches some communities on the Canterbury Plains, New Zealand have used 4-20 m deep soakage shafts, sunk to underlying alluvial gravels (Sinton, 1986). The high permeabilities of the gravels ensured the longevity of the disposal shafts, some having been in service for over 20 years. However, groundwater surveys have shown some evidence of faecal contamination.

On more potentially sensitive sites, a specialist soils/geological consultant should be called in to advise on the design, the likely structural constraints, possible environmental constraints, the ultimate fate of the effluent and the need to provide pre-treatment. Gunn (1989) suggested the following investigation steps:

1. Soil profile using a 50 mm auger hole to identify a suitable permeable soil layer at depth
2. Drill two 600 mm diameter shafts to up to 6m depth, ensuring a clearance of at least 500 mm from the saturated zone or standing groundwater level
3. Test load each shaft with clean water over a 4 hour period at an operating head considered to be appropriate to the soil and site conditions
4. From a plot of the percolation against time, take the minimum soakage rate from the resulting curve for design purposes (this procedure is mentioned in Portfolio 2).

The design process should recognise that eventually a deep shaft could become clogged, and a replacement shaft must be drilled. The life of the shaft could however be extended by improving the standard of wastewater treatment.

Investigations into the stability of sloping sites and groundwater levels in lower lying sites must be undertaken. In the experience of the writer, for individual household sites, a vertical assessment of the soils and soil moisture would usually suffice. For larger developments however at least two shafts, of at least 200 mm diameter, would be drilled and a falling head test undertaken. The procedure described in point (4) above is followed.

2 Technical Aspects of Deep Shafts and Simpson Contribution

2.1 Percolation Testing

The New Zealand Code of Practice 44:1961 and the US Manual of Septic Tank Practice did not specifically outline how to conduct percolation tests for deep shafts. The writer can add that AS/NZS 1547:2000 also does not cover a testing procedure for deep holes.

2.2 New Zealand Deep Shaft Survey

Questionnaires were sent to 23 local authorities known to be using deep shafts, extended from the Bay of Islands in the north to south of Lake Taupo in the centre of the North Island. Many of the councils involved had subsoil types typical of the upper North Island, which consist generally of clays, volcanic ash/pumice, volcanic loams and alluvium. About 60% of the questionnaires were completed and returned.

A typical questionnaire and response given is in Table 7.2.

Table 7.2: Typical Questionnaire and Responses (Matamata County, NZ)

Number	Question	Response
1	Is deep shaft disposal system approved in your area?	Generally yes
2	How long has this system been operating?	About 12 years
3	In what soil types are they most suitable?	Most soil types, although Rhyolite has questionable infiltrative value.
4	What system is used to test deep shafts?	Operating experience, no problems to date
5	Do you consider this system has limitations?	Very few limitations but protection of shallow aquifers is very important.
6	Have you had any specific problems – stability, satisfactory performance, drilling.	This method is as problems free as any other method of disposal.

7	Do you require the deep shafts to be backfilled with rocks?	No since this might create the need for an additional shaft.
8	Does the cost of deep shaft disposal compare with other alternatives.	Very much so.
9	Does this system have any specific applications in your area?	A very favourable option for effluent disposal since more effective than others.
10	Other information you can offer to assist the research project.	Without doubt the most favoured effluent disposal system in use in this County. There have been some minor wall collapses but operational efficiency has not be impacted on.

Notes:

1. Survey conducted by Matamata County Council, NZ, March 1976.
2. The writer considers that the minor wall collapses would have been avoided if shafts backfilled with 200 mm rocks.

The questionnaire gave councils the opportunity to offer additional information and experiences stated in the survey. These responses included the following:

1. Deep shafts can operate under higher heads which enhance infiltration
2. Experience in the Marton/Rangitikei area showed that deep shaft disposal proved to be successful in areas where shallow trenches had failed
3. Porous concrete liners were used near the surface if there was any concern about erosion and instability
4. Alternating twin deep shafts were used in the Tauranga area regulated by a distribution box
5. In some overloaded effluent systems, in developed areas, it was more convenient to install one or more shafts rather than install more effluent trenches.

2.3 Drilling Operator Surveying

The rotary drilling contractors, *Fenwick Drilling*, had installed deep shafts for many years in varying subsoil types. They desired to have systems surveyed to acquire the following information:

1. The degree of acceptance of this effluent disposal technology by various local authorities
2. How deep shafts compared technically and cost wise with other disposal alternatives
3. Was the rotary drilling action tending to seal the sides of the shafts and for how long

4. Had there been any groundwater contamination problems
5. Had there been any site stability problems
6. The degree of acceptance by general public
7. The potential for the approval of additional deep shafts
8. The potential of deep shafts for larger than individual homes.

2.4 Field Drilling Procedure

As a result of the survey findings the writer developed the following procedure to be undertaken during the deep shaft drilling:

1. Record changes in strata as a bore log – soil type, colour, structure
2. Estimate moisture content (%)
3. Note the presence of roots
4. Note the presence of cracks
5. Estimate clay content
6. Estimate particle sizes
7. Estimate soil porosity. The pore diameter was estimated by comparing with an object of known diameter (NZ Soil Bureau Bulletin 25 – Soil Survey Method).

This field procedure has been modified since the introduction of AS/NZS 1547:2000, the former procedure has been covered in Section 5.2.

2.5 Laboratory Procedure

Soils can be analysed in the laboratory to determine their ability to accept effluent. This can involve the determination of texture, structure, particle size, pore size, effect of swelling, clay content and mineral content. This analysis would be more suitable for the design of small and medium sized communal effluent systems. The individual property owner could not be expected to bear the cost of the time and resources laboratory testing would involve.

2.6 Empirical Approaches to Estimate Permeability

All permeability formulae depend on the assumption that the permeable strata are entirely homogeneous. This does not often relate to the field situation.

Darcy's Law can be applied to permeability or infiltration rates.

$$Q = KiA \qquad \textit{Equation (5)}$$

Where K is the coefficient of permeability (m/s), i is the hydraulic gradient and A is the total cross sectional area (cm^2). Q and i can be obtained by testing an undisturbed soil sample with a constant head permeameter.

2.7 Use of Deep Shaft Disposal – Whangarei County Council, NZ

During October 1974, the Northland Catchment Commission and Regional Water Board resolved that the disposal of septic effluent was acceptable under certain conditions. The deep shafts should not be located near shallow unconfined aquifers which could be suitable as a potable water supply. The method of effluent disposal was more applicable in ground with lower permeability soils.

Based on the writer's survey experience and knowledge of soils within the Whangarei County Council, the situations where deep shafts have a definite application are as follows:

1. In sloping country, generally accepted as being stable and with known lower groundwater levels
2. As a viable disposal alternative to conventional trenches and beds in smaller and awkwardly shaped properties
3. Where existing trench and beds systems in developed areas are overloaded or require to be extended. Installing one or more deep shafts cause less disturbance to the landscape
4. For the distribution box application, as used in Tauranga County Council. In most cases two shafts sufficed. The additional shaft could be used as a reserve or all shafts used for alternating effluent dosing and resting
5. The layout adopted by the former Whangarei City Council could be applied, as shown in Figure 7.1.
6. In areas with lower permeability soils. As stated earlier deep shafts often operate better under higher heads. A typical section for a deep shaft is in Figure 7.2.

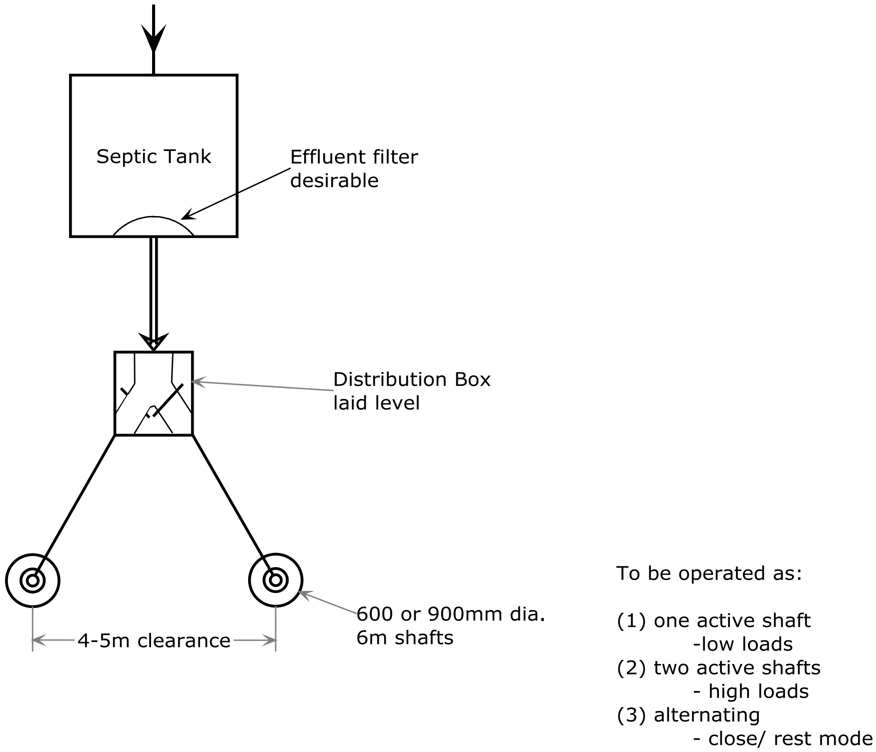
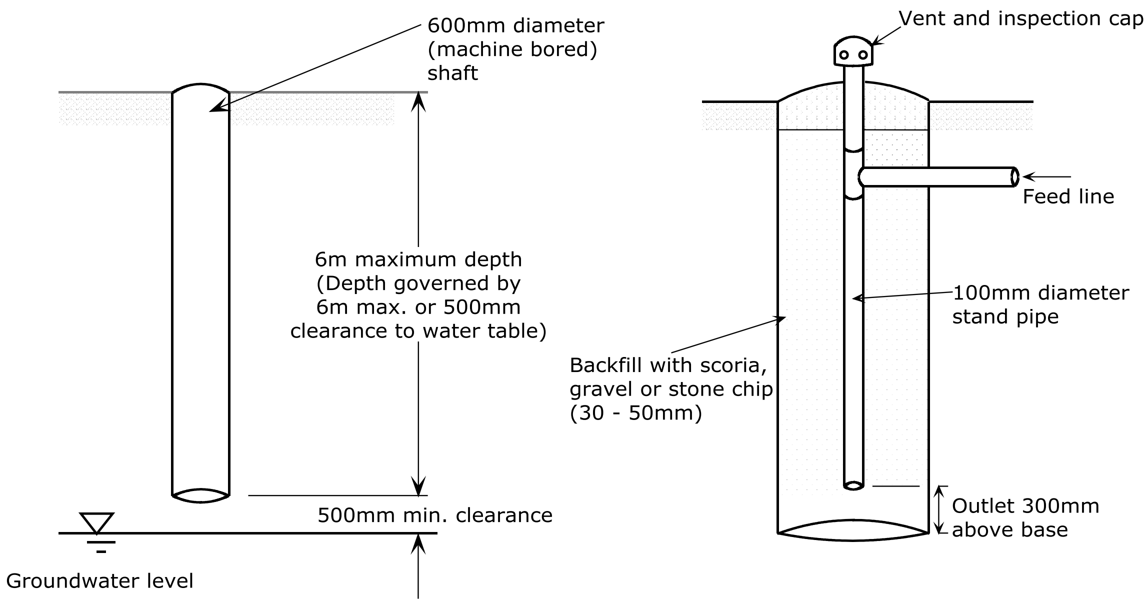


Figure 7.1: Typical Layout of a Deep Shaft



Notes: 100mm diameter stand pipe to be slotted
Media backfill to extend to the bottom of the shaft

Figure 7.2: Typical Section of Deep Shaft

2.8 Evaluation of Deep Shaft Disposal – University of Auckland

As a response to correspondence from the Northland Catchment Commission, Whangarei, an evaluation of deep shaft disposal was undertaken by Ian W Gunn, University of Auckland (Gunn, 1980). The salient points of this evaluation are as follows:

1. There is no doubt that the septic tank/deep shaft system is a tidier solution for on-site effluent disposal than conventional shallow trenches or evapotranspiration/infiltration (ETA) trenches
2. Further treatment or conditioning of the effluent by retention, stabilisation and modification of the properties of the wastewater will occur to varying degrees within the infiltrative interface and the surrounding sub-soils, under saturated and unsaturated flow conditions
3. The build up of head within the shafts provides increased infiltration hence, deep shafts can respond well to continuance of performance during temporary overload or slow down in slime stabilisation during colder seasons
4. When deep sub-soils are more free draining the potential for developing pore-pressure instability is significantly reduced
5. Only one deep shaft stability failure was known. (The writer reported one failure to Ian W Gunn)
6. Desirably two deep shafts should be installed to enable alternate dosing
7. Given that solids carry over from the septic tank and appropriate design and installation, it was felt that deep shafts were a very sustainable system.

The writer concurs with the Gunn (1980) evaluation of deep shaft technology.

3 Challenges with Deep Shaft Disposal

3.1 Groundwater

It was noted in the survey response that deep shaft disposal can be responsible for contaminating groundwater. It is imperative that potential water resources be safe guarded. The writer can recall in the 1970s and early 1980s that a clearance from the base of effluent disposal systems was at least 0.6 m from the groundwater level. Bouma (1975) stated that a clearance of 0.9 m was accepted practice, provided the soil was not overloaded. The unsaturated zone below the effluent disposal system should be maintained to achieve further purification and the removal of pathogenic bacteria and viruses.

Groundwater levels can fluctuate with seasonal and tidal changes. With respect to the deep shafts installed prior to the survey, it was accepted practice that they were bored down to within 1.0 m of the known ground water level during the wet season. By experience, operators generally knew when this level was being approached during the drilling process. The clearance between the bottom of the deep shaft and the groundwater depended mainly on the effluent quality.

In the event of a deep shaft being drilled to the groundwater level, which did happen in the writer's experience, the choices taken were:

1. Abandon the deep shaft
2. Partially backfill the shaft and pour a plug of weak mix concrete.

Deep shafts are a very similar disposal concept to seepage pits, as described in USEPA (1980). They are a chamber, up to 5 m deep constructed of brick or block with open joints embedded in clean rock. The suggested clearance to the water table in unsaturated soil is 1.2 m (USEPA, 1980).

The issue of developing minimum clearances of varying qualities of effluent from groundwater tables has been undertaken by Barnett and Ormiston (2010). This work is within the Horizons Regional Council, in the lower North Island of New Zealand. Refer to Table 7.3 for groundwater clearances.

Table 7.3: Minimum Recommended Effluent Separation Distances from Groundwater

Soil Category (2)	Primary Tank Standard (including effluent filter)	Secondary Standard	Advanced Secondary Standard
1	Note (1)	1,500 mm	1,200 mm
2-3	1,500 mm	1,200 mm	900 mm
4-6	1,200 mm	900 mm	600 mm

Notes:

1. Measures must be made to slow the soakage rate
2. As per AS/NZS 1547:2000, summarised as Category 1 – sands and gravels, Category 2 – sandy loam, Category 3 – loam, Category 4 – clay loam, Category 5 –light clays and Category 6 – medium and heavy clays

It can be seen from Table 7.3 that between 1,200 to 1,500 mm clearances are suggested for a primary effluent, which is close the USEPA (1980) figure of 1.2 m.

3.2 Deep Shaft Side Stability

Comments on soil stability potential of deep shafts, sourced from the School of Engineering, University of Auckland (Gunn, 1980) are as follows:

1. Sub-soils can become unstable if effluent is allowed to accumulate in a manner that will increase pore pressures along an interface. This results in uplift, subsequent adhesion failure and then sliding
2. When sub-soils are free drained, the potential for developing pore pressure instability is significantly reduced
3. Deep shafts are sited in upper soils with low K values entering into lower soils with higher permeabilities. Hence, if soil tests show more moderate K values then sub-soil stability is likely to be assured.

The technical literature on deep shafts as a disposal system is very limited. In the writer's opinion this is due to the fact that they were developed in the North Island of New Zealand and they have been confined there. The postal survey of effluent systems that the writer undertook in Australia, reported in Portfolio 5**,** does not mention this system.

3.3 Potential Problems with Deep Shafts – Northland Catchment Commission

A paper on the potential problems associated with the disposal of septic tank effluent in deep trenches and bores was presented by Griggs (1981). In Northland, New Zealand the clay soils swell, dry and crack due to changes in sub-soil moisture. This cracking can admit large amounts of surface water to deeper horizons which are one of the basis mechanisms for slope instability. These conditions on sloping sites can apply to deep disposal shafts. However, the designs for deep shafts do not allow the entry of surface water, thus minimising ground pore pressure and protecting slope stability. Surface water should be diverted away from the deep shafts. Griggs (1981) advocated that when approving deep shafts the following conditions should be met:

1. There should be no water supply bores or wells in the area
2. Effluent must not enter the groundwater table. This can be determined during the drilling operation. The writer found that when the soil moisture changed, to say semi-saturation, the drilling was likely to be approaching the groundwater level. At times an indication of the groundwater depth can be sought by local knowledge or from other drillers
3. Deep shafts should not be installed in hilly terrain where there is the likelihood of surface seepage appearing down slope, which could trigger landslides or slips

4. Any development site where the wide scale use of deep shaft systems is contemplated should be assessed as if all the proposed earthworks have been completed, to fully understand what the site will ultimately look like.

3.4 Microbial Contamination and Groundwater Mounding

A study on the Canterbury Plains of New Zealand included work on contamination for 4.0 to 6.0 m deep soakage pits into an unconfined aquifer (Sinton, 1985). Faecal coliform bacteria moved 9.0 m horizontally, partial sealing of the side walls was evident but approximately 80% of the effluent percolated into the unconfined aquifer, via the unsaturated zone. Of further interest there was groundwater mounding beneath the soakage pit. Groundwater mounding beneath the disposal system is also reported by Healy and Laak (1974). In the writer's opinion, this raises the point that effective clearance should take into account this groundwater mounding. Refer to a discussion on this topic in the Reflections Section.

3.5 Lower Permeability Situations

Deep shafts were found to be operating satisfactorily in some areas with low permeability sub-soils. It was found that changes in strata within the depth of the shaft, offered seepage paths for effluent disposal. The additional operating head improved the infiltration rate, if there was some degree of soil permeability, say at least 0.15 m/d.

4 Deep Shaft Construction Aspects

Deep shaft constructions aspects have been sourced from the response to the survey and the experience of drillers, plumbers and the writer.

4.1 Deep Shafts in Unstable Sand

Porous concrete well liners were used in locations with unstable sand. These locations include river plains, tidal estuaries and dune country. The installation procedure was as follows:

1. Bore a shaft of at least 1.0 m diameter
2. Place one porous liner and continue boring inside this
3. Place a second porous liner and continue boring inside this
4. The situation may be reached when the boring rig is no longer effective due to the high moisture content of the sand
5. The excess wet sand can then be removed by a sludge pump.

4.2 Deep Shaft Backfilling

The stability of the side wall of the shafts was questioned at times. An unstable wall could collapse and consequently render the disposal shaft useless. All the shafts were either backfilled with hard rocks or porous liners were put in place. Consequently, it was the suggestion of the writer that deep shafts were not to be left un-backfilled or without liners. Rocks of at least 200 mm were used since they would allow a greater side wall area and provide larger voids.

4.3 Stability on Sloping Ground

The location of deep shafts on sloping surfaces was carefully investigated. The depth to weathered rock and hard rocks were determined. It was appreciated that there was little point in penetrating too far into the weathered rock zone. This may cause instability and the possible formation of a slip circle. It was also appreciated that a change in sub-surface water conditions could cause instability. The minimum spacing between deep shafts was 6.0 m.

4.4 Condition of Side Walls

It was generally stipulated that deep shaft side walls were scarified to help provide a natural soil interface for the infiltration of effluent.

Experience in the North Island of New Zealand had shown that infiltration (as gauged by observing the drop in water level in the shaft over a number of days) tended to improve after 2 to 3 days. This indicated that the drilling action could have compacted the deep side wall and it took 2 to 3 days for this action to be rectified.

5 Survey of Existing Deep Shafts in Northland, NZ

The writer undertook a field survey over a very wet period of August to September 1976 and the results are in Table 7.4. The deep shafts had been operating within a range of soil types for up to 30 months.

Table 7.4: Results of Deep Shaft Survey in Northland, NZ

Date	Location	Soil Type	Shaft Numbers	Av. Depth (m)	Time of Operation	Comment
18/08/76	Ruatangata, Whangarei	Volcanic	2	9.7	12 months	2 shafts served Golf Club facilities. No sign of overloading.
19/08/76	Ruatangata, Whangarei.	Clay + shale	2	5.5	20 months	Single deep shaft taking most loading. No side wall failure evident.
19/08/76	Kauri, Whangarei	Volcanic	1	5.8	24 months +	Heavy loading on I deep shaft, no signs of failure.
30/04/76	Haruru Falls, Bay of Islands	Silty clay	2	4.7	22 months	One deep shaft had collapsed to 4.7 m depth, otherwise functioning well under higher loading.
30/08/76	Haruru Falls, BOI	Silty clay	2	9.1	24 months +	No problems evident.
30/08/76	Paihia, BOI	Silty clay	2	6.0	24 months	Partial collapse, no performance failure evident.
30/08/76	Paihia, BOI	Silty clay	2	7.0	12 months	Partial collapse, no performance failure evident
14/09/76	Vinegar Hill, Whangarei	Silty clay	2	7.6	24 months	Heavy loadings, no failure evident.
14/09/76	Kamo West, Whangarei	Volcanic	1	6.1	12 months	Household operating well on single deep shaft only.
28/08/76	Parua Bay, Whangarei	Red clay	2	6.7	12 months	Single deep shaft taking most of the loading, no problems evident.
1/09/76	One Tree Point.	Sandstone	Scheme for 20 houses	9.1	13 months	A temporary scheme under constant supervision. No clogging problems evident.

All deep shafts were not backfilled (an accepted earlier practice) except for the project at One Tree Point, which were lined with old drums. Since some deep shafts had partially collapsed the value of backfilling with rocks was appreciated. A minimum of two deep shafts was recommended by the writer, for alternative dosing and resting when catering for peak field and peak load conditions, as a result of this survey.

6 Deep Shaft Case Studies

6.1 Waiheke Island, NZ Feasibility Report

The writer undertook a feasibility report on sewage collection, treatment and disposal alternatives for Waiheke Island, near Auckland in New Zealand (Simpson, 1984). The feasibility process was to initially classify the Waiheke Island sewerage needs into the categories of individual residential lots; larger facilities as sporting clubs, community halls and schools; groups of residential or commercial properties experiencing problems; a community as a whole and the island as a whole. The second step was to identify the water quality problems. The third step was to select options for wastewater collection and rank these. The fourth step was to select treatment options and rank them to determine the most viable. The fifth step was to select options for effluent disposal and rank these. The final step was to draw conclusions and prioritise the more favourable options for collection, treatment and disposal.

Deep shafts ranked the best alternative, as shown in Table 7.5.

Table 7.5: Ranking Alternative Effluent Disposal Methods – Waiheke Island, NZ

Alternatives					
Factor	**Deep Shaft**	**Trickle Irrigation**	**Spray Irrigation**	**Shallow Trenching**	**Wetland**
Effluent quality	2	3	3	2	3
Design input	2	4	3	2	2
Energy requirement	1	3	4	2	1
Mechanical requirement	1	3	3	2	1
Capital cost	3	2	4	3	2
Operating cost	1	4	4	3	2

Reliability	2	3	3	2	3
Sitting to residential development	2	5	5	3	5
Acceptability	2	5	5	3	4
Extension potential	2	3	1	2	2
Climatic suitability	3	5	5	4	3
Peak load capability	5	3	2	5	2
Monitoring, operation and maintenance needs	3	5	5	4	4
Totals	29	48	47	37	34

The ranking is in order of merit, 1 being the highest ranking and 5 being the lowest. It can be seen from Table 7.5 that:

1. Deep shafts have the highest ranking – most suitable all-round system
2. Very favourable factors include low energy requirement, low mechanical requirement, and low operating costs
3. Other favourable factors include low design input and reliable system, can be sited near to residential development, readily extended and acceptable effluent quality for the receiving environment
4. Monitoring and operation and maintenance needs are acceptable.

6.2 Titirangi Park Estate, Auckland - Deep Shaft Disposal Project

Earlier studies of effluent disposal at the Titirangi Park estate established that shallow soakage was an unsuitable alternative for most of the lots. The alternative of using ET trenches was also ruled out based on the relatively steep terrain and high rainfall in the area. The writer was engaged by *Beca Carter Hollings Ferner Ltd*, Auckland as a specialist consultant, on the advice of Ian W Gunn, University of Auckland. Extensive site testing showed that sufficiently permeable strata of unweathered Manukau Breccia existed below the clays. Percolation testing was undertaken in 200mm diameter bores over four hours. A standard deep bore configuration was developed for the Titirangi Park Estate and is shown in Figure 7.3.

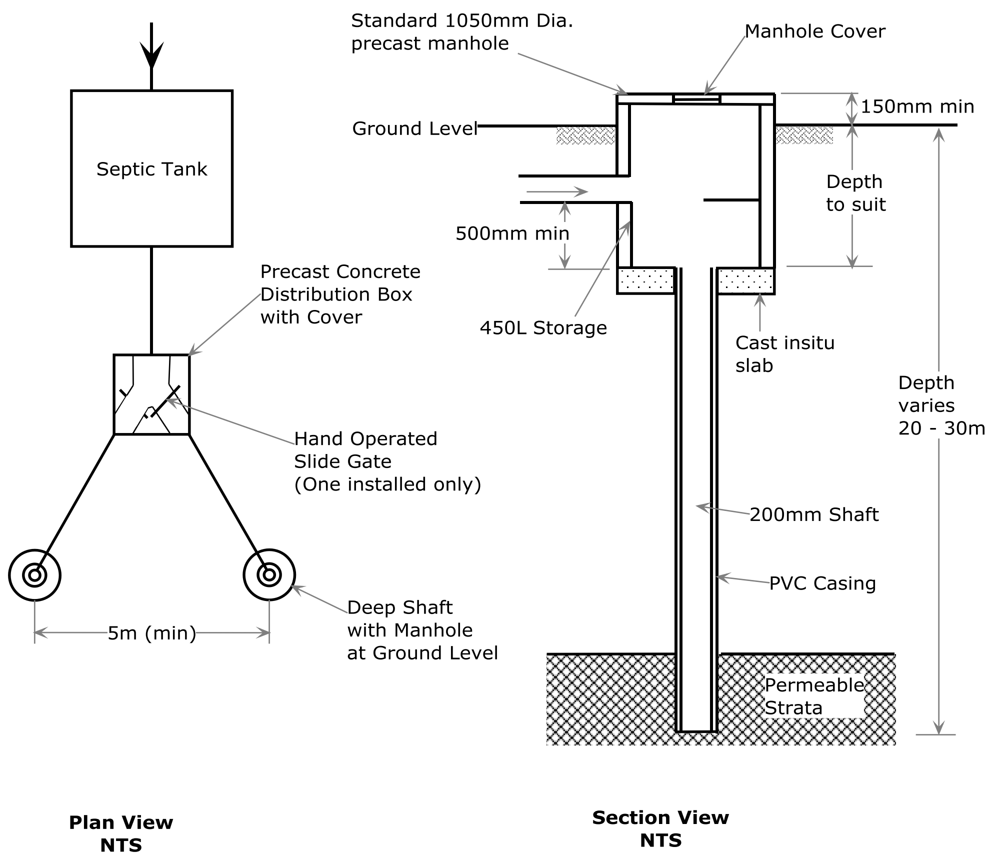


Figure 7.3: Details of Titirangi Estate Standard Deep Shaft

The experience of this study of deep shafts (Simpson, 1976) was utilised and a report was undertaken for effluent disposal on the estate (BCHF, 1976).

7 Reflections and Recommendations on Deep Shaft Disposal

7.1 Enhancing Biological Treatment

The emphasis on the use of deep shafts has been on the disposal of effluent, by utilising various soil horizons and seepage paths. The writer sees this as being a major advantage for the use of deep shafts.

On reflection, these deep shafts offer the opportunity for biological treatment by the means of trickling filtration. The slow downward movement of effluent will soon develop a biomass around the rocks for biological breakdown. Depending on the availability or non-availability of oxygen, the treatment environments will be either aerobic or anaerobic. Degrees of potential effluent treatment have been reported by Foster et al (1994).

These processes can be enhanced by consideration of the following:

1. Providing air access, by installing a vertical slotted PVC vent
2. Possible nitrification within the vented upper section of the deep shaft
3. Using large rocks with a high percentage of voids for the upper section, which will act as a roughing filter
4. Using 20-30 mm gravel within the lower section with a porosity of about 45%, for acting as a secondary filter by increasing the retention time and possibly achieving de-nitrification
5. Placing a layer of wood chips about mid-depth, as a carbon source for de-nitrification.

By reflecting over the technology developments since 1980, the effluent disposal options rating table presented in Table 7.4 is still applicable today.

7.2 Waiheke Island, NZ Study

If the writer was to undertake the Waiheke Island study in the present day, the range of alternatives could change but it is likely the rankings would be similar. The study by Simpson (1976) concluded that deep shaft disposal was suitable for many areas provided that some degree of soil permeability was available; that all the shafts were lined or backfilled with rocks; that there was no danger of polluting underground drinking water; and in situations with acceptable stability.

It was also found that through extensive drilling experience that the drilling action often temporarily sealed the side walls. Testing over a number of days following the drilling showed an improvement in infiltration. (Summary statement by Simpson, February, 1984).

7.3 University of Auckland Evaluation of Deep Shafts

Thinking back the evaluation by Ian W Gunn, University of Auckland on the deep shaft disposal alternative was well conducted. The distinct advantages were presented, which included the following:

1. Very suitable for confined sites
2. Operating two deep shafts on an alternating dose rest sequence
3. Additional in-situ stabilisation of the effluent takes place.

The important limitations of deep shafts were also included. The location of deep shafts on sloping and potentially unstable sites was noted. Locating the base of deep shafts clear of the groundwater resource was also noted.

7.4 Deep Shaft Testing Procedure

The original testing procedure developed by the writer following the deep shaft survey (Section 5.2) has been revised as a result of the introduction of AS/NZS 1547:2000. On reflection, the writer recommends that following aspects should be identified when testing deep shafts for effluent disposal:

1. Soil profile
2. Soil moisture assessment, as per AS/NZS 1547:2000
3. Clay content assessment, as per AS/NZS 1547:2000
4. Presence of cracks and roots (seepage paths)
5. Observe for signs of soil saturation, indicating the possible groundwater table
6. Conduct a falling head test as described in Portfolio 2 and graph the falls against time, to identify the gradient for the design infiltration.

7.5 Depth to Groundwater

It is interesting to note that the former Australian Standard (AS 1547-1994) on disposal systems did not state any clearance figure of disposal systems to the groundwater.

The Queensland Plumbing and Wastewater Code, 2008 specifies 1.2 m for primary effluent and 0.6 m secondary effluent separation from unsaturated soil depth to a permanent water table.

It can be debated that the “safe” clearance of deep shaft bases from the standing groundwater level is probably better not stated as a definite distance for the following reasons:

1. Standing groundwater levels can fluctuate, depending on rainfall and seasons and distances to water bodies
2. The quality of effluent must be taken into account. For primary effluent the clearance should be maximised, to avoid water pollution
3. Subsoils change in texture and clay content so clearance needs to be adjusted accordingly
4. The selection of a minimum clearance should be site specific.

Many soils have seasonal groundwater levels. This situation can interfere with the hydraulic mechanisms since additional liquid from the seepage system will tend to accelerate any natural rise in the water table.

Sinton (1985) reported a study of deep soakage pits for effluent disposal on the Canterbury Plains, New Zealand. This showed that mounding occurs under the pits, and as a consequence there is a radial spread of effluent. This factor must also be considered when deciding on effective clearances to groundwater. On reflection, the 1.0 m clearance would have more suitable for a secondary standard of effluent, and 1.5 m more suitable for primary or septic effluent.

8 Conclusions

Technical Literature

The technical literature available on deep shaft disposal is very limited. This is partially due to the fact that it has been an effluent disposal alternative which has largely been restricted to the North Island of New Zealand for many years.

Deep Shaft Survey

The deep shaft survey was generally considered to be a very useful exercise since this disposal technology has been confined to specific regions with the North Island of New Zealand. The deep shaft survey highlighted the following aspects or needs:

1. For the deep shafts to be backfilled, to avoid collapse or partial failure
2. To install at least two deep shafts for domestic applications, to allow for peak loadings and site conditions and to have the benefit of sequential dosing and resting
3. Very suitable for confined sites
4. A viable alternative for where there are upper clay soils and deeper more permeable soils
5. A viable alternative where changes in stratum with depth offer effluent seepage paths
6. The rotary drilling action can seal the sides of the shafts. Water testing over some days showed that this temporary seal was relieved and infiltration improved
7. There where shallow clay soil types, as clays, in some regions that showed the trenches were not likely to be sustainable, deep shafts were considered to be the best technology.

Specific Applications for Deep Shaft Disposal

Deep shafts have a particular application for the following situations:

1. On confined sites where the land available is restricted
2. Where the upper soils are not suitable for conventional trenches or beds
3. Where groundwater contamination is not a risk
4. Where slope stability is not a risk.

Deep Shaft Testing

The revised deep shaft testing procedure described in Section 10.4 is considered to be appropriate. The writer has found that deep shafts for individual homes can be assessed, without the need for water testing. However, for small institutions and groups of houses it is advisable to include water testing.

Additional Treatment Potential

The writer concurs with Foster et al (1994) that ongoing biological treatment within the body of deep shafts takes place. This would apply particularly to deep shafts receiving primary effluent where there is the opportunity for nitrification and denitrification.

Viability of Deep Shaft Disposal

This technology is simple and robust. It has a particular application in areas with low permeability upper soils. It is imperative that slope stability is preserved and the contamination of groundwater avoided.

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Part (B) Sand Mounds

1 Introduction, Aims, Objectives, and Historical Development

The overall aim is to study the performance of sand mounds as a wastewater treatment and effluent disposal alternative. The objectives of Part (B) of this Portfolio are to:

1. Outline the historical development of sand mounds, as an effluent disposal system
2. Report the past work and more recent work undertaken by the writer on sand mounds, which includes two case studies
3. Review the work by other researchers, including the review the specific work undertaken at Salt Ash, New South Wales (Whitehead and Geary, 2009)
4. Develop a model sand mound for South East Queensland by assessing all the design information and performance data of sand mounds in similar climatic conditions
5. Reflect on past and more recent developments
6. Draw conclusions.

Slowly permeable soils with seasonal high water tables and shallow bedrock cannot be used for conventional trenching and bedding. The sand mound has an application in such situations. Sand mounds are, in effect, like intermittent raised sand filters with no base. They are pressure dosed at a frequency which depends on the pump chamber storage and the household wastewater loading.

Mounds, known as *NODAK* systems, were first developed by the North Dakota Agricultural College in the late 1940s (Crites and Tchobanoglous, 1998). This is a shallow mounded system inserted into the ground some 300 mm to provide contact with the subsoil. The system is utilised in clay and clay loam texture soil on flat sites (Gunn, 1989). Slowly permeable soils with seasonal high water tables and shallow bedrock cannot be used for conventional trenching and bedding.

To overcome poorer soils, perched water tables and shallow rock constraints, the absorption area was raised above the natural soil, by using suitable fill material and sand. Firstly, this would allow the percolating effluent to enter more permeable soils in a lateral direction. Secondly, the biological slimes that would normally develop at the seepage trench bottom will not clog the sandy fill to the degree it would in the natural soils. Finally, no construction is undertaken in the subsoil where smearing and compaction is often unavoidable (Converse et al, 1976)

AS/NZS 1547:2000 states that raised mounds are suitable for the following applications:

1. Slowly permeable soils
2. Permeable, but shallow soils over creviced or porous bedrock
3. Permeable soils with high water tables.

Approximately 50% of the soils in the State of Wisconsin are unsuitable for conventional effluent disposal systems. Because surface disposal and earlier work with evapotranspiration systems were not successful, four experimental mound systems were set up and tested under the Small Scale Waste Management Project (SSWMP), University of Wisconsin (Bouma et al 1975). The objectives were to achieve:

1. Continuous underground disposal of effluent from a septic tank
2. Percolation of effluent through a sufficient volume of soil to achieve purification, before any possible human contact with the reclaimed water.

Tyler (1984) reported that an earlier research mound failed at the toe. It was found that the original grass cover and mound soil interface created a clogging matt (Bouma et al, 1975). Converse (1978) recommended ploughing the surface which solved the problem. He also compiled a mound manual. The soil and site characteristics included:

1. Greater than 600 mm from highest water table level
2. Depth to bedrock greater than 1.5 m
3. Slopes less than 6%
4. Soil permeabilities lower than that allowed for trenching.

The writer has inspected sand mounds with unplanted surfaces, partially planted and fully planted. The planting is obviously advantageous, as reported in Portfolio 5. One sand mound was covered with dead plants, due to over-design (as catered for one elderly lady only). Presumably, the larger surface of the sand mound and the ET action had resulted in planting that could not be sustained. Converse et al (1976) advocates that moisture tolerant shrubs should be selected for the toe area of the mound as the soil in this area will be quite moist most of the year. In the opinion of the writer, this aspect is worthy of inclusion in the design of future sand mounds.

The Ellesmere County in New Zealand developed a fill system which has an application on flat land with high water tables (Gunn, 1989). This is constructed by stripping the topsoil over an area of 250 m^2, backfilling with clean pit run river gravel to a depth of 300 mm above the surrounding ground level and using the excavated topsoil to cover the whole system. It has a gravity distribution system which is flooded by an effluent pump.

A modified sand mound was developed in Georgetown, California by Borgerding

(1998), which involved several design modifications and changed operation criteria, which resulted in a lower failure rate and more long term reliability.

The Maryland Department of Environment studied 36 sand mound systems. Prager and Glotfelty (1994) reported that the factors affecting sand mound performance included infiltration, sand size, loading rates, surface preparation, mound orientation, planting, depth to a limiting horizon and soil permeability.

Converse and Tyler (2000) report that designs should include the following components:

1. Loading on the manifold, in terms of m/d
2. Basal loading
3. Linear loading, the most critical factor to avoid breakout of effluent.

2 Sand Mound Work by Simpson

2.1 Background

The writer's early work with sand mounds in the late 1970s and early 1980s was in Northland, New Zealand. The writer recognised, from work by researchers Converse and Bouma, that sand mounds were initially designed for the following applications:

1. Applications where the groundwater level was high hence, there was less than sufficient clearance between the groundwater table and the bottom of the sand mound
2. Where rock existed near the surface or where rock dominated the soil profile below the sand mound site.

There were no recognised sand mound standards in New Zealand or Australia at the time so the writer's design approach was as follows:

1. Treat the sand mound as fully reliant on the disposal mechanisms of evapotranspiration (ET). Any available soil permeability was considered to be a factor of safety
2. Refrain from locating a sand mound on sloping ground, to minimise effluent leakage
3. Dose the sand mound by pump action. Pumping is necessary since the mounds are raised above the septic tank outlets. An alternative is the tipping bucket method
4. Grass and plant the surface, to enhance ET
5. Add a factor of safety of at least 1.2 when designing the basal area
6. Adopt the other design criteria on the North American practices and operational experiences

7. Undertake some assessment of the basal areas by comparison with case examples with matching or similar climates in North America.

The writer's main concern, other than under-designing, was the escape of effluent from the sides and the toes, during less conducive disposal and weather conditions. To overcome this possible problem the writer incorporated a compacted clay fringe around the sand mound perimeter. The writer has since noted that Converse et al (1976) suggests that fringe planting is undertaken, to minimise this possible problem.

2.2 Sand Mound - Woodford Case Study

An interesting design problem arose recently when in Woodford, South East Queensland a sand mound had been designed, as per AS/NZS 1547:2000, for a house with two bedrooms on town water supply. The writer investigated the sand mound and it was operating well with four persons in the house. The plan was to extent the house by the addition of three more bedrooms. This would have overloaded the sand mound but extending the sand mound was not practical.

The solution was to investigate the soil profile at the end of the mound. This revealed sandy loam to 300 mm, into clay loam to 500 mm, into sandy clay plus weathered rock to 1,000 mm. A trial falling head percolation test gave a fall of 190 mm in 10 minutes. The limitations of falling head percolation testing are appreciated, as outline in Portfolio 2. However, the result was certainly indicative of a higher soil K value, at least 1.2 m/d. The approach was then as follows:

1. At the end of the sand mound locate the invert level or floor level of the mound
2. Excavate a trench 4.0 m long, 1.5 m wide and 1.0 m deep and back fill with 20 - 40 mm clean aggregate
3. Place the same sized aggregate over the top edge of the trench, to allow effluent build up within the sand mound to seep into the trench
4. Place topsoil of the trench.

This simple overflow trench would minimise any build up of effluent within the sand mound and prevent any break outs on the sides and toes of the mound, which can normally be difficult to repair.

2.3 Sand Mound Monitoring – Morayfield Case Study

To the writer's knowledge very limited work on the performance of sand mounds has been undertaken in Australia. This has been confirmed by Whitehead and Geary (2009) who have stated that there is a growing interest in the application of sand mounds in Australia, but to date little has been published on the performance of such mounds in Australian settings.

AS/NZS 1547:2000 sets out construction details for Wisconsin raised mound systems. Typical horizontal and vertical dimensions and slopes are given.

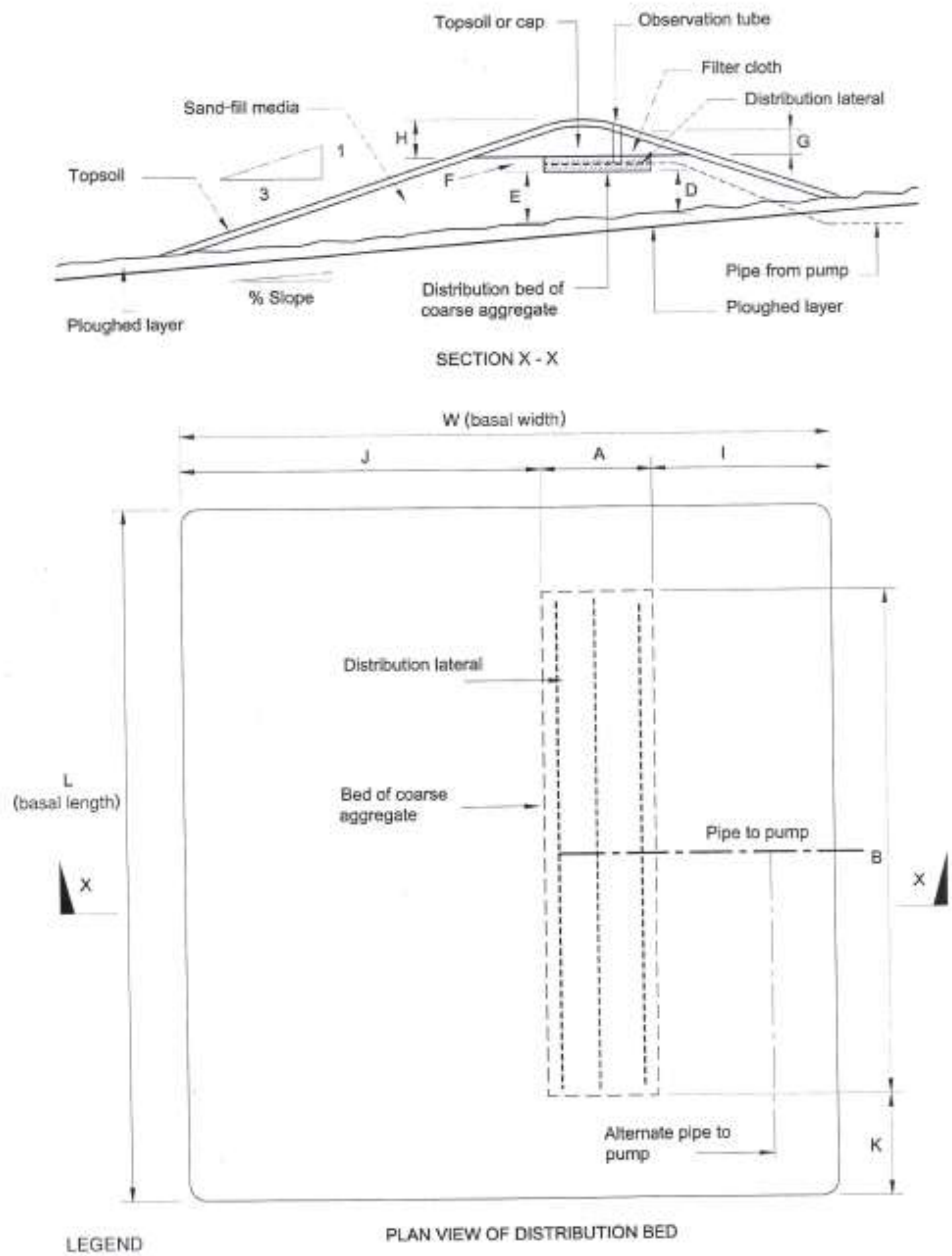
To demonstrate the performance of a raised mound in Morayfield the writer undertook a case study in 2009-2010. In this case the site was within the Caboolture River water supply catchment, which discounted the option of septic tank treatment and trench or bed disposal. The owner also preferred the raised sand mound option to effluent trenching. Design criteria and construction guidance is given in AS/NZS 1547:2000. To the knowledge of the writer a mound system has not been tested in an Australian sub-tropical environment, in terms of sizing and performance.

The case study design criteria included:

1. Loading based on a 4 bedroom house
2. Town water supply based on 120 L/person/day, using standard water saving devices (slightly more than AS/NZS 1547:2000). Total flow per day 792 L (6 persons x120 L x 1.10 peak factor)
3. Pre-treatment using 3,900 L capacity septic tank, with an effluent filter
4. Design loading rate (DLR) equals 8 mm/d
5. Design basal area equals 120 m^2
6. Effluent dosing system by pumping.

The basal area and upper distribution area were based on AS/NZS 1547:2000 but the writer also considered the outcome of a recent workshop, involving South East Queensland plumbing officers (Brian Boreham, Caboolture, pers. comm., 2009). The main concern with raised sand mounds, which have limited underlying soil permeability, is insufficient basal and mound surface areas.

The mound cross sectional detail from AZ/NZS 1547:2000, reproduced in Figure 7.4, shows the dimensions and bed materials. In this instance the surface was not scarified but the grass cover was removed. The mound surface was planted in natural grasses, to enhance evapotranspiration loss.



LEGEND

Typical dimensions:

A 1200 to 2000 mm

B 6 to 8 times A

D 600 mm

E 600 mm on flat ground, >600 mm on sloping ground

F 225 mm

G 300 mm

H 450 mm

I Determined by ground slope and 1 in 3 mound face slope

J 2000 mm minimum on sloping ground (equals I on flat ground)

K Determined by height of finished mound and 1 in 3 mound face slope

Figure 7.4: Section of Sand Mound (NTS)

The typical soil profile is gritty loam, into clay loam (220 mm), into silty clay, with friable silty clay at 950 mm. This is a soil profile unsuitable for conventional effluent trenching due to the lack of at least 200-250 mm of free draining sandy loam or loam. It is more suitable for a raised sand mound. The AS/NZS 1547:2000 recommends scarification of the upper soil to at least 200 mm. In this case the writer suggested no scarification so the upper 220 mm of loam and clay loam could be fully utilised as the infiltrative surface.

Sampling of the final effluent quality started in January 2010. The effluent results are in Table 7.6.

Table 7.6: Sand Mound Effluent Monitoring Results – Morayfield Case Study

Date	BOD_5	Suspended Solids	NO_X - N	TKN	Total N	Comments
14/01/10	11.0	19.0	37.9	39.0	76.9	Rainfall limited to 63 mm 14 days before sampling
19/02/10	<3.0	7.0	15.2	7.0	22.2	High rainfall 14 days before sampling – 190 mm.
8/04/10	21.0	13.0	2.39	24.9	27.3	Low rainfall 14 days prior to sampling – 23 mm.
6/05/10	15.0	46.0	0.09	46.0	46.1[1]	Low rainfall 14 days prior to sampling – 23.7 mm

Notes:

1. Noted when sampling that the effluent contained a light floc type material, which could have come off the sides of the sampling bore, hence the higher suspended solids result
2. Parameter units are in mg/L.

Comments on the results in Table 7.6 are as follows:

1. The overall results show that the sand mound is capable of ongoing treatment as well being an effluent disposal system
2. It is appreciated that the monitoring is limited to four sampling runs (limited budget)
3. There is no correlation between BOD_5 and suspended solids; longer term monitoring could have shown some correlation
4. There is no correlation in the nitrogen parameters; again longer term monitoring could have shown some correlations

5. The February 2010 result shows some consistency when comparing all the parameter results. This is probably due to the high rainfall and subsequent dilution that took place
6. Generally, the results show that the sand mound is capable of producing a secondary standard of effluent, with one exception
7. Some nitrification has taken place, as indicated by the lower Total N figures
8. NO_X - N results are low but a reason for this cannot be offered
9. TKN results are high
10. The inconsistent performance could be due to the rainfall influence coupled with some intermittent use. The owner goes camping periodically

The project scope and budget did not include influent sampling. A photograph of the sand mound, septic tank and pump chamber is in Plate 7.1. The sampling pipe had been removed by the owner prior to the photograph being taken. It was located on the far side of the mound.

Plate 7.1: Morayfield Case Study Sand Mound

The sand mound results for the Morayfield case study in Table 7.6 have been compared with Table 7.5 and the work by Geary and Gardner (1996) and the following observations have been made, as given in Table 7.7:

Table 7.7: Sand Mound Comparative Monitoring Results

Parameter (mg/L)	Morayfield Case Study (Table 7.6)	Geary and Gardner (1996)	Bouma et al (1975)
BOD_5	<3.0-21.0	1.0-10.0	11.0-13.0
Suspended Solids	7.0-46.0	5.0-20.0	-
TKN	7.0-46.0	30.0-50.0	-
Total N	22.2-76.9	-	3.7-18.0

It was concluded from the Bouma et al (1975) study that several mounds would need to be constructed, of this design, and monitored closely, before these systems could be considered to be successful. The sand mound studied by Geary and Gardner (1996) would have had the benefit of being designed on the basis of systems installed since 1975. The Morayfield sand mound was designed on the basis of AS/NZS 1547:2000, as directed by the former Caboolture Shire Council.

The following observations can be made from Table 7.7:

1. With one exception, the Morayfield study BOD_5 results are within the same order as Bouma et al (1975) and Geary and Gardner (1996) results
2. With one exception, the Morayfield study suspended solids results are within the same order the Geary and Gardner (1996) results
3. The TKN results for Geary and Gardner (1996) are within the same order as the Morayfield study results
4. The Total N results for the Morayfield study are much higher than Bouma et al (1975).

The lack of de-nitrification within the confines of the sand mound, over the monitoring period, is likely to have been due to the fully aerobic conditions within the sand mound. The kikuyu grass roots would ensure an oxygen supply. Rainfall would also supply dissolved oxygen to enhance nitrification and other biological treatment.

2.4 Development of a Model Sand Mound – SE Queensland

The development of a sand mound suitable for South East Queensland conditions has been undertaken as an academic exercise. The development methodology the writer elected to undertake is the following:

1. A review of AS/NZS 1547:2000 – sand mounds
2. Conduct basic monitoring of a sand mound of his design, based on AS/NZS 1547:2000 and his personal experience
3. Review the work of sand mound researchers as Converse, Tyler and Bouma, as reported earlier in this Portfolio
4. Review the work by Joe Whitehead**,** Whitehead and Associates, Newcastle centred on a standard designs for Port Stephens Council, NSW
5. Utilise the Port Stephens Council design, with minor changes, based on his personal experience and findings
6. Develop a model sand mound design, for use in South East Queensland.

Based on AS/NZS 1547:2000 the design details of a typical sand mound in Moreton Bay Regional Council, Queensland, are in Table 7.8.

Table 7.8: Standard Sand Mound – Moreton Bay Regional Council

Design Flow (L/d)	**Gravel Bed Adsorption Dimensions**	**Overall Mound Dimensions (m)**	**Area (m^2)**	**Media**	**Surface Cover and Topsoil**
440	9.0 m length, 1.2 m width, 300 mm depth	5.6m width, 15.0 m length	84	Fill Area: washed gravel of 2-5 mm grain size Distribution Bed Media: 20-40 mm clean gravel	Grass or shallow rooted ground covers

Note:

1. Based design on loam soil with an assessed K sat of 0.65 m/d and a DLR of 12 mm/day.

Comments on Table 7.8 are as follows:

1. Based on restricted water supply, so design flow is 440 L/d

2. Note the rectangular shape of the base, which is favoured for down slope sites

The parameters and features of the model sand mound for South East Queensland, with supporting comments, are in Table 7.9.

Table 7.9: Parameter and Features – Model Sand Mound for SE Queensland

Design Feature	Dimensions/Particulars	Comments
Surface: Topsoil depth Planting	 100 mm (minimum) Grass cover (1) or selected plants/sedges	
Gravel absorption bed: Depth	 225-300 mm	 Typically 250 mm depth (Bishop et al 2007)
Absorption bed media	20-40 mm clean gravel	Bishop et al (2007)
Mound media	2.5 mm	Medium sand (Bouma et al 1975)
Mound slopes	3 (Horizontal) : 1 (vertical)	Bishop et al (2007) and AS/NZS 1547:2000
Ground slopes	From 0 to 15%	Crites and Tchobanoglous (1998); Bishop et al (2007); Bouma et al (1975)
Allowance for surface gradients	Increase length, as per AS/NZS 1547:2000	Consider a rectangular configuration, long side downhill
Allowance for linear loading	Increase basal area by a factor of 1.2 (Simpson suggestion)	To avoid effluent breakout (Converse and Tyler , 2000; J Whitehead, pers. comm. September 2010)
Separation from ground surface to bottom of absorption bed	600 mm (min)	Converse and Tyler (2000); AS/NZS 1547:2000; Bishop et al (2007), and Bouma et al (1975)

Note:

1. Denotes that kikuyu grass preferred due to its potential for nutrient uptake

Using the information in Table 7.9 and working on a daily flow of 700 L/d, the sizing of a model sand mound for South East Queensland for a level site has been undertaken by applying AS/NZS 1547:2000 (Appendix 4.5B). Using Figure 4.5B1 in the standard:

A = 1,200 mm distribution trench width
B = 7,200 mm distribution trench length
E = 600 mm clearance from under side distribution trench to ground surface
F = 225 mm depth of distribution trench
G = 300 mm height above distribution trench to mound top
H = 450 mm height above distribution trench to topsoil surface
I = 3,900 mm distance from distribution trench edge to upper side of mound edge
K = 3,900 mm distance from end of distribution trench to side edge of mound
L = 17,000 mm basal length
W = 7,800 mm basal width
Batter slopes = 1 vertical: 3 horizontal
Basal area = 117 m^2
Adjusted basal area = 141 m^2 (increased to compensate for linear loading by a factor of 1.2)

Refer to Figure 7.4 that shows the above dimensions.

3 Literature Review on Sand Mounds

Some of the earlier research work on mounds is reported in Section 2.1. Originally, the Wisconsin mound was designed for specific soils and site limitations (Converse and Tyler, 2000).

3.1 Earlier Mound Designs in New Zealand

Gunn (1989) reports on the *NODAK* mound, the Wisconsin mound and the Ellesmere County fill system. Gunn (1994) reported that sand mound designs in New Zealand were based on the Wisconsin mound. The design sizings were based on the following:

1. 50 mm/d – distribution area
2. 35 mm/d – basal area over free draining soils
3. 12 mm/d over slowly draining soils
4. 75 L/m/d – toe loading, maximum value

Since late 2000 the sand mound design would have been based on AS/NZS 1547:2000.

3.2 USA Sand Mounds

Based on further research and evaluation, mound technology was expended to larger systems and more difficult soil types and site conditions. The new criterion was incorporated into a siting, design and construction manual (Converse and Tyler, 1990). Since the mound is essentially a combination of a single pass sand filter and a disposal unit, many of the sand filter research findings should be implemented into mound technology. Useful information and design criteria sourced from Converse and Tyler (2000) is as follows:

1. Minimum separation from the ground surface to bottom of the distribution bed is 600 mm
2. Minimum depth to bed rock is 600 mm
3. Sand fill composed of coarse sand with a minimum of 5% fines
4. Mound slope is not a factor of mound performance but rather a safe construction aspect and a slope that can be mown.

The Wisconsin mound has been used in Clermont County, Ohio since 1985 but it has not been a popular option due to the size of mound required. A study reported by Benson and Griffith (2001) used the secondary pre-treatment methods of aerobic treatment, intermittent sand filtration, peat filtration and suspended growth aerobic treatment. Preliminary sampling data indicated that the mounds dosed with secondary effluent would provide excellent effluent renovation, on the poorly drained Clermont silt loam.

3.3 University of Wisconsin Mound Study

Earlier work by the University of Wisconsin was undertaken and reported by Bouma et al (1975). Medium sand was used as fill material within the mound. Loamy sands or sandy loams have better filtration properties than sands but their potential for clogging is higher. Bouma et al (1975) reported on the performance results of three of the mounds. Their results are provided in Table 7.10. Toe seepage was not detected in these mounds.

Table 7.10: Experimental Wisconsin Mound Performance Results

Mound No.	**BOD_5 (mg/L)**	**COD (mg/L)**	**NH_4- N (mg/L)**	**NO_3-N (mg/L)**	**Total N (mg/L)**	**Faecal coliforms (CFU/100 mL)**
1	12	166	0.4	1.5	3.7	5
2	11	140	2.7	2.3	6.2	6
3	13	57	0	17	18	1
4[1]	-	-	-	-	-	-

Comments on Table 7.10 are as follows:

1. BOD_5 results are very low, well within the secondary treatment maximum of 20 mg/L
2. COD results are of less significance in domestic wastewater
3. There are strong reductions in faecal indicators
4. Complete or near complete nitrification being achieved, as indicated by the low Total N levels.

The study concluded that the mounds performed satisfactorily in soils that would not have been suitable for conventional sub-surface trenching.

The objective of a recent study by Blasing and Converse (2008) was to evaluate the quality of effluent that accumulates in a mound and modified mound toes under saturated conditions when receiving the following:

1. Septic tank effluent
2. Aerobically treated effluent.

Fifteen mound and modified mound systems with occurrences of saturation at the mound toe were sampled over 2001/2002. Blasing and Converse (2008) reported the following treatment responses:

1. All categories had a Total N reduction of at least 55%
2. When all systems were evaluated as a group the geometric mean value was 79 CFU/100mL
3. Also as a group the BOD_5 geometric mean values ranged from 2 to 5 mg/L.

It was concluded that as mound and modified mound systems are placed in more difficult sites which pushes the limits of the soil, the potential for mound toe leakage increases.

3.4 Experimental Sand Mounds - Salt Ash, NSW

Salt Ash is a small community of about 300 people, located near Port Stephens. The community has the water quality constraints of high groundwater, permeable sandy soils and abundant shell fishing resources.

Two sand mounds were installed at Salt Ash and monitored for six months to address this issue. The results of this monitoring have not been presented as the focus was on the contamination of groundwater rather than on sand mound performance.

An overall description of the sand mound features is presented by Whitehead and Geary (2009) in Table 7.11. Whitehead and Geary (2009) reiterate that little research has been conducted in Australia on the performance of sand mounds.

Table 7.11: Features of the Salt Ash Sand Mounds

Description	Pre-treated effluent is pressure dosed via a manifold in coarse aggregate near the top of a mound of sand through which it permeates.
Uses	Mounds are used where soil permeability is low, rock is close to the surface, or if water tables are high.
Performance	Depending on the design, mounds can significantly reduce BOD_5 and SS. Nitrification can be significant.
Space Requirements	Area is determined by analysis of soil tests and is quite variable but can require a large footprint.
Maintenance	The system requires reliable power and pump maintenance or replacement. Alternately on sloping sites siphons may be used to eliminate the need for power or maintenance. Mound vegetation requires maintenance.

Benefits of sand mounds include (Whitehead and Geary, 2009):

1. Increased separation distances to groundwater
2. BOD_5 and SS reduction
3. N reduction
4. Reduced risk of pathogenic bacteria contamination
5. Significantly reduced contaminates entering the shallow groundwater. Whitehead and Geary (2009) feel this is due to the unsaturated soil conditions, due to the periodic effluent dosing, by pumping.

A comparison of effluent quality from septic tanks, aerobic treatment plants and sand mounds is reported by Geary and Gardner (1996) in Table 7.12.

Table 7.12: Comparison of Effluent Monitoring Results - Septic Tank, Aerobic Plant and Sand Mound

Parameters (mg/L)	Septic Tank	Aerobic Plant	Sand Mound
BOD_5	120-150	5-80	1-10
Suspended Solids	40-190	5-100	5-20
TKN	50-60	25-50	30-50
TP	10-15	7-12	5-10

The following observations from Table 7.12 can be made:

1. The septic tank effluent results are typically of a primary standard
2. The aerobic plant results, particularly BOD_5 and suspended solids, are widely variable
3. The sand mound results are typically of a secondary standard
4. TKN results indicate that limited nitrification is taking place
5. Total P results indicate that limited reduction is taking place.

The sand mound has the following suitability qualities:

1. Soil permeability – suitable for slower and rapid K values
2. Depth to bedrock – suitable for shallow and deep
3. Depth to water table – suitable for shallow and deep
4. Slopes – suitable for slopes from 0 to 15%.

This is a useful suitability assessment but it tends to be generalised.

3.5 Port Stephens Council Mound Designs

The standard designs of four types of Wisconsin mounds and raised pressure dosed beds have been developed by Whitehead (2005) and they are as follows:

1. Wisconsin mound on estuarine clay
2. Wisconsin mound on sand
3. Raised pressure compensating sub-surface irrigation
4. Raised pressure dosed adsorption bed.

The designs have been centred on the work by Converse and Tyler, AS/NZS 1547:2000 and experience gained in the area. In today’s world, in South East Queensland, the most popular sized homes have four bedrooms. A selection of design criteria, for a four bedroom house on reticulated water supply, is covered in Tables 7.13 and 7.14.

Table 7.13: Wisconsin Mounds on Estuarine Clays

Design Flow (L/d)	Gravel Adsorption Bed dimensions	Overall Mound Dimensions (m)	Area (m^2)	Media	Surface Cover and Topsoil
860	21.5 m length 1.2 m width 300 mm thickness	28.5 m length 8.4 m width 1.2m height	277	**Sand Fill Media:** 0.25 - 0.75 mm, uniformity co-efficient <4, <5% fines **Distribution Bed Media:** 20-40 mm clean	Grass and 100 mm (min) loam/sandy loam

Table 7.14: Wisconsin Mounds on Sand

Design Flow (L/d)	Gravel Bed Adsorption Dimensions	Overall Mound Dimensions (m)	Area (m^2)	Media	Surface Cover and Topsoil
840	Length 17.5 m, width 1.8 m, thickness 300 mm	8.1 m width, downslope width 3.3m, height 1.2 m	194	**Sand Fill Media:** 0.25 - 0.75 mm uniformity coefficient <4, <5% fines **Distribution Bed Media:** 20-40 mm clean	Grass and 100 mm (min) loam/sandy loam

Notes:

1. Suitable for up to 5% slope
2. Scarify natural soil to a minimum depth of 200mm
3. Maximum batter slope 1 (vertical) : 3 (horizontal)

The writer's comments on Tables 7.13 and 7.14 are as follows:

1. The design flows, for four bedrooms and reticulated water supply of 840 and 860 L/d are very similar to the writer's standard approach. His own basis is 145 L (with standard water devices fitted), times 4 plus 1 persons, times a peak factor (dependant on soil category) of at least 1.15 = 834 L/d
2. In terms of basal area, the sand mound on estuarine clay is at least 70% larger than a mound on sand, due to the large different in soil K values
3. Media sizes, topsoil depth, slope suitability, maximum batter slopes and the scarification depth are the same for both standard designs.

The details of the standard design raised pressure compensating sub-surface irrigation bed and the raised pressure dosed adsorption bed have not been tabulated. However, they are both considered to be useful modifications or alternatives to the Wisconsin mound.

3.6 Whitehead and Associates - Comments on Sand Mound Design

Bishop and Whitehead (2007) examined the mound design work by Converse and Tyler (2000) and others to find that some factors were not well addressed by current Australian guidelines and standards. For example, in free draining soils liquid will move vertically down. In soils with flow restrictions and limiting layers, liquid movement may include a horizontal component which can lead to a saturated fringe and toe seepage failure. This principle forms the basis for incorporating linear loading rates in mound design which is given insufficient coverage in AS/NZS 1547:2000 (Joe Whitehead, Newcastle, pers. comm. September 2010). This is the most critical factor to avoid breakout.

Joe Whitehead (pers. comm. September 2009), offers the following additional suggestions and comments for sand mound design:

1. If a sand mound is properly designed on level ground, toe seepage should not be a problem
2. Monitoring of seepage from toes has shown good results
3. If there is any concern at the design stage, consider a bund around the toe as an option
4. Consider a gravel filled trench behind the toe as an option
5. Consider trees or shrubs on the down slope.

It is understood that about 60 sand mounds have been constructed, based on the Port Stephens Council design by *Joe Whitehead and Associates*. They are working well according to Phillip Geary, University of Newcastle (pers. comm., March 2009).

4 Reflections on Sand Mounds

4.1 Applications for Sand Mounds

The sand mound was originally designed for the specific applications of higher bedrock and groundwater levels. On reflection, the writer feels that they are still an appropriate option for those applications but their use can be extended to areas with slower permeability soils.

The former Caboolture Shire Council approves mounds in slow permeability soils. The writer has undertaken soil assessments in all areas within South East Queensland and has only encountered the occasional location where soil K values are very low. As a factor of safety, the writer feels that some degree of soil permeability must be available, that is at least K varying from 0.08 and 0.13 m/d, to avoid overloading and toe seepage.

4.2 Sand Mound Design Aspects

Having had discussions with Joe Whitehead, on reflection the writer concurs with the following:

1. Converse and Tyler (2000) are considered to be the authorities on modern sand mound design.
2. The *Whitehead and Associates* design for Port Stephens Council generally has a larger basal area than AS/NZS 1547:2000 designs. In this case they would work better and be a more robust solution since the existing AS/NZS 1547:2000 does not capture all the facets of design as reported by Converse and Tyler (2000)
3. Linear loading is important to avoid effluent breakouts
4. In the case of sloping ground, longer and narrower configurations are considered to be better.

4.3 Ellesmere County Fill System in New Zealand

The development of this simple fill system described in Section 1, to cater for clay soils and high water tables, was a proactive move in the 1980s. Thinking back this was a good example of the development of an area-specific system.

4.4 De-nitrification

In the Morayfield sand mound case study, it is possible that de-nitrification occurs within the sandy loam, clayey sand loam and sandy clay soils beneath the mound, before effluent eventually reaches the ground water. The confirmation of whether de-nitrification has taken place is outside the scope of this case study.

4.5 Sand Mound Medium

Earlier work by Bouma et al (1975) found that the selection of medium (or media) type and size was important to avoid blockages. They reported that medium sand had less blocking potential. Converse and Tyler (2000) have found that coarse sand, with a minimum of 5% fines is suitable. Since the writer has been concerned about potential blocking of sand filters for many years, on reflection, it is felt that the following aspects should be weighed:

1. Avoid potential blocking, by noting instances where this problem occurred
2. Selecting a sand medium size and type that produces the treatment objectives, based on the experiences of other reported sand mounds
3. Ensuring effective septic tank capacity and effluent filter pre-treatment.

5 Conclusions on Sand Mounds

The aims and objectives of Part (B) of Portfolio 7 have been addressed. Earlier development work on the design of *NODAK* and Wisconsin sand mounds has been reported. The writer's earlier work in Northland and more recent work in Caboolture have been reported. The work of other researchers have been reviewed and summarised. A model sand mound design for South East Queensland has been developed.

Suitability of Sand Mounds

The Wisconsin-type sand mounds were developed in the 1970s for use in situations where groundwater levels were shallow and where bedrock was shallow. It can be concluded that they also have an application where slow permeability upper soils exist.

Sand Mound Concentration

Possibly the greatest concentration of domestic scale sand mounds in Australia are in the Port Stephens area, NSW, which are based on the *Whitehead and Associates* design.

Case Studies

A case study in Woodford showed how an existing sand mound could be extended to cope with three bedrooms, using an overflow system.

The monitoring of a sand mound in Morayfield demonstrated that the technology was capable of achieving a secondary standard of effluent (BOD_5 and suspended solids), but the results showed some inconsistency and no particular correlations. It is to be noted however, that the monitoring period was limited and perhaps a clearer picture could have been obtained with more monitoring of effluent as well as monitoring of the influent.

Sand Mound Treatment

Sand mounds are capable of achieving a secondary standard of treatment, in terms of BOD_5 and suspended solids removal. Some sand mounds are achieving substantial Total N removal since aerobic conditions have been available, carbon has existed, and then anoxic conditions have also been available.

In contrast, others sand mounds are achieving limited nitrification and no de-nitrification, probably since suitable soil environments have not been available.

The degree of effluent treatment in sand mounds is aided by unsaturated flow conditions and the action of periodic dosing and resting which encourages aerobic activity (Geary and Gardner, 1996). The writer concurs with the above mechanisms that enhance effluent treatment in sand mounds.

It is apparent that when it comes to Total N reduction in sand mounds there are different responses. During the limited monitoring of the Morayfield sand mound limited Total N reduction was achieved, during unsaturated conditions.

Strong reductions in Faecal Coliforms have been reported (Bouma et al, 1975). This would have been due to the action of sand or fine gravel filtration and retention time. This capability is a definite advantage of sand mound since it ensures that a refined effluent enters the ground water resource.

Seepage from Mound Toes and Toe Planting

Seepage around the toes of the mound can be a problem on slopes and if the mounds are overloaded. Corrective measures include overdesigning the basal area, incorporating a clay toe and planting vegetation around the fringes, in the case of downslope sides.

The writer concurs with Converse et al (1976) that planting should be undertaken along the toes of mounds, only along the downslope side. It would not be a sound practice to plant the entire fringe, particularly with taller shrubs, since this would minimise the exposure of the sand mound to sun and wind.

6 Portfolio References

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Portfolio 8 – Lime Seawater Treatment

1 Introduction and Historical Development

1.1 Introduction

This Portfolio examines the chemical treatment of wastewater with lime and seawater and alternative lime and magnesium products.

The aim, in the form of an hypothesis, addressed in this portfolio is that "after accounting for the writers research work in the treatment of wastewater by lime/seawater/magnesium salts, reviewing the more recent literature and reflecting back, that this method of treatment is viable and worthy of further research".

Consequently, the objectives of this portfolio are as follows:

1. To summarise the writer's experimental work based on lime, seawater, and magnesium salts (Simpson 1986) and the joint QUT/Simpson study (Shanableh et al, 1995).
2. Undertake a further literature review to identify more recent developments
3. Confirm that the process of using magnesium salts is viable
4. Identify future developments and recommendations for further research and design
5. Reflect on past and more recent findings
6. Draw conclusions.

Publications by Simpson Appropriate to Portfolio 8

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2. Simpson, J S 1977, Feasibility Report on Environmental Aspects of Mixing Sewage Sludge with Kiln Lime, Natumix Fertilizers Ltd, Hamilton, NZ

3. Simpson, J S 1986, Chemical Treatment of Wastewater, Research Report for Diploma in Science, Auckland Technical Institute

4. Simpson, J S 1993, Innovative and Appropriate Engineering for Wastewater Treatment – Case Studies, Water Industry Workshop, Charters Towers

5. Burgess, M and Simpson, J S 1993, The Potential for a Low Technology, High Performance Wastewater Treatment Plant for Smaller Coastal Communities, *AWWA 15th Federal Conference*,

Gold Coast, April

6. Shanableh, A, Simpson, J S and Jomaa, S 1995, The Lime/Solar Salt Brine Process Performance and Optimisation, Queensland University of Technology, Department of Civil Engineering

The term chemical clarification, as used in this thesis, is a treatment process made up of the three distinct mechanisms of coagulation, flocculation and clarification, or otherwise known as sedimentation. The meaning of these mechanisms has been provided in the Glossary of Terms and Abbreviations.

1.2 Historical Development of Lime Treatment

The treatment of wastewater and the disposal of effluent from individual houses and communities on the coastlines and lake sides of New Zealand and Australia has been a challenge for many years. The writer recognised these challenges when he was undertaking post-graduate studies (Simpson 1974). Typically, population patterns can vary considerably, the protection of surface water and groundwater is a concern, plant management and economics can be constraints. The treatment of wastewater by lime/seawater and the management of fluctuating wastewater loads are linked, as shown in Portfolio 1 in that the potential for this method of treatment is greatest in lake side, estuarine and coastal areas.

Using chemicals for the treatment of wastewater is not new. An early reference to the use of lime for wastewater treatment is Barwise (1904). Vrale (1978) reported that chemical precipitation with lime and seawater was tried in 1972 and later carried out in a full scale operation at Sandvika in Norway, in 1974. The precipitation of magnesium in wastewater with lime is well reported by Mawson (1970), Cooper (1975), Walton (1976), Jenkins (1977), Culp (1978) Hruschka (1980) and Ayoub (1994).

Mawson (1970) reported in New Zealand that complete clarification of wastewater was achieved by using the chemical constituents of seawater and lime**.** A series of jar tests were carried out on laundry water, domestic wastewater and an abattoir wastewater. Burnt lime was mixed in with proportions of seawater. The clarification process was rapid and positive; there was also significant colour reduction in the abattoir wastewater. Following the jar testing program, Mawson undertook the treatment of macerated domestic wastewater with burnt lime or calcium oxide and 30% seawater.

2 Lime/Seawater Treatment Research by Simpson

The writer undertook a literature review of chemical clarification of municipal wastewater Simpson (1986) which gave the treatment efficiencies presented in Table 8.1. This provided background information, treatment performances and a basis for comparison with the lime/seawater and substitutes methods.

Table 8.1: Treatment Efficiencies of Treatment Options (as % reduction in concentrations)

Method	Suspended Solids	BOD_5	Bacterial	Reference
Wastewater chemical treatment	80-90	70-80	80-90	Metcalf and Eddy (1979)
Wastewater chemical treatment	70-90	50-85	40-80	Fair (1968)
Wastewater chemical treatment	84-91	61-92	n.d	Ockershausen (1980)
Wastewater chemical treatment	70-90	50-85	40-80	Seelye (1964)
Lime+ Ferric chloride	88-92	61-76	n.d	Seelye (1964)
Lime + Ferric sulphate	95	67	n.d	Dept of Env (1979)
Lime + aluminium sulphate	84	40	n.d	Dept of Env (1979)
Lime	84-90	66-82	n.d	Jenkins (1977)
Lime	77-83	36-82	n.d	Hruschka (1980)
Lime /seawater	92-96	79	n.d	Ferguson (1984)

Note: n.d denotes not determined

The following observations can be made from Table 8.1:

1. Generally high suspended solids removal is achieved, when 80% reduction could be considered to be reasonable and over 90% very acceptable

2. BOD removal is generally high with some exceptions. Over 75% reduction could be considered reasonable
3. Limited bacterial removal data shows reasonable reductions
4. The lime/seawater method (Ferguson, 1984) compared favourably with more conventional methods, in terms of BOD_5 and suspended solids reductions.

A secondary standard of treatment is achieved in terms of BOD_5 and suspended solids, which is 20 mg/L BOD_5 and 30 mg/L suspended solids. The lime/seawater and magnesium salts methods have the added advantage of being able to achieve a tertiary or advanced secondary treatment standard for phosphate and E coli reductions.

By applying lime clarification the following substances can be substantially reduced from wastewater:

1. Suspended organic and inorganic matter
2. Particulate phosphorus and some dissolved phosphorus due to pH increase
3. Some heavy metals due to pH increase
4. Reduction in harmful bacteria and viruses by enmeshing in settleable material or die-off, when lime is used to increase the pH.

This Portfolio includes seawater substitutes, which includes magnesium sulphate and solar salt brine or bitterns. It is to be noted that Section 2 focuses on the writer's experimental work but it has been found to be appropriate to include some work by other researchers, for a more cohesive understanding.

2.1 Background and Objectives of Jar and Pilot Plant Tests

The writer made initial contact with Keith Mawson in 1974 whilst undertaking a university dissertation (Simpson, 1974).

A Norwegian researcher Vrale (1978) claimed that jar tests on the chemical precipitation of wastewater with lime and seawater was undertaken for the first time in 1972. Keith Mawson however, undertook jar tests in New Zealand prior to 1970, which was reported by Mawson (1970).

During the 1980s the chemical clarification of wastewater definitely emerged as an attractive option, some reasons for this including the following (Simpson, 1986):

1. It had become a requirement in many countries including New Zealand, Australia, and Norway to reduce phosphorus levels in effluent prior to surface and groundwater discharge. Chemical treatment has been used for this purpose

2. Chemically assisted sedimentation requires smaller settling tanks and more efficient solids removal can be achieved
3. Toxic materials can often be chemically precipitated allowing further treatment
4. High fluctuations in wastewater strength and flow can impact on physical and biological treatment. Chemical pre-treatment can have the effect of smoothing out these fluctuations. (Refer to Portfolio 1, which deals with handing fluctuating flows)
5. In some instances the effluent quality can be acceptable enough to be discharged without additional treatment.

Early researchers agreed that that the lime/seawater clarification process had considerable potential in seaside or coastal locations where phosphorus reduction would minimise weed and algal growth. A further consideration was that bathing and shell fish areas could be protected by substantial reductions in harmful bacteria and organic pollution in effluent discharges.

More detailed contacts were made with Keith Mawson (pers. comm. August 1983 and August, 1984). The writer, based on these discussions and correspondence, undertook a lime/seawater research project, using domestic wastewater, laundry water and a range of industrial process waters, starting in 1985. This work was part of a Diploma in Science research project at the Auckland Technical Institute (Simpson 1986).

The objectives of the writer's research in 1985-1986 were as follows:

1. Conduct jar tests using lime and seawater and some lime and magnesium substitutes and focus the study on the lime/seawater treatment of macerated municipal wastewater. Develop mixing and dosing techniques
2. Select optimum dose rates and pH ranges and conduct pilot plant trials, as a batch process
3. Conduct at least one pilot plant trial using a continuous process, using the upflow sludge blanket technique. Determine treatment efficiencies of batch and continuous pilot plant trials. Investigate some chemical substitutes to seawater, for application in non-saline environments
4. Determine sludge properties and develop a viable and economic method of sludge stabilization, based on his previous sludge management experience
5. Derive parameters for scale up to full scale plant design. Investigate briefly the impact of discharging the treated effluent into fresh and saline receiving waters
6. Recommend future developments for the lime/seawater process and the stabilised sludge. Also refer to Section 3.

The project objectives were to achieve the following standards of effluent treatment:

1. Up to 90% BOD_5 removal from domestic wastewater
2. Up to 90 % suspended solids reduction
3. High bacterial reductions, greater than 99%
4. Significant phosphorus reduction, greater than 65-70%
5. Reducing the final sludge to at least 70-80% moisture content, to a level suitable for landfill co-disposal

The following sub-sections, based on the list of objectives, give an account of the writer's experimental program components. A list of the experimental jar test and pilot plant tests is in Table 8.2. The experimental program consisted of the following activities:

1. Jar testing, which encompassed variable dosage rates, a pH range, mixing times and settling rates
2. Pilot plant batch trials, involving greywater and municipal wastewater
3. Pilot plant continuous trials, using municipal wastewater
4. Sludge management
5. Effluent dilution in fresh and saline waters.

Table 8.2: Summary Description of Jar and Pilot Plant Tests

Experimental Tests (Simpson, 1986)	Test Description
JT 1 to 12	Seawater dose rates and mixing
JT 14	Seawater mixing
BT 15 and 17	Optimum seawater dose and pH
BT 18, 26, 31, 32, 33, 40 and 47	Seawater substitutes
BT 26	Optimum pH, mixing and settling
BT 28	Changes in effluent concentrations
BT 22, 24, 30, 37, 38, 42, 44, 45 and CPPT 48	Sludge tests
BT 40	E coli
BT 41	Sludge recycling to improve treatment efficiency
BT 43	Effluent dilution in fresh and saline waters

Note:

1. JT denotes jar tests, BT denotes batch pilot plant tests, CPPT denotes continuous pilot plant tests

2.2 Jar Testing and Associated Aspects

The first phase of the writer's experimental program was an exploratory procedure. This was followed by a more controlled procedure and the following functions of jar testing were confirmed during the research project:

1. Determination of coagulant doses, lime doses and seawater content by volume
2. Determination of optimum pH
3. Determination of floc sizes and strengths
4. Identifying rapid and slow mixing times
5. Measuring settling rates.

2.2.1 Seawater Content

Ferguson (1984) and Odegaard (1989) reported seawater content for their work varied from 1 to 15%. As little as 1 to 2% seawater was needed if pH values exceeded 11. If pH values were around 10.5 then 10% seawater was required. Mawson (1970) experienced success with early jar testing in New Zealand using 10 to 30% seawater with lime. Vrale (1978) experimented with seawater contents varying from 1 to 15% in Norway. The writer had similar success in the jar testing and pilot plant trials (Simpson, 1986) by confirming 10% seawater efficiently treated municipal wastewater, as reported later. The key factor was that the magnesium in the seawater was sufficient to supplement the lower magnesium levels in the wastewater to promote $Mg(OH)_2$ precipitation.

2.2.2 Hydrogen Ion (pH)

It is recognised that pH is important in most unit processes in water and wastewater treatment and it plays a major role in the destabilisation of suspensions (Brady, 1980). The jar testing program showed that various process waters and wastewaters responded to coagulation and flocculation at varying pH levels. It was evident during the writer's jar testing with municipal wastewater that higher pH levels gave better responses to mixing, settling velocities and clarity. When working with fresh municipal wastewater better performances were achieved within the higher pH range of 10.16 to 11.18. The near optimum pH was 10.6 to 10.7.

2.2.3 Mixing Times

The function of flash mixing ensures that coagulation is homogeneous within a relatively short time. Earlier work with jar testing (JT 1 to 12) was undertaken under a rapid mix time of 1 minute, with no slow mixing. These jar tests involved municipal wastewater and a range of process waters with varying amounts of seawater. Following some detailed mixing trials the remainder of the jar testing was standardised on 2 minute rapid mixing followed by 2 minutes of slow mixing. This showed the following improvements:

1. Considering the slow solubility of lime in water, this gave more time to attain the higher pH needed for better performance
2. This gave more time for larger and stronger flocs to form
3. Definite clear zones were soon developed near the surface and acceptable settling velocities were obtained.

2.3 Staging of Pilot Plant Trials

2.3.1 Stages 1 and 2

The jar testing program revealed good performance results in that it demonstrated the broad scope or capability of the treatment technique. This was also the case with the jar experimental program reported by Shanableh, et al (1997)**,** as discussed later. The next progression was to undertake larger scale pilot plant testing.

The pilot plant trials were undertaken in two stages. Stage 1 consisted of batch tests, based on the experience gained from the jar testing program.

Stage 2 was based on the experience of Stage 1 batch testing by conducting a continuous trial to determine design dosed rates, mixing times, and detention times. The continuous trial data was considered to more relevant to scale up design. Refer also to Section 2.3.

A photograph of the pilot plant is in Plate 8.1 and pilot plant slow mixing during a batch test is in Plate 8.2.

Plate 8.1: Lime/ Seawater Pilot Plant

Plate 8.2: Pilot Plant Slow Mixing during a Batch Test

2.3.2 Batch Testing Results

A relationship between % seawater/substitutes, flash mixing, slow mixing, and settling velocities are shown in Table 8.3.

Table 8.3: Seawater and Substitutes, Mixing and Settling Velocities

Test/trial No	% Seawater substitute	Rapid mix time (mins)	Slow mix times (mins)	Settling velocity (mm/s)	Clarity
BT 18	10 seawater	1	4	0.20	
BT 26	10 seawater	2	2	0.20	
BT 31	Dominion salt	2	2	0.25	3+
BT 32 (1)	10 seawater	2		0.20	3+
BT 32 (2)	Dominion Salt	2	2	0.30	3
BT 33 (1)	5 seawater	2	3	0.24	
BT 33 (2)	Dominion Salt	2	3	0.18	
BT 33 (3)	Calcined Magnacite	3	3	0.24	

Notes:

1. BT – denotes batch test in pilot plant
2. The data gaps in the results table denote that testing was not undertaken.
3. The salt was sourced from a company called Dominion Salt (NI)

Table 8.3 highlights the following experimental findings:

1. Dominion Salt gives the quickest settling rates of 0.25 and 0.30 mm/s with clarities of 3+ and 3
2. 10% seawater gives an acceptable settling rate of 0.2 mm/s and a clarity of 3+
3. A rapid mix time of 2 minutes followed by a slow mix time of 2 minutes would appear to be the optimum combination and also give 3 to 3+ clarities.

For the specific purpose of grading the clarity of various supernatants, after jar testing, the writer devised the classifications given in Table 8.4.

Table 8.4: Clarity Classifications

Class 1, 1+	Class 2-, 2, 2+	Class 3-, 3, 3+	Class 4-, 4, 4+	Class 5-, 5, 5+
Lower order clarity, not able to be seen through, e.g. milky	Less cloudy, some solids in suspension	Moderately clear, some colloidal material may be evident	Relatively clear, some colour may be evident	Highest order clarity, e g 5+ is approaching drinking water standard

This system was intended to be a simple and rapid method of determining supernatant quality. It was a visual grading of the combined effects of suspended material, colour and turbidity, but it did not attempt to correlate or separate them. It was a project specific classification which also eliminated the need and costs of determining turbidity and colour. Seawater settling velocities and mixing times are presented in Table 8.5.

Table 8.5: Seawater Settling Velocities and Mixing Times

Test No	% Seawater (by volume)	Settling Velocity (mm/s)	pH after Mixing	Rapid Mix (mins)	Slow Mix (mins)
1	10	0.13	10.61	1	n.d
2	10	0.16	10.30	3	n.d
3	10	0.17	10.20	1	2
4	10	0.20	11.18	2	2
5	10	0.09	10.44	2	3
6	10	0.10	10.20	2	6

Note:

1. Extracted from batch test (BT) 26 results
2. n.d denotes not determined

From Table 8.5 the following relationships and conclusions can be made:

1. An efficient standard of treatment can be achieved, using 10% seawater, with 2 minute rapid and 2 minute slow mixing, if the pH is taken to 11.18. The settling velocity was an acceptable 0.20 mm/s
2. There is no clear advantage of increasing slow the mixing time greater than 2 minutes

3. Using 10% seawater, over a pH range of 10.3 to 10.6, there is no advantage in increasing rapid mixing times greater than 2 minutes
4. The 2 minute rapid and 2 minute slow mixing combination is more beneficial
5. Slow mixing, following rapid mixing, based on observations, is definitely advisable.

2.3.3 Hydrogen Ion (pH) Results

The results from batch test 26 (BT 26) are plotted in Figure 8.1. It can be seen that 10% seawater gives a more favourable performance in terms of settling times, due to the shape of the curve. The optimum settling time was about 660 seconds which could be applied to design. It is also to be noted that the optimum pH was 10.6 to 10.7, that is, the pH at which the best performances took place.

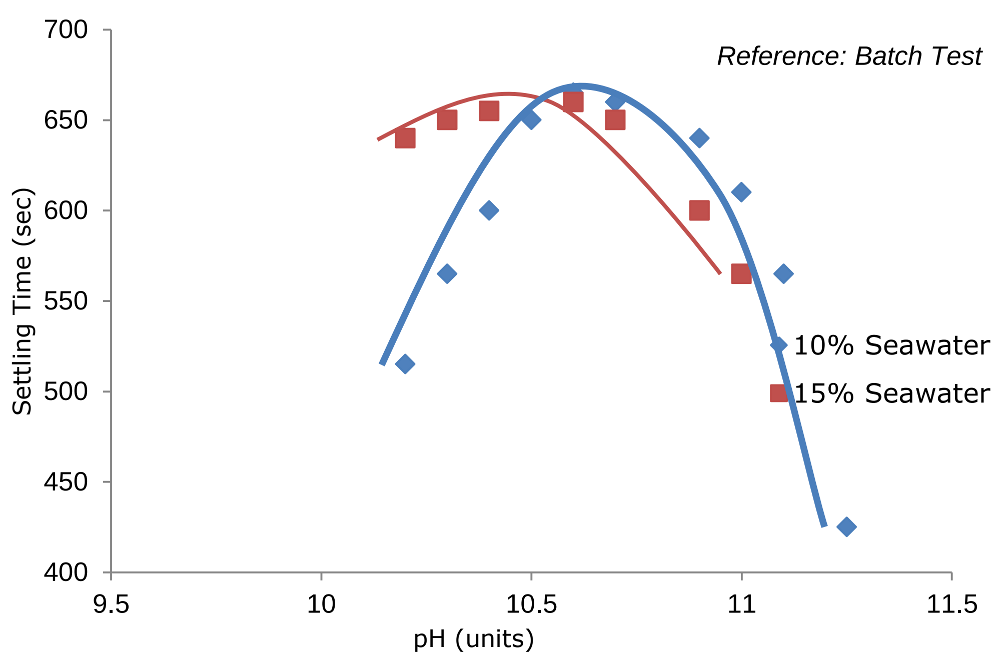

Figure 8.1: Batch Test 26 – Plots of Settling Times and pH

2.3.4 Seawater Substitutes

Mawson (1970) suggested chemicals such as dolomite lime, bitterns and the concentrated liquor remaining when common salt was crystallized, as being possibilities for inland or fresh water applications. This would reduce the sodium and chloride concentrations considerably. Mawson (1970) had concluded that the main flocculating precipitate was magnesium hydroxide.

The writer experimented with the following magnesium substitutes:

1. Dolomite, containing calcium and magnesium, was used in the batch tests, Simpson (1986). When used on municipal wastewater at a pH of 10.6 this promoted only an acceptable degree of flocculation, when compared with other magnesium products, but it was followed by rapid initial settling
2. A sludge from Dominion Salt (NI) Ltd, New Zealand contained calcium, magnesium and sodium chloride showed considerable more potential if worked to a pH of 11.0
3. Calcined Magnacite was dosed as slurry and it showed much potential if worked to a pH of 10.8
4. Magnesium sulphate as a substitute demonstrated the greatest potential.

Table 8.6 shows the comparative performances of seawater substitutes.

Table 8.6: Comparative Performances of Seawater Substitutes

Test/Trial No.	Process/ wastewater	Seawater substitute	Settling Velocity (mm/s)	% SS Removal	Optimum pH	Clarity
BT 15	Tannery wastewater	Dolomite	n.d	n.d	12.5	2+
BT 17	Municipal wastewater	Dolomite	0.6	n.d	10.6	3+
BT 31	Municipal wastewater	Dominion Salt waste	0.25	n.d	11.0	3+
BT 32	Municipal wastewater	Dominion Salt waste	0.3	n.d	10.85	3
BT 33	Municipal wastewater	Dominion Salt waste	0.18	n.d	11.80	n.d
BT 33	Municipal wastewater	Calcined Magnacite	0.24	n.d	12.24	n.d
BT 40	Municipal wastewater	Dominion Salt waste	n.d	85	10.70	n.d
BT 47 (1)	Brewery	Magnesium	0.2	n.d	11.12	4+

	wastewater	sulphate				
BT 47 (2)	Municipal wastewater	Magnesium sulphate	0.19	n.d	10.6	4

Note:

1. BT denotes batch trial, n.d denotes not determined

The clarity classification system was devised by the writer for this specific project, based on observed clarity of tested effluents, 1 being the lowest and 5 the highest clarity – see clarity classification in Table 8.2. Some parameters were not determined hence the missing data in places within Table 8.6.

The settling velocity was based on the time it took for substantial settling (visually) to take place. The following aspects can be concluded from Table 8.6:

1. The quickest settling rate is 0.6 mm/s, using Dolomite
2. Using magnesium sulphate gives the slowest settling rate of 0.18 mm/s, using municipal wastewater with Dominion Salt waste.

2.4 Treatment Efficiencies and Continuous Pilot Trial

When comparing treatment efficiencies in Table 8.1 it can be seen that the lime/seawater/magnesium salts methods compare favourably with more conventional methods, in terms of suspended solids and BOD_5 reductions.

A secondary standard of treatment is achieved in terms of suspended solids and BOD_5. The lime/seawater/magnesium salts methods have the added advantage of being able to achieve a tertiary or advanced secondary standard for phosphate and E coli reductions.

Since the pilot plant component of the experimental program focused on municipal wastewater, Table 8.7 compares the various treatment efficiencies.

Table 8.7: Comparative Treatment Efficiencies

Chemicals used	**Test type**	**Detention time (mins)**	**Clarity**	**% Removal BOD_5 (mg/L)**	**% Removal SS (mg/L)**
Lime + 10% seawater	BT 27	30-60	n.d	67-88	83-93[1]
Lime + 7.5% seawater	BT 36 [2]	50-60	n.d	49	81-82

Lime + 15% seawater	BT 21	50	n.d	n.d	83
Lime + 10% seawater	CPPT 46	n.d	n.d	78-81	78-92
Lime + Dominion Salt waste	BT 40 [2]	50-60	3+	63-68	77-85
Lime + magnesium sulphate	BT 47 [2]	n.d	4	n.d	n.d
Lime + Dolomite	BT 17	n.d	3+	n.d	n.d

Notes:

1. Denotes reactive phosphate reduction of 90 %
2. Denotes E Coli removal of >99.9%
3. BT denotes batch trial, CPPT denotes continuous trial' n.d denotes not determined

Observations made from Table 8.7 include the following:

1. The batch trial with lime and 10% seawater gave higher BOD and SS removals
2. The lime/Dominion Salt waste continuous trial gave a good clarity and higher BOD and SS removals
3. A very high clarity was achieved in the jar test with lime/magnesium sulphate.

For the purpose of comparing suspended solids and BOD_5 results of the batch and continuous trials on municipal wastewater, Table 8.8 has been compiled.

Table 8.8: Comparative BOD and Suspended Solids Results

Test/Trial No	**% Seawater (by Volume)**	**Effluent SS (mg/L)**	**SS % Reduction**	**Effluent BOD_5 (mg/L)**	**BOD_5 % Reduction**
BT 27	10	25	88	24	88.5
BT 36 (1)	10	57	88	44	67
BT 36 (2)	7.5	82	82	67	49
CPPT 46	10	38	78	23	78
CPPT 46	10	14	92	20	81

By graphing the suspended solids and BOD_5 results in the above Table 8.8 it can be seen that the points does not lie precisely on a line or a curve. Refer to Figure 8.2.

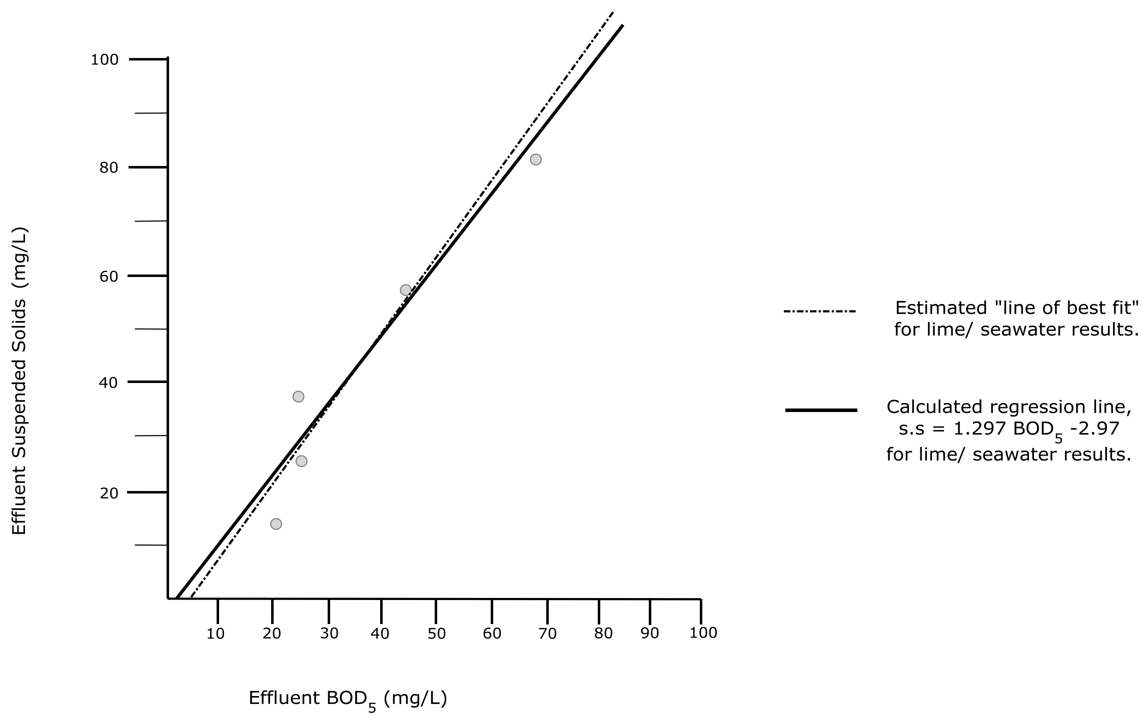

Figure 8.2: Plots of Suspended Solids and BOD_5

There is however, some connection between both the effluent parameters. The relationship between them depends on the ratio of insoluble to total organic matter present.

In municipal wastewater, the bulk of the oxygen demanding substances are probably insoluble in water. The suspended solids to BOD_5 correlation for effluent is worthy of more discussion. It may be feasible to monitor plant effluent largely on the basis of suspended solids results if a reasonably reliable correlation for the lime/seawater can be derived. Suspended solids results can be easily and quickly obtained. On the other hand BOD_5 results take 5 days to produce, during which time the results may be of limited use for plant operational purposes. Suspended solids results are also cheaper to obtain.

There is a near linear relationship of suspended solids to BOD_5 ratio of about 1.3, as shown in Figure 8.2. This has been examined in more detail by statistical analysis using the “method of least squares” by Fitzpatrick (1983). An estimated regression line was generated by the writer on Figure 8.2. This was in a similar position to the calculated and plotted regression line.

It would appear to be feasible to monitor a pilot plant using the lime/seawater treatment method on the basis of suspended solids results. Assuming that the feedstock had a reasonable constant composition, which is often the case with domestic wastewater; a satisfactory working ratio would be in the range of 1.10 to 1.35. It would then be advisable to periodically check that the effluent BOD_5 to confirm that the suspended solids to BOD_5 ratio was in the right order.

To examine the change in water quality, before and after lime/seawater treatment, results from BT 28 have been tabulated in Table 8.9. BT 28 was selected since it had 10% seawater content and the pH after mixing was 10.93. This wastewater contained an estimated 3% industrial content and 10 % seawater was used.

Table 8.9: Changes in Effluent Parameters

Parameter	Municipal wastewater	Treated effluent	Increase in Concentration	Decrease in Concentration
pH	7.17	10.89	n.d	n.d
TDS	326	2,721	830%	n.d
Calcium	104.5	13.7	n.d	87%
Magnesium	54.4	3.9	n.d	93%
Sodium	62.6	988.6	1,580%	n.d
Potassium	79.9	97.5	20 %	n.d
Iron	0.79	0.14	n.d	82%
Manganese	0.05	<0.01	n.d	> 80%
Zinc	0.3	0.01	n.d	97
Copper	0.03	<0.01	n.d	67%
Reactive phosphate	21.9	2.2	n.d	90%

Notes:

1. pH in units, all other parameter units in mg/L
2. n.d denotes not determined

The following conclusions can be drawn from the results in Table 8.9:

1. Due to the use of seawater, the increase in salinity, as shown by the TDS and sodium, increases

2. High decreases in calcium, magnesium, iron, manganese, zinc and copper
3. High reduction in reactive phosphate.

2.5 Scale Up and Effluent Discharge

2.5.1 Scale Up to Full Size Plant

A project objective was to scale up to a full sized plant that could be capable of treating wastewater from a small community. The detailed design was outside the scope of the experimental program and the report. At the time this project was undertaken, reported scale-up work for water and wastewater treatment, to use as a guide was very limited in New Zealand.

It was reported by Shaw (1981)**,** on the Mercer water treatment pilot plant (located south of Auckland), that the 10 m^2 settling tanks used could be representative for settling tanks up to 150 m^2 in area. This is a scale up factor of 15. According to Gallot (1986) this scale up factor, decided by *Binnie and Partners (London)* was felt to be conservative so a factor of 20 was considered more realistic.

Leentvaar et al (1979) reported on the clarification of beet sugar wastewater using a laboratory scale pilot plant having a flow rate of 60 L/hour. Since this flow rate approximated the flow of the writer's pilot plant the Leentvaar et al (1979) scale up factors were considered to be of direct interest, as in Table 8.10.

Table 8.10: Laboratory and Larger Scale Pilot Plants – Scale up Factors

Laboratory scale pilot plant (Leentvaar et al, 1979)	Larger scale pilot plant	Scale up Factor
Q = 60 L/hr	At Q = 4,000 L/hr (lower)	66.7
	At Q = 6,000 L/hr (average)	100
	At Q = 8,000 L/hr (upper)	133.3

Notes:

1. The laboratory scale pilot plant clarifier was a Dortmund Type settling tank
2. The larger scale pilot plant was based on a circular sludge blanket clarifier

Since a good correlation of batch, laboratory and larger scale pilot plant results were obtained by Leentvaar, based on COD reductions, the writer adopted a scale up factor of 100 for the design of the full scale plant. Assuming a per capita flow of 180 L/h/d and on a dry weather flow basis, this represents a treatment plant with an EP of about 800.

The methodology the writer used for the scale up from jar tests, to batch pilot plant tests, to continuous pilot plant and then to a full scale plant design was similar to that outlined by Bratby (1980), Purchas (1977) and Schmidtke (1983).

Scmidtke (1983) advocated that relationships between systems of differing scales can be established by applying the theory of similarity with regard to a chemical wastewater treatment manifested in the following ways:

1. Geometric similarity
2. Kinematic similarity
3. Dynamic similarity
4. Chemical similarity
5. Thermal similarity.

Bratby (1981) reported that following aspects:

1. There had been good correlation between laboratory scale continuous and full scale continuous plants, treating the same wastewater
2. There have been good correlations between laboratory scale and full scale batch plants
3. Batch testing results are not appropriate to solids contact or blanket type clarifiers designs

Bratby (1980) and Purchas (1977) do however, concede that although continuous pilot plant testing is the most reliable, reasonable correlation can be achieved between batch results and continuous results have been achieved provided that:

1. The geometry of both systems is similar
2. Stirring mechanisms of both are similar
3. Engineering judgment is exercised by applying efficiency or safety factors.

In an exterior situation, for the purpose of full scale design, the following physical factors, based on the writer's experience, must be considered:

1. Wall turbulence and non-uniform flow
2. Density currents induced by temperature and concentration differences within the vessel
3. Stagnant zones
4. The tendency for settled material to resuspend.

Jorgenson (1979) suggested a factor of safety of 1.50 for smaller clarifiers which were baffled and reasonably protected. Gunn (Dip PHE lecture notes, 1974) suggested applying a correction factor by reducing the settling rate by 25%.

2.5.2 Effluent Dilution in Fresh and Seawater

Based on the writer's experimental experience and the experience of Vrale (1978), in practice the overall salinity of the effluent can be reduced significantly during a continuous mode of treatment, whereby 3 to 4% seawater will give acceptable performance results.

This means that the TDS could be reduced to 1,000 mg/L and sodium to 400 mg/L. These levels are still higher than the recommended maximum (at the time of research) for irrigation waters as reported in Aikman (1983). Over more recent years acceptable levels for irrigation waters have been developed by the Federal Government and for most states of Australia.

Ferguson (1984) found that it was not cost effective to waste expensive chemicals for the purpose of reducing lime/seawater treated effluent pH before discharging into water environments. Due to the concern of discharging the high pH and high salinity effluent, the writer undertook experimental work on the effects of discharging lime/seawater to fresh water, estuarine water and seawater (Simpson, 1986).

The writer found that disposing effluent to fresh water required high dilution, to reduce the pH, TDS and sodium effluent. Effluent disposal to estuarine and marine waters required low dilution since the pH reduces rapidly and the background salinity is high. Water quality standards and regulations normally set an upper pH limit of 9 for effluent discharge to waters.

BT 43 examined the dilution of effluent in fresh and seawater. The dilution results are given in Table 8.11.

Table 8.11: Effluent Dilution in Fresh Water and Seawater

% Seawater (by volume)	Freshwater Dilution	Seawater Dilution
5%	67:1	3.8:1
10%	69.7	3.8:1

The following aspects can be noted from Table 8.11:

1. The increase in seawater content from 5 to 10% makes little difference to the dilution ratios

2. The fresh water dilution ratio is considerably higher than with saline water
3. The seawater dilution is very low, which is probably due to the buffering action of the saline water.

The difference in dilution of 10% seawater has been graphed in Figure 8.3 to demonstrate the difference in diluting in fresh water and saline waters.

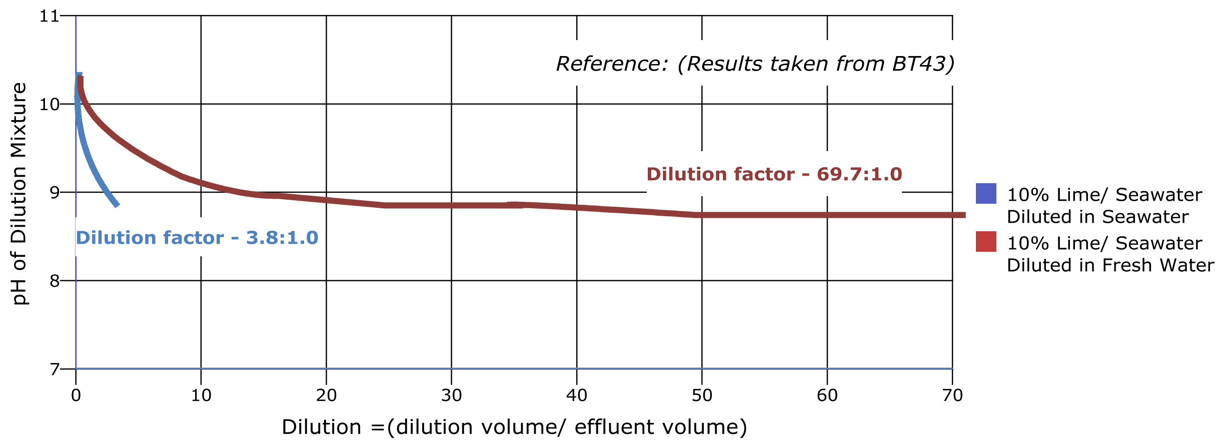

Figure 8.3: Curves of Effluent Dilution in Fresh Water and Saline Water

It can be concluded from Table 8.11 and Figure 8.3 that there is considerable merit in discharging lime/seawater effluent into estuarine and marine waters due to the considerably lower dilution rates.

The lime/seawater treatment method is responsible for the precipitation of metals. It is outside the scope of this thesis to discuss the various mechanisms involved other than it is by magnesium hydroxide precipitation.

Based on other research it has been found that various metals are precipitated with a slow increase in pH. Lanouette (1977) developed theoretical precipitation curves for various metal hydroxides. Of particular interest was the reduction of Zn, Ni, Cu, Cd, over a pH of 7.5. These metals are often present in domestic wastewater. Experimental work by Ockershausen (1980) showed that by raising the pH with lime to over 9.5 this precipitates Pb, Cu, Cd, As, Cr (111) and Zn from municipal wastewater.

The result of 90% reactive phosphate removal in BT 28 represents a significant orthophosphate reduction. Since orthophosphates are approximately 80% of the Total P in municipal wastewater, the overall phosphorus removal was very acceptable.

E coli organisms before and after lime/seawater treatment were examined by the writer in BT 40. Greater than 99.9% reductions were achieved. The clarification process would account for some E coli reduction. The high pH is considered to be responsible for considerable die-off**.** Vrale (1978) reported that high reductions of E coli were achieved in Norway, particularly when the pH was raised by lime to over 10.5. Vrale also verified that this was due to the combined effects of die-off by high pH exposure and the flocculation/sedimentation processes.

2.6 Sludge Properties and Management

2.6.1 Sludge Properties

The extent to which wastewater sludges will consolidate provides useful information on the volume of sludge that requires disposal. Earlier work by Mawson (1970) showed that during jar tests large over doses of lime resulted in increased Mg $(OH)_2$ sludge volumes but good settling and compaction properties were obtained.

Lime/seawater sludge settling and compaction trials were undertaken by Simpson (1986) and the results are in Table 8.12.

Table 8.12: Sludge Properties

Test/Trial No	**% seawater and Dominion Salt**	**pH**	**Imhoff sludge vol (% vol of original)**	**Lime dose (Lime to sludge or mg/L)**	**Sludge age (days)**
BT 22	10	10.89	36	n.d	4
BT 24	10	12.67	n.d	1:2	n.d
BT 30	10	n.d	65	n.d	1
BT 37	10	n.d	74	n.d	6
BT 38	10	n.d	n.d	n.d	n.d
BT 42	Dominion Salt waste (1)	n.d	44	n.d	n.d
BT 44	10	8.95	n.d	100 mg/L	n.d
	10	9.39	n.d	200 mg/L	n.d
	10	10.68	n.d	500 mg/L	n.d

	10	10.96	n.d	700 mg/L	n.d
BT 45	5	11.08	n.d	n.d	n.d
	10	10.89	n.d	n.d	n.d
	15	10.80	n.d	n.d	n.d
CPPT 48	10	>11.0	60	n.d	7

Notes:

1. Seawater substitute
2. Moisture content by volume for BT 24
3. Specific gravity for BT 38
4. BT denotes batch trial, CPPT denotes continuous pilot plant trial, n.d denotes not determined

Observations made from Table 8.12 included the following:

1. Lower lime doses do not attain the near optimum pH
2. BT 22 sludge, using 10% seawater and 4 days old, compacted the best
3. CPPT 48 sludge, using 10% seawater and 7 days old, compacted nearly the worst
4. Dominion Salt sludge compacted comparatively well
5. BT 30 confirmed, by the poor consolidation, that fresh municipal wastewater sludge requires aging prior to final storage, decanting or other treatment.

The following observations were made by the writer, 20 days after the sludge volume test, on the sludge from BT 22, using 10% seawater:

1. It was chemically stable, as indicated by no changes in colour or odour
2. It was biologically inert, since there were no signs of gas or biological action
3. It remained in a compact state and tended to shrink, indicating that greater consolidation would be obtained in practice than was shown in the 1.0 L Imhoff cone test.

The moisture contents of the lime/seawater sludges generated in the batch and continuous trials were not tested. Vrale (1978) suggested they could be in the range of 92.5 to 95%.

The writer undertook an experimental program using burnt lime on municipal wastewater sludge to develop a soil conditioner (Simpson, 1977) which in some respects was a forerunner to the lime/seawater project. The environmentally safe disposals of kiln dust from the cement industry and wastewater sludge were prevailing problems at the time.

The burnt lime was from the Wilson Portland Cement works in Whangarei, New Zealand and the digested sludge, which had a moisture content of 97.5%, came from the Whangarei Wastewater Treatment plant. The writer's experimental work involved bench scale testing which produced very encouraging results but larger scale testing was undertaken to confirm moisture contents and pathogenic micro-organism die-off. This work was undertaken to gain approval from the New Zealand Department of Agriculture in order that the final product could be marketed as a soil conditioner/natural fertilizer.

This earlier research of the writers (Simpson 1977), using burnt lime on municipal wastewater sludge, demonstrated the following:

1. Excess moisture was almost immediately bound up by hygroscopic action
2. Some heat was generated and Ammonia gas was liberated, over a period of 10-15 minutes
3. Tests undertaken by the Ruakura Agricultural Research Station, Hamilton, NZ proved that no pathogens survived
4. pH slowly reduced over a period of weeks by CO_2 absorption
5. Bulk mixes, ranging from 1 to 15 tonnes, showed that temperatures were generated to 85 degrees C and additional testing confirmed a completed die off of pathogenic micro organisms.

Sludge stabilisation scale up was not covered in the writer's lime/seawater research program. From the bench scale and bulk scale testing by Simpson (1977), the following data has been extracted and presented in Table 8.13.

Table 8.13: Kiln Lime to Sludge Ratios

Kiln Lime: Sludge	Test Mixing Scale	Approximate Moisture Content (%)
1:1	Bench scale	50
1:1	100 kg	45-50
3:1	Bench scale	17
2.7:1	1.0 tonne	20-25
3.3:1	7.5 tonne	20-25

It was apparent that scale up from bench scale to 100 kg field tests and to bulk field testing was probably not necessary when equal or higher proportions of burnt lime are mixed with wastewater sludge. The 1 tonne and 7.5 tonne field tests gave the same range of percentage moisture contents.

The bench scale test at a kiln lime to sludge ratio of 3:1 gave the lowest percentage moisture content. It was concluded that a kiln lime to sludge ratio of 3:1 was a suitable design mix.

It has been accepted for some years by process design engineers that the return of chemical sludges can benefit the performance for some water and wastewater treatment plants. Vrale (1978) reported that in a full scale plant operation the recycling of lime/seawater sludge improved the overall treatment efficiency.

To confirm the merit in recycling the lime/magnesium sludges the sludge from BT 41 was selected. This was a 10% seawater/municipal wastewater batch trial. Varying amounts of sludge were added to coagulated wastewater and the efficiencies determined by the settling times, the clarity of the supernatant and by comparison with control tests. The results showed that 1.5% (by volume) recycled sludge were close to the optimum for more rapid settling, and higher clarity.

2.6.2 Reuse of Lime Stabilised Sludges

The writers study on the mixing of kiln lime with municipal wastewater sludge (Simpson, 1977) concluded that lime stabilised wastewater sludges can be processed into a marketable fertilizer and soil conditioner. The finished product was economical to produce and environmentally acceptable. The study was undertaken to gain the approval of the New Zealand Ministry of Agriculture for use on dairy farms and orchards. The writer understands that *Natumix Fertilizers Ltd* obtained a US Patent.

2.7 QUT Lime and Solar Salt Brine/Seawater Joint Research

Based on the lime/seawater reporting by Burgess and Simpson (1993), Simpson (1986) and Ayoub (1994) the Queensland University of Technology and the writer jointly applied for research funding from the Queensland Foundation for Local Government Engineering. This research involved experimental treatment work with lime, lime/seawater and solar salt brines (bitterns).

The objectives were to investigate the effectiveness in achieving reductions of nutrients, BOD, suspended solids, turbidity, colour and enhance disinfection.

Published research on the lime/seawater process has not considered the enhancing effects of increased ionic strength on coagulation due to the increased concentrations of positively charged ions in solution (Shanableh et al, 1995). The various concentrations of seawater and solar salt brines are compared in Table 8.14.

Table 8.14: Mineral Composition of Seawater and Salt Brines

Parameter	Seawater (mg/L)	Solar Salt Brine (mg/L)[1]	Diluted Solar Salt Brine (mg/L)
Mg	1,350	91,800	1,350
Ca	400	30	1
Na	10,500	6,100	90
K	380	19,200	280
SO_4	2,760	76,000	1,120
Cl	19,000	239,000	3,520

Notes:

1. Sourced from Central Queensland Salt (Shanableh et al, 1995)
2. Concentrations are expressed as the averages

Table 8.14 is very revealing for the following reasons:

1. The very high concentrations of seawater in terms of Na and Cl, particularly when compared with diluted solar salt brine
2. The high Mg contents overall
3. The solar salt brine is very high in Mg, Na, K, SO_4 and Cl
4. The merit in diluting solar salt brine is very clear, as shown by the high Mg but lower salinity in the forms of Cl and Na. This very important attribute is covered in the reflections.

To compare the performances of lime/seawater and lime/solar salt brine processes, ten experiments were undertaken with raw wastewater and primary wastewater. The experiments were conducted using a combination of two limes doses (300 and 400 mg/L) and two seawater/solar brine doses (% by volume) in addition to a blank sample. The magnesium and lime doses and the resulting effluent parameters are in Table 8.15 (Shanableh et al, 1996).

Table 8.15: Performance Comparison of Lime /Seawater and Lime/Solar Salt Brine

Parameter	**RW**	**PW**	**SB**	**SW**	**SB**	**SW**	**SB**	**SW**	**SB**	**SW**
Mg Dose (mg/L)	0	0	24	24	24	24	48	48	48	48
Lime Dose (mg/L)	0	0	300	300	400	400	300	300	400	400
pH	7.5	7.5	10.9	10.7	11.3	11.2	10.7	10.8	11.1	11.0
Turbidity (NTU)	-	48	21	21	4.7	5	20	17	4.5	4.3
SS (mg/l)	288	84	44	28	8	8	48	20	8	8
Total P (mg/L)	9.78	7.87	1.82	1.55	0.39	0.37	1.53	1.30	0.38	0.33
BOD (mg/l)	260	155	93	78	66	53	88	102	75	63
COD Total (Mg/l)	580	247	230	183	212	185	195	190	180	165
TDS (mg/l)	1013	1070	1110	1990	1160	2020	1340	2910	1290	2960
Conductivity (mS/cm)	1.36	1.36	1.41	2.46	1.47	2.49	1.68	3.56	1.63	3.61
Sludge Volume (mL/L)	30	30	39	38	80	74	48	48	90	97
TKN (mg/L)	52	47	32.5	30	32.5	29.1	31.4	30.2	28	30.2

Abbreviations:

RW = Raw Wastewater
PW = Primary Wastewater
SB = Solar Salt Brine
SW = Seawater

These data indicate that both treatment processes gave comparable results. More specific observations from Table 8.15 include:

1. Lowest suspended solids and Total Phosphorus results for both processes are at a lime dose of 400 mg/L
2. Lowest BOD_5 results for both processes are at a lime dose of 400 mg/L

3. Lowest sludge volume for SB process with Mg dose of 48 mg/L and lime dose of 400mg/L
4. The salinity of the SW effluent was about double that of the SB effluent, which is very significant when the effluent is disposed of and the sludge is applied to land or reused.

The main outcomes of the QUT joint research work were as follows (Shanableh et al, 1995; Shanableh, 1997; Shanableh, 1998):

1. Lime/solar salt brine technology was capable of achieving reductions of 76% BOD_5, 71% COD, 90% turbidity, 96% Total Phosphorus, 98% Soluble Phosphorus, 96% Suspended Solids, >99.9% Faecal Coliforms, 85% Filtered Colour and 43 % TKN
2. Precipitation of a minimum of about 24 mg/L (100 mg/L as $CaCO_3$) was required to achieve optimum treatment results. Precipitation of additional Mg quantities did not significantly improve the treatment results and resulted in increased sludge production. The writer also noted the same behaviour in his experimental work (Simpson, 1986)
3. Both processes gave comparable treatment results. A major and very important difference was that the salinity of the final effluent from the seawater addition was nearly double the salinity resulting from using the solar salt brine
4. Disinfection was achieved, as shown by a 99.9% reduction of coliforms
5. The lime/solar salt brine process is useful for transient communities that experience extreme flow variations, such as tourist areas and military camps. (Refer to the section on fluctuating load treatment in Portfolio 1).

The treatment test results the writer reported in 1986 were never compared with the QUT joint study results. It is however, to be noted that the laboratory testing program for the QUT joint study was far more comprehensive, since it was financed by a study grant, than the Simpson (1986) project. The QUT joint study results were similar for lime/seawater and lime/solar salt brine, as noted in point 3 above. The 1986 results were for lime/10% seawater. On reflection, the percentage treatment reductions in point 1 above are compared with the findings of Simpson (1986) in Table 8.16.

Table 8.16: Performance Comparison of QUT Joint Study and Simpson 1986

Parameter	QUT Joint Study	Simpson 1986	Comments
BOD	76%	78-81%	Very similar for both studies
COD	90%	n.d	n.d
SS	96%	78-92%	Slightly higher, using lime/solar salt brine.
Total P	96%	90%	Lime/solar salt brine slightly higher.
Soluble P	98%	n.d	n.d
TKN	43%	n.d	n.d
Coliforms	>99.9%	>99.9%	The same result for both studies
Colour	43%	n.d	n.d
Turbidity	90%	n.d	n.d
TDS	(1)	(2)	n.d

Notes:

1. (1) - TDS increases substantially with increased lime doses, when using lime/seawater, TDS increases to a lesser extent, with increased lime doses, when using solar salt brine (Shanableh et al 1996).
2. (2) -A TDS increase of 8.3 fold (Simpson, 1986).
3. n.d denotes not determined

Conclusions from both studies in Table 8.16 are:

1. The use of lime/solar salt brine is generally more efficient
2. TDS results are lower when using solar salt brine, as mentioned in the notes above
3. Effluent disinfection, as indicated by the coliform reduction results for both treatment methods, is excellent.

Sea water brine or bitterns, is a rich source of magnesium. Compared with seawater, which contains about 1,350 mg/L Mg and 10,500 mg/L Na, the brine is relatively free of sodium and may contain more than 90,000 mg/L Mg. The main advantage of brine as a source of Mg includes reduced effluent volume and salinity and the need for a smaller chemical feed system (Shanableh, 1997).

The reduced sodium and chloride concentrations also provide a scope for inland situations and effluent reuse for irrigation and industrial purposes.

3 Literature Review on Lime Seawater Treatment

3.1 Research by Vrale, Norway

Vrale's (1978) early work was a very useful basis for the writer's experimental work (Simpson, 1986). Laboratory studies and full scale treatment plant work showed that seawater additions as low as 1 to 2% could greatly improve phosphorus removal efficiency when lime was used as a precipitant. The improved results obtained after seawater addition were believed to be due to the addition of magnesium, present in the seawater. The high lime, aided by the coagulation/flocculation process, was responsible for high E coli die off. The experimental work confirmed the above aspects (Simpson 1986).

3.2 Research by Odegaard, Norway

Odegaard (1989) advocated that the lime/seawater treatment was appropriate technology for coastal tourist areas and it had the following merits:

1. Low cost chemical treatment
2. Can meet most effluent quality requirements for SS, BOD, P and E coli.
3. Can deal with extremes in flow and wastewater composition. Refer to Portfolio 1
4. Simple to operate
5. Economically acceptable
6. Produces a well stabilised sludge.

Based on the writer's research and work experience he concurred with all the above merits. Shanableh et al (1995 and 1997) concur with merits 2 and 3 above.

The writer concurs with the article by Ferguson and Vrale (1984) that the lime/seawater process has considerable advantages for coastal communities and should be considered whenever biological treatment for BOD removal is not required and when discharge is to marine waters.

3.3 Research by Ayoub et al

Laboratory studies in the coagulation of alkalinised municipal wastewater using seawater bitterns were undertaken by Ayoub et al (1999). Seawater and liquid bittern was found to be an economic and effective source of magnesium. Turbidity and suspended solids removal exceeded 95% and COD removal was in excess of 75%.

Ayoub et al (1992) reported that the flocculation of wastewater at high pH achieves highly efficient removal of particulates, colloids and certain dissolved materials as humic acids and heavy metals. Alkaline flocculation has been found to be most effective in suspensions containing relatively high magnesium ions. The objectives of their research were to explore additional applications of seawater flocculation and to gain a better understanding of the conditions under which effective flocculation is achieved. The experimental results demonstrated the following:

1. That seawater flocculation is effective for demulsification of oily wastewater
2. That flocculation of alkaline wastewaters can be accomplished merely by adding seawater
3. The process is efficient for suspended solids and phosphorus removal
4. It removes forms of COD and nitrogen
5. It removes textile dyes.

The writer feels that the findings of Ayoub et al (1992) confirm the versatility of wastewater treatment by alkaline flocculation. Ayoub (1994) reported on the merits and disadvantages of the lime-seawater process compared with other chemically enhanced processes, which are summarised as follows:

1. Appreciative cost savings using lime and seawater
2. High pH induces disinfection
3. Lime enhances sludge de-watering
4. There are larger sludge volumes with lime-seawater, but it is well stabilized
5. High pH of lime/seawater effluent is not readily discharged into fresh waters which is a disadvantage. However, this is not a problem in estuarine and marine waters.

Other advantages of the lime/seawater/magnesium salts treatment methods are presented in Section 4.

Semerjian and Ayoub (2002) recognised that chemically enhanced wastewater treatment was attracting substantial interest, especially for wastewaters that are not amenable to treatment by conventional biological treatment methods. The paper presented the following:

1. A comprehensive review of high pH magnesium coagulation-flocculation processes in wastewater
2. Advantages (expanded in Section 4) and disadvantages
3. Process efficiency

4. Lime based sludge
5. Factors impacting on the process economics.

Ayoub and Merhedi (2002) studied, at laboratory scale, the characteristics and quantities of wastewater sludge produced for seawater, bittern and lime or caustic treatment. The parameters tested included sludge depths, settled sludge volumes, sludge volume indexes and water contents. The results indicated that a better settleable and more compact sludge is generated from the lime-alkalised than the caustic-alkalised process. The dewatering efficiency was however similar for both processes.

Ayoub and Semerjian (2006) presented a synopsis of low cost techniques for the removal of phosphates from wastewater. This included seawater and bittern for coagulation and flocculation. Using 10% seawater exhibited highly efficient removal (> 90%) of Total P. This compares closely with the writers work (Simpson, 1986) and the QUT/Simpson study (Shanableh et al 1995).

3.4 Research by Shin and Lee

The removal of ammonia N and P from wastewater, using magnesium salts was reported by Shin and Lee (1998). They studied factors such as pH, reaction times, N: P ratios, to find optimum conditions for magnesium ammonia phosphate formation. By increasing the pH to 10.5, up to 82.5% of ammonia-N and 97% of P were removed. Using industrial wastewater up to 72% of ammonia-N and 99% of P were removed.

4 Future Developments and Reflections

4.1 Chemical Treatment Recognition

In the writer's experience, since the 1960s, there has been a mindset that chemical treatment is restricted to water treatment processes only. When the writer upgraded the Queensland "Guidelines for Planning and Design of Sewerage Systems" (DPI 1991-1992) for local government this was the case. The writer incorporated options for chemical wastewater treatment in Volume 2, Section 11 of the guidelines. This was based on his review of the literature and the New Zealand experimental work with lime/seawater/magnesium substitutes. At the time P removal from effluent generated along the Queensland coast and the Great Barrier Reef Marine Park and within the Pumicestone Passage was a topical issue. As in Scandinavia, phosphorus removal from effluents discharged to water environments is a very important issue.

It is interesting that the public accepts the use of various chemicals for water treatment since this has been the traditional method of drinking water treatment for many decades. The writer found in New Zealand, up until he left in 1990, that chemical treatment for municipal wastewater was not recognised. During the early 1990s in Australia, chemical phosphorus removal in biological nutrient removal plants was being recognised. There were lime wastewater treatment plants in Darwin, Kambalda (WA), and the Lower Molonglo advanced treatment plant in Canberra (Burgess and Simpson, 1993).

On reflection, there is a need for public education on the use of chemicals for wastewater treatment because of the following merits with the use of lime/seawater/magnesium salts:

1. It is essentially a one off treatment process
2. Relatively high removal of BOD_5 and suspended solids.
3. High total P removal
4. A very well stabilised sludge is produced, when compared with the more difficult gelatinous sludge from water treatment
5. Disinfection is also undertaken when the pH is elevated. More expensive options include ozonation, UV radiation and membrane filtration
6. Colour removal is possible.

4.2 Significant Challenges in Wastewater Treatment Technology

By reflecting, in the opinion of the writer there are at least five significant challenges in wastewater treatment technology, which are presented in Table 8.17. These challenges are applied to the lime/seawater/magnesium salts processes.

Table 8.17: Significant Wastewater Treatment Challenges

Significant Treatment Challenges	**Lime/seawater or Magnesium Substitute Merits**	**Additional Comments and Reflections**
Treatment economy	Seawater has no cost, unless it is pumped. Lime is a cheap and it is natural chemical. Solar salt brine would have a minimal cost for procuring and transport.	In the writer's opinion probably one of the most economically competitive wastewater treatment methods.
Using a single unit process for solid/liquid separation	In the writer's opinion some screening may be desirable in larger plants but it essentially a single process.	Requires limited land area for the treatment tanks.
Achieving P reduction	The lime /seawater and substitutes methods achieve high soluble P and Total P. (Simpson, 1986; Shanableh et al 1995)	P reduction by biological treatment requires operator skill and it is a more sensitive process.

Sludge stabilisation and moisture reduction	A safer sludge is produced with no harmful bacteria surviving the high pH process. The lime enhances the dewatering and stabilisation mechanisms. (Simpson 1986; Shanableh et al 1995)	In the writer's experience this is one of the best conditioned wastewater sludges, with limited handling difficulties.
Reuse of sludge	The dewatered sludge is ready for agricultural/horticultural use as a soil conditioner. The high pH sludge has a definite application for acid clay soils. (Simpson, 1977) Lime is a natural product hence; it is compatible with soils and land use activities.	A successful soil conditioner in New Zealand, particularly in the dairy farming and horticultural industries. (Simpson, 1977)

4.3 Reflections on Simpson Lime Seawater Treatment 1986 Research Project

The writer has made some reflections on the recommendations he made in 1986 for future work, which are listed in Table 8.18.

Table 8.18: Reflections on Simpson Project, 1986

Simpson 1986 Recommendations	**Reflections**	**Comments**
Further continuous pilot plant trials be undertaken with municipal wastewater, using 5% seawater, to confirm sludge blanket characteristics, upflow rates and treatment efficiency parameters	A very useful exercise since the sodium and chloride concentrations would be lower than with 10% seawater. Dilution rates in both fresh and seawater would also be lower.	Earlier researchers have had good treatment results with seawater contents from 2 to 4% (Vrale, 1978).
Further continuous pilot plant trials are undertaken with seawater substitutes, in the form of magnesium salts, to confirm doses rates, pH levels, mixing requirements and sludge volumes.	This would be very useful since salinity levels would be suitable for irrigation and other forms of reuse. The sludge would be suitable for land disposal.	Lime and solar salt brine has produced very encouraging treatment results (Shanableh et al, 1995).
The viability of using magnesium salts to precipitate metals and to remove colour, to at least batch pilot	Some urban populations can generate wastewater with higher heavy metal contents, due to trade	A simple treatment process. Basically, the reduction in metal concentrations and

plant stage.	wastes from light industries. The removal of tannins is a problem for some Queensland coastal water.	colour removal is carried out when the wastewater pH is increased with lime, as found by Simpson (1986), Shanableh et al (1995) and others.
To investigate the viability of reusing lime/magnesium salts sludges.	This would have value in determining salinity levels and suitability for agriculture and horticulture, bacterial die-off, fertilizer value and the immobility of heavy metals.	Demonstrates the value of a single treatment process.

The continuous plant trial was undertaken at the Whenuapai Air Base near Auckland, over a 6 hour period. The writer appreciated that since the continuous pilot plant run was of this limited duration the design criteria would not be set from the experimental results alone. On reflection, the writer considered this to be sound thinking at the time. The experimental results are most useful for scale up and final plant design. This should be coupled with the design experience of similar flows and processes by others, which is considered to be an invaluable design contribution.

Thinking back, the lime/seawater process involves low costs for the chemicals or recycled substitutes; sludge compaction, dewatering and aging are not a problem, and the final product is environmental sound.

4.4 Reflections on Joint QUT/Simpson Research Project

It was certainly a move in the right direction during the QUT/Simpson study, when seawater used originally as a magnesium substitute, was replaced by a more concentrated magnesium source, which did not elevate salinity levels. This renders the final effluent suitable for land application and the resultant sludge for reuse as a soil conditioner or fertilizer, due to the lower TDS.

This research study confirmed the treatment versatility of the process by effective BOD_5, COD and Total N reductions; very high suspended solids and E coli reductions; and high turbidity and colour reduction. The study also demonstrated the value of using diluted solar salt brine, which had high magnesium content, and lower chloride and sulphate contents. It was also found that the magnesium hydroxide process could be undertaken with reduced lime dosages.

Chemical treatment is less sensitive to changes in ambient temperature and solar radiation than biological treatment. This is important in sub-tropical and tropical areas (Shanableh et al, 1995).

It could be concluded that the lime/ solar salt brine process was certainly viable and it could be applied to arrange of wastewaters and process waters. The resultant effluent and sludge could be readily reused.

4.5 Reflections on BOD Removal

The removal of BOD from effluents discharging into the Norwegian fiords was not a priority. The focus there was on the removal of phosphorus and harmful bacteria. Operational data from Norwegian treatment plants revealed a BOD reduction of 79% (Ferguson and Vrale, 1984).

The writer's experimental work gave BOD_5 reductions of 63-88% from batch pilot plant tests and 78-81% in the continuous pilot plant trial. These BOD_5 reductions are lower than the 90% reduction stated as one of his study aims. The joint QUT/Simpson study produced up to 76% BOD_5 removal (Shanableh et al, 1995). Wastewater treatment authorities in New Zealand and Australia would not accept these moderate levels of BOD reduction, without additional treatment.

The determination of soluble BOD_5 reduction levels, by lime/magnesium salts treatment, would be interesting. This would determine what insoluble and settleable amounts of BOD were required to be removed by physical/biological treatment. It is interesting to note that the lime/magnesium salts process reduces large concentrations of soluble phosphorus (Shanableh et al, 1995).

On reflection, based on the writer's experimental experience and his interest in the lime/magnesium salts process since 1974, there are two options that could be followed for initial BOD_5 reduction:

1. More focus on the selection of lime products and magnesium salts, to optimise treatment performance and minimise salinity levels
2. Some form of biological pre-treatment for initial BOD_5 (settleable, soluble and insoluble) reduction. A reed/gravel bed would suffice for this purpose and also be capable of reducing some Total Nitrogen.

A conceptual two stage reed/gravel bed system, this time as a post-treatment process to lime/magnesium salts method, is presented in Section 4.14.

4.6 Reflections on Research by Ayoub et al 1992 on Emulsified Oils and Alkaline Wastewaters

It is not surprising that the findings of Ayoub et al (1992) confirmed in many ways the success of the earlier work by Mawson (1970), Simpson (1986) and the QUT/Simpson Research (Shanableh et al 1995). Mawson (1970) investigated the following wastewaters and process waters:

1. Municipal wastewater
2. Greywater
3. Abattoir wastewater

Simpson (1986) investigated the following:

1. Municipal wastewater
2. Greywater
3. Tannery wastewater
4. Milk processing water
5. Anodizing effluent
6. Quarry process water
7. Ready mixed concrete wash water
8. Coal mine water
9. Meatworks wastewater.

The QUT/Simpson Joint Research (Shanableh et al, 1995) investigated the two different municipal wastewaters, which had different background colours.

On reflection, the wide range of effluents and process water the writer worked on demonstrates the versatility of the lime/seawater/magnesium salts treatment method. Hence, it is not surprising that Ayoub et al (1992) found it effectively treated emulsified oily wastewater. Oily and fatty wastewater is a challenge to treat by biological means. In the writer's experience, and in discussions with others, the oil and grease effectively coats organic matter, which inhibits biological breakdown.

4.7 Lime/Seawater/Magnesium Salts Treatment of Greywater

The writer reflects on the performance results of experimental work with lime/seawater/magnesium salts and greywater. Mawson (1970) reports that earlier tests on laundry water produced almost clear water clarity. The clarification process was both rapid and positive. The writer reported (Simpson, 1986) the following results:

1. Jar test 2 (JT 2) using greywater - the flocs formed rapidly, settling velocity was 0.85 mm/s and a high clarity resulted
2. J T 15 using greywater – 5% seawater was the most effective and the settling velocity was 0.43 mm/s
3. Batch test 9 (BT 9) using greywater – a clear liquor zone was apparent within two minutes and the batch test was considered to be successful from all points of view.

Greywater has a low organic content which favours chemical treatment. Jeffersen and

Solley (1994) report a greywater BOD of 160 mg/L. This can be compared with a BOD_5 range for raw wastewater of 100-500 mg/L.A mean greywater BOD_5 of 128.9 mg/L is reported by Veneman and Stewart, (2002).

The phosphorus content of greywater is conducive to lime/seawater/magnesium salts treatment. Mawson (1970) reported that lime treatment precipitates P and additional P, in the form of magnesium ammonium phosphate, is precipitated when seawater is used. His jar tests showed the elimination of the foaming of effluent usually associated with phosphatic detergents. Jeffersen and Solley (1994) report a typical P concentration of greywater is 8 mg/L. Veneman and Stewart (2002) report the ortho-phosphate range of <0.5 to 3.6 mg/L.

Colour reduction is achieved. Mawson (1970) considered that the colour removal was thought to be the reaction of sodium ions, under elevated pH conditions, on proteins and other organics present.

There is a high die-off of harmful bacteria (Mawson, 1970). The writer recorded faecal coliform reductions of >99.9% (Simpson, 1986). The joint QUT/Simpson study recorded the same percentage reductions.

4.8 Advantages of Lime/Seawater/Magnesium Salts Treatment

The rationale for the increasing popularity for chemical wastewater treatment plants pertains to the advantages in Table 8.19. The writer considers that this list of advantages is comprehensive and therefore worthy of further comment and reflection.

Table 8.19: Review of Advantages of Lime/seawater/magnesium Salts Treatment

Associated Advantages[1]	**Reflections and Comments by Simpson[2]**
Relatively tolerant to toxic compounds	Not impacted, when compared with biological processes.
Amenable to automatic control	Simple flow control and dosing system required, which can be automated.
More predictable performance	Far more tolerant to fluctuating loads, when compared with biological treatment methods.
Can be operated on a on-off basis without the need for a settling in or acclimatization period	Very limited commissioning period required when turned off.
Almost complete removal of P	High reactive P and Total reductions

Significant particle removal	High settling efficiency, enhanced by chemical coagulation and flocculation.
Very good removal of heavy metals, bacteria and viruses	High pH enhances high heavy metals and bacterial removals.
Can be designed to produce useful by-products	When using the lime/magnesium salts method the final effluent is amenable to land irrigation and other reuse applications. Sludge has reuse potential.
Lower land space requirement	One to two unit processes reduces land requirements.
Lower installation costs	Unit operations of a medium sized plant would be limited to screening, reaction vessels and sludge disposal. No tertiary and disinfection processes needed. Reduced costs when compared with conventional plants.

Notes:

1. Based on a review by Semerjian and Ayoub (2003)
2. Based on the experimental findings of Simpson (1986) and Shanableh et al (1995) and design and operational experience of Simpson

4.9 Treatment Economics

Earlier work on lime/seawater/magnesium salts treatment gave mention to the fact that the process was economic. Odegaard (1989) reports that the process is relatively simple to operate and it was economically acceptable**.** Odegaard (1992) mentions the low energy requirements and low operating costs.

Ayoub (1994) reports that the lime/seawater process introduces appreciable savings in chemical costs, over other chemical enhanced processes. The cost of chemical wastewater treatment can be high. The writer investigated the feasibility of primary settling enhancement using conventional chemicals and polymers at the former Dalby Town Council wastewater treatment plant, in the early 1990s. The process was feasible but, due to the daily flow, the chemical costs were not attractive.

Semerjian and Ayoub (2003) reported that there were appreciable savings in chemical coagulant and disinfection chemical costs. They recommended that in-depth economic assessments were required into the comprehensive evaluation of the capital, operational and maintenance costs incurred by the lime/magnesium salts treatment process. The writer would certainly concur with this recommendation.

The writer maintained in his study (Simpson 1986) that the process was amongst the most economic wastewater treatment methods. This was on the following basis:

1. Seawater was free
2. Solar salt brine was a waste product and readily available
3. Lime was one of the cheapest chemicals
4. It is essentially a single effluent treatment process
5. Readily available lime waste products could be used. The writer used burnt lime or kiln lime for his sludge study (Simpson 1977) since it was a waste product and it did not contain toxic materials.

The resultant lime/magnesium salts sludge is readily de-watered and in a suitable state for land application.

On reflection, there are no apparent reasons why the lime/magnesium salts process could not be one of the most economically competitive treatment methods. Full scale treatment plant trials would be required to confirm the low cost of the lime/magnesium salts process, for the following reasons:

1. Confirm the choice of chemicals (form of lime and magnesium)
2. Optimise dose rates
3. Optimise the chemical mixing times
4. Optimise the flocculation and coagulation times
5. Optimise the settling times.

5 Conclusions

In summary, the aim and objectives of this Portfolio have been achieved in that the lime/seawater/magnesium salts wastewater treatment process is viable and more research work is suggested.

5.1 Lime Seawater Testing and Treatment

Value of Jar Testing

The writer found that jar testing was the most practical method of determining the chemical conditions required for coagulation and then flocculation (Simpson, 1986). This has since been confirmed during the lime/solar salts brine work by Shanableh et al (1997).

Treatment Merits

The treatment of wastewater using lime to raise the pH and magnesium salts to form a positive magnesium hydroxide floc, has considerable merit in terms of pollutant removal and sludge management.

A continuous pilot trial using Lime/Dominion Salts waste gave up to 81% BOD_5 and up to 92 % suspended solids reductions (Simpson, 1986).

The specific merits with lime/seawater/magnesium salts experimental work are the high phosphorus removal (96%), high suspended solids (96%), COD (96%), acceptable BOD_5 (76%), colour reduction and high disinfection quality (99.9% E coli), as reported by Shanableh et al (1995).

The lime/seawater/magnesium salts process, when applied to greywater, produces a very clear liquid with minimal odour (Mawson, 1970 and Simpson, 1986). The lime/seawater /magnesium salts process results in substantial colour reduction in meat wastes effluent (Mawson, 1970 and Simpson, 1986).

The reduction of Total N by the lime/seawater/magnesium salts methods is limiting in that reductions are in the range of 40-50%. It is to be noted that Total N reduction was outside the scope of the Simpson (1986) study. Shanableh (1998) reported 43% TKN reductions. An Ammonia Nitrogen reduction of up to 82.6% was achieved by Shin and Lee (1998), using bittern at a pH of 10.5.

It has been established that lime/magnesium salts treatment is viable and economic (Simpson 1986; Shanableh 1997; Odegaard, 1989). Sea salts brine is a rich source of magnesium, it reduces effluent volumes and it is relatively free of sodium and chlorides, when compared with seawater.

Lime/Seawater versus Biological Treatment

The use of lime/magnesium salts has definite benefits when compared with biological wastewater treatment, which include the following:

1. The lime/seawater/magnesium salts method of treatment is not inhibited by very cold and very hot climates. Climatic changes and extremes can impact on the performance of biological treatment processes
2. There are no problems with the lime/seawater/magnesium salts treatment when dealing with of non-biodegradable and or toxic wastewater (Semerjian and Ayoub, 2001)
3. Sludge bulking and foaming are major problems with biological wastewater treatment (Hartley, 1987). The lime/seawater/magnesium salts treatment method eliminates this important operating problem.

Chemical treatment of wastewater generally has its benefits but the usual concerns, based on the writer's experience, include the following:

1. Cost of the chemicals or polymers

2. Sludge volumes generated
3. Sludge compaction
4. Sludge dewatering
5. Sludge aging, as determined by the degree of stabilisation obtained
6. Suitability for safe disposal.

5.2 Lime Seawater Sludge

Lime Wastewater Sludge

The burnt lime and wastewater sludge study by Simpson (1977) concluded that lime stabilised wastewater sludges could be processed into a soil conditioner/fertilizer and the finished product was viable, economical and environmentally acceptable. This study was undertaken for the purpose of gaining approval for the New Zealand Ministry of Agriculture. This study was in several respects a useful forerunner to the Simpson (1986) experimental lime/seawater/magnesium salts project. The use of magnesium salts is preferable to seawater so the final effluent can be readily discharged to saline water and reused for land irrigation. The resulting sludge is chemically and biologically stable. The writer found that burnt lime and sludge mixing results in complete pathogenic organisms die-off and low moisture contents, at a design mixing ratio of 3 (lime): 1 (sludge) (Simpson, 1977).

The lime/magnesium salts process, when combined as pre or post unit process with biological treatment, is considered to be worthy of further investigation, in order to achieve Total Nitrogen removal and more complete wastewater treatment.

Sludge Management and High pH

Shanableh (1997) reported that the high sludge production and high residual pH were major limitations of the lime/seawater/magnesium salts methods of treatment. The writer sees that these are more minor disadvantages, which could be turned into advantages. The resultant sludge is chemically and biologically stable and readily dewatered. In situations where there are acidic clay soils, the resultant sludge can be reused to achieve agricultural and horticultural, improvements in yield (Simpson, 1977). This sludge has high N, P and organic matter (Shanableh, 1997).

The writer's experimental work on high pH sludge found that the CO_2 from the atmosphere reduced pH levels to acceptable and usable levels in the relatively short term (Simpson 1977).

Reuse of Lime/Seawater Sludge

Shieh and Roethel (1989) examined the physical and chemical behaviour of stabilised wastewater sludge blocks in seawater. Dewatered wastewater sludge (20% solids) was successfully stabilised into block form by using fly ash, gypsum, lime and Portland cement. The results indicated that stabilised wastewater blocks maintained their structural integrity in seawater and would be classified as a non hazardous material. A potential use of the blocks is for artificial reef construction.

There would be merit in following up on the outcomes and developments of the above reuse applications of lime stabilised sludge.

Lime/Wastewater Sludge Project

The writer's study for *Natumix Fertilizers Ltd* (Simpson, 1977) was unique for the following reasons:

1. It was a world leading study at the time
2. Butter fat contents of milk were increased considerably, which is of major importance in New Zealand
3. The condition and yield of citrus trees were improved
4. Grass could be grown on highly acidic soils.

5.3 Conceptual Small Scale Lime/Magnesium Salt Treatment

The writer has developed a potentially viable total wastewater treatment concept for a resort facility or a small community subjected to fluctuating loads, as shown in Table 8.20.

Table 8.20: Conceptual Lime/Magnesium Salts and Wetland Polishing Treatment

Unit Process	Treatment Objectives	Comments
Primary screening or maceration pumps	Improve treatability	Advisable pre-treatment
Dual reaction and settling vessels	Flash mixing, coagulation, flocculation effluent separation	BOD, colour, disinfection, P removal and some Total N reduction.
Reed/gravel bed (Stage 1)	Additional BOD removal and nitrification	Needed for Total N reduction. 500 mm deep bed with planting and air vents.
Reed/gravel bed (Stage 2)	De-nitrification	Needed for further Total N reduction. 700 mm deep bed, with no planting, to achieve anoxic conditions.

Effluent discharge or reuse	Effluent to land disposal, estuarine and marine waters.	Effluent would be in a suitable state for reuse and land, estuarine and marine disposal
Sludge storage and drying	Possible reuse application	Suitable for beneficial reuse

The modes of operation of the above concept for dealing with fluctuating wastewater loads are:

1. Low flows or "normal" or design flows – single screen or macerating pump, one reaction and settling vessel, two stage reed /gravel bed system
2. Higher flows – use two screens or macerating pumps and two stage reed /gravel bed system.

Refer to the conceptual lime/magnesium salts and two stage reed/gravel treatment systems in Figure 8.4.

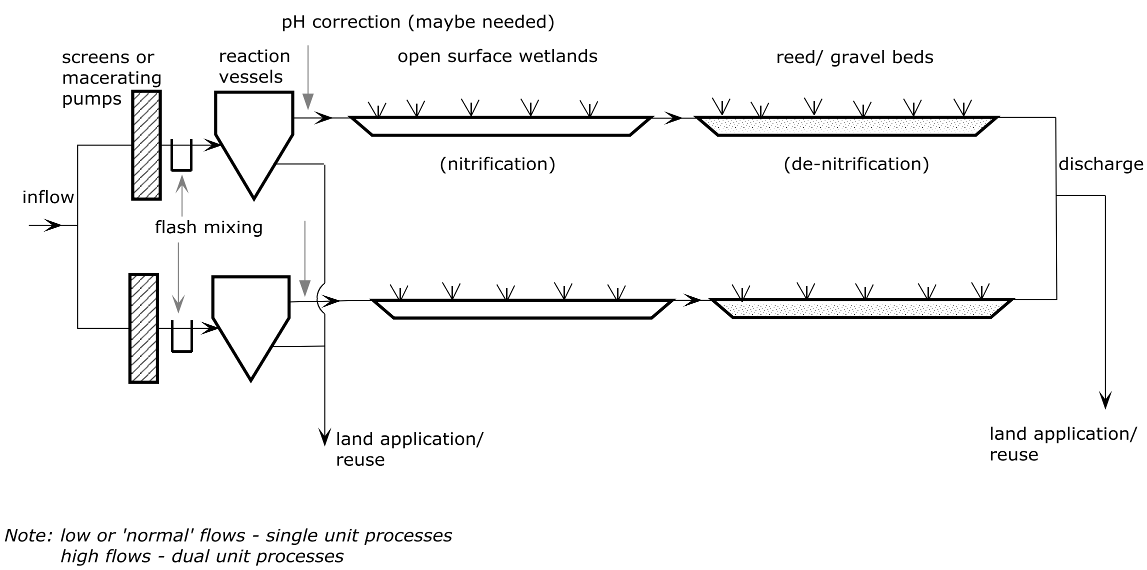

Figure 8.4: Conceptual Lime/Magnesium Salts Treatment and Two Stage Reed/Gravel Bed

6 References

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