DESIGN AND MANAGEMENT OF TASMANIAN SEWAGE LAGOON SYSTEMS

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NOVEMBER 1996

PRINTING

Printed by the Printing Authority of Tasmania
OCTOBER 1996

PRODUCED AND PUBLISHED

This document has been produced by DELM and DPIF and published by Environment Tasmania, Environment and Planning Division
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SUMMARY

There are approximately fifty "permitted" sewage lagoon sites located throughout Tasmania and about 70% of them do not comply with regulatory standards. Therefore, discharges from these sites potentially contribute to a poorer quality of receiving waters.

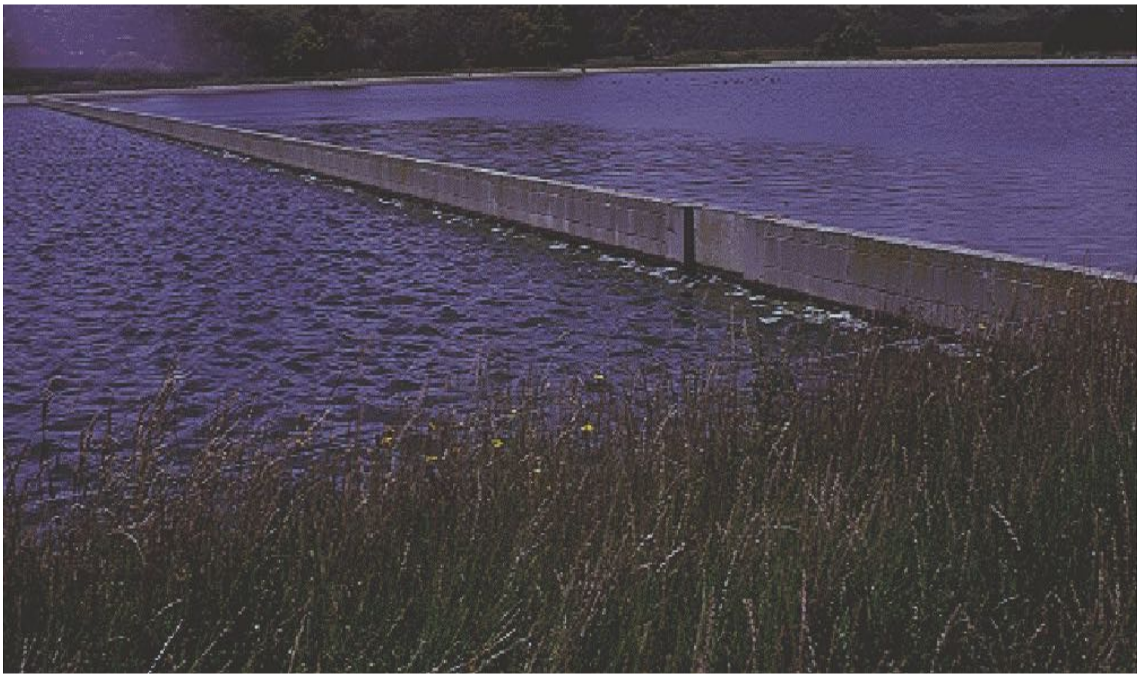
A joint sewage lagoon project, the "Sewage Lagoon Performance Improvement Program" (SLPIP), was carried out by the Department of Primary Industry and Fisheries - Land and Water Resources Division (DPIF - LWRD) and the Department of Environment & Land Management - Environmental Management Division (Environment Tasmania) (DELM - ET), under a Management Committee which consisted of DPIF - LWRD, DELM - ET and the Local Government Association of Tasmania (LGAT). SLPIP was funded by the Federal and State Governments. Financial assistance was provided also by the Huon Valley, Northern Midlands and Break O' Day Councils. The objective of the 2 year project was to develop principles for the design, operation and maintenance of sewage lagoons in Tasmania and to assist rural Councils who have limited technical expertise.

As a result of the project, this manual has been produced as a guide to engineers, technical personnel, and operators concerned with the design and operation of sewage lagoon systems serving Tasmania's towns. The principles described are applicable to the design of new systems and upgrading of existing systems.

The study found that the principal reason discharges of bacteria from lagoons, do not comply with regulatory standards, is due to lagoon design. In this study general design principles are discussed and compared with observations made. For cost effective designs that perform environmentally in Tasmania, an assessment of the lagoon site is essential. This requires wind exposure and the hydraulic and organic loads on the lagoon be measured whenever possible. The manual describes how to use these measurements in design equations.

For situations of particular complexity, the designer may need to seek additional information from the reference/bibliography sections and/or DPIF - LWRD and DELM - ET. It is recommended that designers with limited experience with sewage lagoon treatment read the whole manual before proceeding to design.

This manual is offered as a guide only. All care is taken in the presentation of the results of the research work and design recommendations. However, the responsibility for the proper function of any lagoon system rests with the designer and operator of that system.



ACKNOWLEDGMENTS

This project was undertaken by a project team consisting of Greg Dowson, Daniel,Ray and Dao Norath. The Management Committee consisted of Alan Harradine, Brendan Boon - DPIF - LWRD, Dr Ian Woodward, Dr J McCambridge and Jamie Wood - DELM - ET and John Hingston - LGAT. Dr Mike Johns from the University of Queensland was commissioned to provide expert advice on the project and, in addition, Chapter 2 of this manual has been adapted from one of his papers, as referenced.

This project was one of a kind in Australia. The project would not have been possible if not for the funding from the Federal and State Governments. Their contributions are acknowledged. Thanks also to those Councils which contributed.

We would like to express our gratitude to the Municipalities that participated in the study. These were Northern Midlands, West Tamar, Meander Valley, Central Highlands, Central Coast, Break O' Day and Huon Valley. The staff at the sites chosen were particularly co-operative and helpful, especially Ian Wickham and Tony Branch. The supply of valuable analytical data by Launceston City Council - Ti Tree Bend Laboratory not generated as part of the study is greatly appreciated. Thanks are also due to John Anderson of Consulting Environment Engineers and Jonathan Crockett of Gutteridge Haskins and Davey (Melbourne) for their ideas and review of a draft of this manual respectively. Thanks to Ken Herbert of Water Training Centre for his comments on chapter 8 (Operations and Maintenance).

Thanks also to Dr Mike Johns who acted on behalf of the Co-operative Research Centre for Waste Management and Pollution Control-Queensland Node, and provided valuable assistance in relation to monitoring advice and analysis of dye tracing data.

DEFINITIONS OF TECHNICAL TERMS

Activated Sludge: A dark brown suspension of microbial flocs produced by the aeration of sewage; prolonged aeration in admixture with this sludge effectively purifies sewage.

Aerobic: Condition of freely available oxygen (in aqueous state) for biological processes. Oxygen is provided by algal photosynthesis and atmospheric diffusion.

Anaerobic: Condition where oxygen is totally depleted (in aqueous state) and nitrate is absent.

Anoxic: Condition where oxygen is totally depleted in aqueous state and nitrate is present.

Average Dry Weather Flow (ADWF): The average flow measured over a period of seven consecutive days, the period to be chosen such that rainfall is less than 0.25 mm/d, infiltration of stormwater into the sewerage system is at a minimum and any abnormal influences such as public holidays are excluded. In the study, however, the measured dry weather flow was defined as the flows measured when less than or equal to a total of 2 mm of rain fell over the day prior and day during the flow measurements.

Baffle: Board(s) or solid partition submerged in a sewage lagoon to disperse or direct the flow in the lagoon.

Biochemical Oxygen Demand (BOD_5): The mass of oxygen consumed by a unit volume of liquid (expressed as mg/L) during biochemical oxidation in the course of five days at 20ºC.

Biological Filter (also known as a trickling or percolating filter): A porous bed of suitable, graded, inert material. Bacteria and other organisms flourish on the surface of this material and bring about oxidation of the organic matter in the settled sewage applied to the filter.

Chemical Oxygen Demand (COD): A measure of the organic content (polluting strength) of domestic and industrial wastes. The COD test characterises a waste in terms of the total quantity of oxygen required for oxidation to carbon dioxide and water.

Conductivity: A measure of the ability of water to carry an electric current. This is used as a measure of soluble salts.

Domestic wastewater: Wastewater derived from human origin comprising faecal matter, urine and liquid household waste from sinks, baths, and basins.

Duty: Capacity of a pump, for example a flow of 15 L/s at 20 m head.

Effluent: The liquid discharged from a treatment unit or system and usually qualified according to the type of treatment received, e.g. septic tank effluent or sand filter effluent.

Equivalent Person (EP): The number of persons who would contribute the same quantity and/or quality of domestic sewage as the establishment or industry being considered.

Filter Medium: The material, such as hard clinker or broken stone or plastic, with which a biological filter is filled.

Final Effluent: The liquid discharged finally from a sewage treatment works.

Flow proportional composite (weighted) sample: A sample which consists of combined sub-samples taken proportional to the flow.

Greywater: Kitchen, laundry and bathroom waters.

Humus Tank: A tank, through which biological filter effluent is passed, to settle solids which should be removed at frequent intervals.

Hydraulic Retention Time: The period a set volume unit is within a body of fluid.

Infiltration/Inflow: Groundwater infiltration and stormwater inflow into a sewerage system.

Influent: The liquid discharged into a treatment unit or system, usually sewage.

Inlet: The structure which carries the waste into the sewage treatment plant.

Nutrients: Compounds required for growth. Nitrogen and phosphorus are the most common nutrients.

Outfall: The final point of discharge from an industrial plant into receiving water.

Outlet: The structure which carries the sewage out of a sewage treatment plant or unit in a plant.

Pathogens: Natural organisms, such as bacteria and viruses, which can cause disease in animals (and plants).

Preliminary Treatment: The pre-treatment processes of screening, grit removal and flow balancing and splitting. Sewage is prepared for subsequent major treatment.

Primary Treatment: The processes that remove a substantial amount of suspended matter but little or no colloidal and dissolved matter.

Receiving Waters: Water into which sewage is discharged.

Rising Main: A main through which sewage or water is pumped under pressure.

Secondary Treatment: The processes that remove or reduce suspended and dissolved solids and colloidal matter causing the reduction of organic material.

Septic Tank: A tank through which sewage is passed to settle solids which are retained to undergo digestion by anaerobic bacterial action.

Sewage: The discharge from domestic and sanitary appliances (e.g. toilets, wash waters) within communities (often referred to as wastewater).

Sewage Lagoons: A system of lagoons designed to treat domestic and industrial wastewaters.

Sewage Treatment Plant: An industrial plant for reducing the strength of the water-waste mixture known as sewage. Also known as wastewater treatment plant.

Sewer: A pipe carrying sewage.

Sewerage: A system of pipes, access holes, pumps, treatment facilities and other items for handling sewage.

Short-circuiting: The under-utilisation of the lagoon or treatment facility through selective flow which results in dead spots and a reduction in the HRT compared to the theoretical HRT.

Sludge: The slurry which is formed in a sewage treatment plant or lagoon by deposition of settlement solids.

Storm Overflow: A device, on a combined or partially separate sewerage system, introduced for the purpose of relieving the system of flows in excess of a selected rate, so that the size of the sewers downstream of the overflow can be kept within economical limits, the excess flow being discharged to a convenient watercourse or holding facility. Such overflows are often required to be licensed as a discharge in NSW.

Supernatant Liquor: The layer of liquid overlying the settled solids which have separated from it.

Suspended Solids (SS): The solids which are suspended in a sewage or effluent, otherwise known as non-filterable residue (NFR).

Tertiary, Advanced or Polishing Treatment: Those processes that treat the effluent from secondary treatment to further reduce residual solids, dissolved organics and nutrients.

Trade Waste: The fluid discharge, with or without matter in suspension, resulting wholly or in part from any manufacturing process and including farm and research institution effluent.

Turbidity: A measure of the dispersion of colloidal material.

LIST OF ABBREVIATIONS

ADWF	=	average dry weather flow in sewers
BOD_5	=	biochemical oxygen demand (determined over 5 days)
cfu	=	coliforms (a type of bacteria) measured by standard units
COD	=	chemical oxygen demand
DELM	=	Department of Environment and Land Management
DO	=	dissolved oxygen
DPIF	=	Department of Primary Industry and Fisheries
E coli	=	Escherischia coli
F strep	=	Faecal streptococci
HRT	=	hydraulic retention time (days)
k	=	first order kill rate constant
LGAT	=	Local Government Association of Tasmania
N	=	total nitrogen
NH_3-N	=	ammonia-nitrogen
NO_2-N	=	nitrite-nitrogen
NO_3-N	=	nitrate-nitrogen
P	=	total phosphorus
pH	=	negative logarithm of molar hydrogen ion concentration
PO_4-P	=	phosphate (ortho)
PWWF	=	peak wet weather flow in sewers
Sol BOD_5	=	soluble biochemical oxygen demand (determined over 5 days)
Sol COD	=	soluble chemical oxygen demand
SLPIP	=	Sewage Lagoon Performance Improvement Program or sometimes referred to as Sewage Lagoon Study or simply " the study ".
SRT	=	solids retention time (sludge age)
SS	=	suspended solids (or non-filterable residue - NFR)
SSF	=	slow sand filter
T coli	=	total coliform
TKN	=	total Kjeldahl nitrogen
TWL	=	top water level
WWTP	=	wastewater treatment plant
20:30	=	20 mg/L BOD_5 and 30 mg/L SS effluent standard

LIST OF UNITS

EP	=	equivalent person
g/c/d	=	grams per capita per day
g/m^3	=	grams per cubic metre
kg/ha.d	=	solids loading rate in kilograms per hectare per day
$kg/m^2.d$	=	solids loading rate in kilograms per square metre per day
kL/d	=	kilolitres (cubic metres) per day
L/c.d	=	litres per person (capita) per day
L/EP.d	=	litres per equivalent person per day
L/s	=	litres per second
mg/L	=	milligrams per litre
m/h	=	metres per hour
$m^3/m^2.d$	=	surface loading rate in cubic metres per square meter per day
m/s	=	metres per second
µg/L	=	micrograms per litre

CHAPTER 1

INTRODUCTION

Contents

Table

1.1 General

In Tasmania there are some 50 “permitted” lagoon systems (i.e. regulated by planning permit) located throughout the State. Most of these serve a population of less than 5,000 persons. Some 70% of them are not meeting the current regulation discharge limits (refer to Chapter 5). These non-complying lagoons are potentially impacting on the water quality and hence the protected environmental values of the receiving waters.

This manual is written to assist Councils to understand the design, operation and maintenance of their lagoon systems. Design works have been carried out in Tasmania but there are no real data to support them. This is the first major co-ordinated investigation carried out on sewage lagoons in Tasmania to establish optimal design and operating principles.

A problem which designers confront is that there are many ways of designing sewage lagoon systems but there is no one "correct" way. Our task is to find a simple design procedure that is economical and reliable to provide a performance that consistently produces effluent of a quality that falls within the environmental and design expectations for the system. It is a well known fact that lagoon performance is very much influenced by local climatic conditions, organic loading and the hydraulic behaviour of the lagoon system. To simply import a design is potentially a hit-and-miss exercise.

1.2 Manual Objectives and Scope

This manual provides a concise overview of sewage lagoon systems for Tasmania through a discussion of factors affecting treatment, process design principles and applications, aspects of lagoon physical design and construction, nutrient and algae removal processes.

Chapter 1 discusses the definitions used by different people associated with sewage lagoons. The advantages and disadvantages are listed and the different types of lagoons and their functions briefly discussed.

Chapter 2 expands on the different types of lagoon systems and the biochemical reaction that occurs in each. The advantages and disadvantages of each type of lagoon are listed in a table for easy reference. In addition a diagram is provided to assist easy visualisation of each lagoon type.

Chapter 3 outlines the existing Tasmania lagoons, their general design principles, cell layout, inlet and outlet arrangements. It also goes on to discuss their performance and compliance in terms of the State's discharge limits and the need to upgrade.

Chapter 4 discusses the different factors affecting lagoon treatment and performance. This includes shapes, cell layout, wind, temperature, flows, seasons, inlet/outlet and baffling.

Chapter 5 is the most up-to-date and comprehensive paper on environmental legislation and policies relevant to sewage lagoon discharges in Tasmania when this manual went to press. This chapter is recommended reading for all people concerned with the legal obligations associated with the design and operation.

Chapter 6 outlines recommended design principles for sewage lagoons in Tasmania. It incorporates our research findings over the last two years. Examples employing the loading method and other equations are shown with easy to follow steps. Discussions on expected performance of these designs are included.

Chapter 7 covers the principles and practices of the physical construction of lagoons. This section is brief and draws on the American experience. It is intended for novice designers and technical people.

Chapter 8 discusses the operation and maintenance of sewage lagoons in Tasmania. It covers the routine maintenance required by operators as well as the monitoring of key parameters to ensure the optimum performance of the lagoon system. A "solutions to problems" table for the effective management of lagoons is also included.

Chapter 9 is a brief manual on the operation of software, titled "Pondcal", developed as part of the study to assist in lagoon design, operation and management. Insertion of theoretical data into the program enables the prediction of performance under different design and load scenarios.

1.3 Types of Lagoon

Lagoons are one of the oldest, simplest and lowest cost forms of treatment for domestic waste. Lagoons are used throughout the world for treating domestic and industrial waste. There are approximately 7000, 1400, 1400, 1000, and 1000 lagoon systems in the USA, Germany, France, Great Britain and Canada respectively. Lagoon systems are also used extensively for agricultural industries such as piggeries, tanneries and abattoirs.

Mara et al *(1992)* described sewage lagoons as follows:

"In simple terms, sewage lagoons are impoundments into which wastewater flows in and out after a defined retention period. Treatment relies solely on the natural processes of biological purification that would occur in any natural water body. No external energy, other than that derived from sunlight, is required for their operation. Treatment is optimised by selecting appropriate organic loadings, retention periods and lagoon depths, to promote the maximum growth of organisms beneficial to the treatment process."

When properly designed and operated, lagoon systems may have advantages over conventional systems, some of which are listed below (Adapted from Mara et al (1992)):

1. Simple construction;
2. Low construction costs (provided suitable land is available and cheap);
3. Little or no machinery, resulting in simple plant operation with low maintenance costs;
4. High microbiological quality of the final effluent so that additional disinfection is not needed;
5. Lagoons reduce the bio-availability of some of the nutrients and organics, and hence reduce the impact on the receiving water;
6. Due to their long retention periods, lagoons can readily accept fluctuating hydraulic loads and can therefore cope well with storm water without washout of biomass;
7. Lagoons are ideally suited to coastal tourist locations because increased summer temperatures raise treatment efficiency and therefore allow an increased loading;
8. Lagoon systems incorporate integrated sludge treatment. If anaerobic lagoons are utilised, much of the sludge is digested, especially during the warmer summer months; and
9. Lagoons can be used to treat many industrial waste waters and provide high removals of heavy metals.

The main disadvantages of lagoon systems are:

1. They require a relatively large area of flat land compared to more conventional treatment processes such as trickling filters or activated sludge;
2. Lagoons have a reputation for producing an effluent with higher SS and BOD concentrations than conventional treatment processes due to the presence of a high concentration of algae;
3. The potential for odour if overloaded or intermittent odour due to overturns;
4. They are influenced more by seasonal variations and climatic conditions;
5. Operators have less control over the treatment process; and
6. There may be difficulties in achieving reliable nutrient removal.

Sewage treatment lagoons are generally classified according to the following criteria:-

The nature of biological activity in the lagoon.
The form of aeration supplied, if any.

The four main types of lagoons are given in Table 1.1 with a synopsis of their primary characteristics.

Table 1.1 Sewage lagoons classification (Adapted from Johns (1991))

Lagoon Type	Biological Activity	Typical Depth (m)	Type of Aeration
Aerobic (Oxidation) (Maturation)	Aerobic	0.9 - 1.5	Natural
Anaerobic	Anaerobic	> 4	Avoided
Facultative (Stabilisation)	Anaerobic/ Aerobic	1.2 - 2.5	Natural
Aerated	Aerobic	3 - 4	Mechanical

For existing Tasmanian lagoons refer to Chapter 3 of this manual.

The above terms are mainly defined according to oxygenation conditions in reaction to bacteria existing within the lagoon or how the designer intends the lagoon to perform. However, in practice, the dissolved oxygen present in any lagoon is largely dependent on the strength of organic loadings. For simplicity, when a high organic load flows into the lagoon system, anaerobic conditions can be expected in all of the lagoons where the oxygen consumption rate exceeds that of re-aeration rate. In contrast, if the lagoon receives a low organic loading, aerobic conditions can prevail through the lagoon system. Other than organic loading, physical and chemical factors can also affect the oxygen presence in the lagoon. So the terms used in Table 1.1 largely explain the general concept of lagoon design associated with oxygen presence in the lagoon system.

When considering the treatment process, however, lagoons are generally classified as follows:-

(1) *Primary lagoon* refers to the first lagoon(s) receiving raw sewage and this term is used whenever there are two or more connected in the series.

(2) *Secondary lagoon(s)* is the term for the lagoon receiving effluent from the primary waste stabilisation lagoon. This lagoon is also termed a maturation or polishing lagoon; and there may be more than one of these operating in series.

The utilisation of the various terms may be a result of different focus by people. For instance, for those whose interests focus on biochemical reaction within lagoons, the terms facultative, aerobic and maturation lagoons are usually used in order to reflect such mechanisms. Others, who might be interested in the engineering aspect of a lagoon system, e.g. level of treatment, use the second group of terms, i.e. primary and secondary lagoons. The varying use of terms occurs because the latter group is interested in lagoon processes, while the first group focuses on biochemical science. Nevertheless, the two groups of terms are often used interchangeably among people depending on their personal interest.

For the purpose of this manual the terms will be used interchangeably to reflect the different focus of the discussions. For example, if lagoon order in a series is referred to the terms primary and secondary shall be used; but if biochemical reaction in the lagoon is referred to, terms in Table 1.1 shall be used.

A popular term used by researchers and engineers for lagoon is pond. We shall generally stick with lagoon throughout this manual for consistency.

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CHAPTER 2

CLASSIFICATION OF LAGOONS

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A literature search carried out on this topic revealed many authors and approaches, some of whom are listed in the references. Dr Mike Johns, who is regarded as an authority on this subject in Australia, presented the science of lagoon treatment clearly; and it is considered his work is suitable for the purpose of this manual. Therefore, this chapter is adapted with permission from Dr Mike Johns' paper (reference 4). Readers are referred to the course papers for additional information not covered here.

2.1 Aerobic Lagoons

Aerobic lagoons are designed to be naturally aerated so as to maintain an aerobic population of micro-organisms to degrade wastes, oxidise nutrients and eliminate pathogenic micro-organisms. They are also called oxidation lagoons or maturation lagoons. The characteristics and overall process of aerobic lagoons are detailed in Figure 2.1.

The crucial factor in operating aerobic lagoons is keeping them aerobic! The stabilisation of wastes by bacteria consumes dissolved oxygen. In naturally aerated, aerobic lagoons, the dissolved oxygen stocks are replenished by two mechanisms. Firstly, from the activity of photosynthetic micro-algae and, secondly, from surface aeration, which is greatly improved by wind action. The consumption of oxygen by bacteria is essentially proportional to lagoon volume, whereas oxygenation of the lagoon is related to lagoon surface area. For this reason, aerobic lagoons must be shallow (1-1.5 m) to prevent anaerobic conditions developing and are typically rectangular with the lagoon aligned to the prevailing wind to maximise wind aeration.

The aerobic population is diverse. Micro-algae play an important role in generating oxygen by photosynthesis during daylight hours and utilising CO_2 for growth. During the night they continue to respire but cease to photosynthesis and therefore consume oxygen without replacing it. Bacteria and multicellular organisms consume nutrients and can eradicate viruses and pathogenic micro-organisms, if present. The latter is an important function of maturation lagoons.

An evaluation of aerobic lagoons is summarised in Table 2.1. These lagoons are typically placed last in a lagoon series.

The actual organic loading rate for aerobic lagoons is determined by the strength and breakdown ability of the waste. Typical BOD loading rates in Tasmania, based on literature search and research, ranges :-

40-70 kg BOD/ha/d conventional aerobic
< 15 kg BOD/ha/d maturation lagoons

There is evidence that higher loading rates are possible in certain locations where natural aeration is good.

Table 2.1: Aerobic Lagoons:- Strengths and Weaknesses

STRENGTHS

Cheap (to construct & operate)
Achieve significant pathogen/virus reduction
Can handle shock load

WEAKNESSES

Requirement for large land area
Can handle only low organic loadings
Generation of algal biomass
Limited oxygen supply capability
Night sag in dissolved oxygen
Temperature-sensitive

2.2 Anaerobic Lagoons

The characteristics and overall process of an anaerobic lagoon is given in Figure 2.2. The salient feature of this type of lagoon is that the lagoon environment is maintained in an anaerobic (oxygen-free) state.

Anaerobic lagoons are typically used as the first step for treating strong organic wastes, particularly those from primary processing industries, but are not usually employed in domestic wastewater systems. They should be followed by aerobic biological treatment systems.

Essentially, two diverse groups of micro-organisms exist in this environment and the activity of both groups is necessary for healthy lagoon performance. The first group are called acid-forming bacteria. They catalyse the first set of reactions indicated in Figure 2.2 in which complex organic compounds in the effluent (lipids, proteins and polysaccharides) are degraded into lower molecular weight acids and alcohols, principally the volatile fatty acids, acetic (C2), propionic (C3) and butyric (C4) acids. Very little methane is generated in this process. Carbon dioxide and hydrogen are liberated, and the nitrogen and sulphur in proteins are converted to ammonia (NH_3) and hydrogen sulphide (H_2S) or metal sulphides respectively.

The activity of the acid-forming bacteria is extremely important, since methane is only generated from a small variety of intermediates (mainly acetic acid and CO_2). Acid formation occurs at a faster rate than the second, methane-formation step.

The product of the first stage is an effluent which is (a) acidic (b) odorous due to volatile fatty acids formation (c) suitable for the action of methane-forming bacteria.

The second group of organisms are the methane-forming bacteria. This population of organisms is strictly anaerobic, slower growing and more sensitive than the acid formers. Methane formation is the critical, rate-determining step in anaerobic systems. Consequently, anaerobic lagoon design should be targeted at meeting the requirements of this group. Given appropriate conditions, the methane-formers catalyse the conversion of the acids generated in the first step into largely gaseous products, including methane and CO_2 (Figure 2.2). The gas formation is important in helping mix the lagoon contents.

The advantages of properly operating anaerobic lagoons are listed in Table 2.2. They can be summarised as the ability to convert soluble and insoluble pollutants into gaseous products with minimal sludge formation. The disadvantages include the slow rates of activity of anaerobic systems, their sensitivity to environmental factors and the odorous products produced in an anaerobic environment.

Table 2.2: Anaerobic Lagoons:- Strengths and Weaknesses

STRENGTHS

Low biomass formation
Pollutants are converted to gaseous products
Can handle high strength effluent. Typically 60-80% average BOD_5 reduction.
Crust formation reduces odour release, heat loss and oxygen penetration

WEAKNESSES

Low microbial population and growth rate
Vulnerability to shocks (pH, sudden load changes)
Odours from reduced compounds
Effluent not suitable for receiving waters (usually BOD_5 > 100 mg/L).

Anaerobic lagoons are usually designed for treatment of strong industrial and agricultural wastes, or are used in a pre-treatment step where industry discharges into the municipal scheme.

As mentioned earlier, this process is not normally used in treating domestic wastewater, and therefore, will not be discussed further.

2.3 Facultative Lagoons

Facultative lagoons (also called waste stabilisation ponds (WSP)) contain a hybrid environment (Figure 2.3). The aim of these lagoons is to provide a deeper, anaerobic zone in which high strength waste is degraded and "stabilised" while maintaining a surface, aerobic zone in which the reduced (and frequently odorous) products of the anaerobic zone can be oxidised to CO_2 and water. Consequently, the biological flora of these lagoons is the most diverse of all the various types of lagoons and combines the populations and activities described above for anaerobic and aerobic lagoons in the lower and upper zones respectively.

Facultative lagoons, in view of their complexity, are more difficult to design properly. Their characteristics are summarised in Table 2.3.

Table 2.3 Facultative Lagoons:- Strengths and Weaknesses

STRENGTHS

- Medium biomass formation
- Can handle medium organic load
- Pollutants are converted to gaseous products
- Odourless provided an aerobic zone is maintained
- Can handle shock load

WEAKNESSES

- Difficulty in maintaining aerobic zone
- Requirement of large land area
- Generation of algal biomass
- Temperature sensitive
- Night sag in dissolved oxygen
- Limited oxygen supply capacity

2.4 Aerated Lagoons

Aerated lagoons are classified as lagoons in which mechanical or diffused aeration is provided to enhance the oxidation rate of pollutants (Figure 2.4). Aerated lagoons can be further divided into two categories depending on the extent of mixing power provided.

- Completely Mixed Aerated Lagoons
- Facultative Aerated Lagoons

2.4.1 Completely mixed aerated lagoons

The purpose of the mechanical aerators in these lagoons is to achieve complete suspension of solids in the entire volume of the lagoon and to ensure adequate oxygenation of the lagoon. Properly designed completely mixed aerated lagoons should have no sludge. Substantial power input and a suitable aerator (typically with a draft-tube attachment) are required to achieve these aims. Eckenfelder (1980) quotes power to volume ratios of 2.3-3.9 W/m3 lagoon volume for these lagoons.

The environment resulting is aerobic with a considerable suspended solids concentration. In essence, it is an activated sludge unit without sludge recycle.

2.4.2 Facultative aerated lagoons

Less agitation (typically 0.8 - 1.2 W/m3 lagoon volume) is provided by the aerators in these lagoons, so as to generate a facultative lagoon environment. The purpose of the aeration is to maintain a positive dissolved oxygen tension in the surface zone of the lagoon without disturbing the deeper anaerobic zone. The lagoon will achieve higher rates of activity than non-mechanically aerated lagoons.

In both types of aerated lagoons, the objective is to overcome the problem of maintaining a positive dissolved oxygen concentration in naturally aerated aerobic lagoons. The provision of aeration permits greater rates of aerobic microbial activity to be achieved, which allows higher organic loadings to these lagoons. Oxygen supply and hence loading, is a function of aeration power.

Some considerations for these lagoons are detailed in Table 2.4.

Table 2.4 Aerated Lagoons :- Strengths and Weaknesses

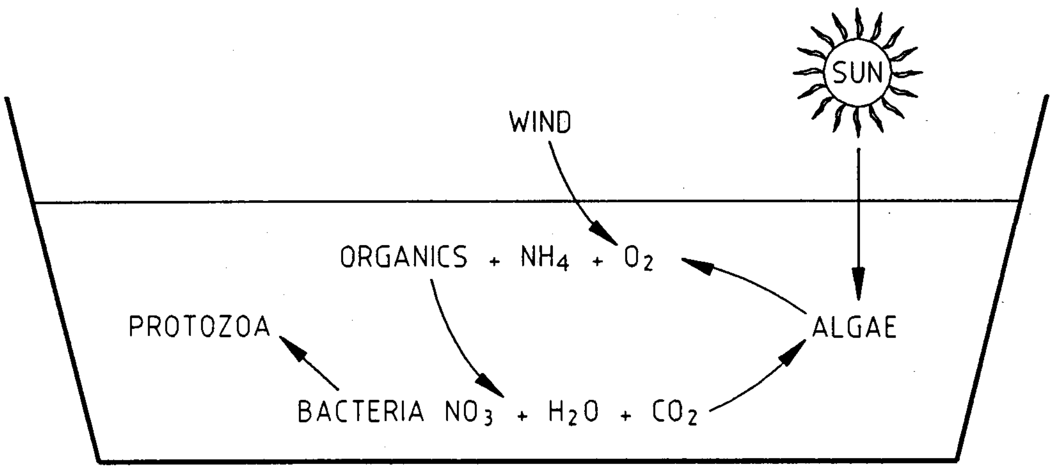
STRENGTHS

- Can handle higher strength waste
- Better mixing characteristics than unstirred lagoons
- More resistant to influent changes/shocks
- Generate high quality effluent

WEAKNESSES

- High suspended solids concentration in the effluent
- Rapid heat transfer leads to cooling during winter
- Operating and capital costs of aerators
- Odour and volatility of organic compounds

Figure 2.1 Aerobic Lagoon Characteristics



CHARACTERISTICS:
- Aerobic Environment
- Shallow
- Diverse Microbial Population
- Oxidised Products
  - Cells
  - $CO_2 + H_2O$
- Limited to Low Organic Loadings

OVERALL PROCESS:
Aerobic

Organic material + O_2 $\xrightarrow[\text{micro-organisms}]{\text{Aerobic}}$ Aerobic micro-organisms + CO_2 + H_2O

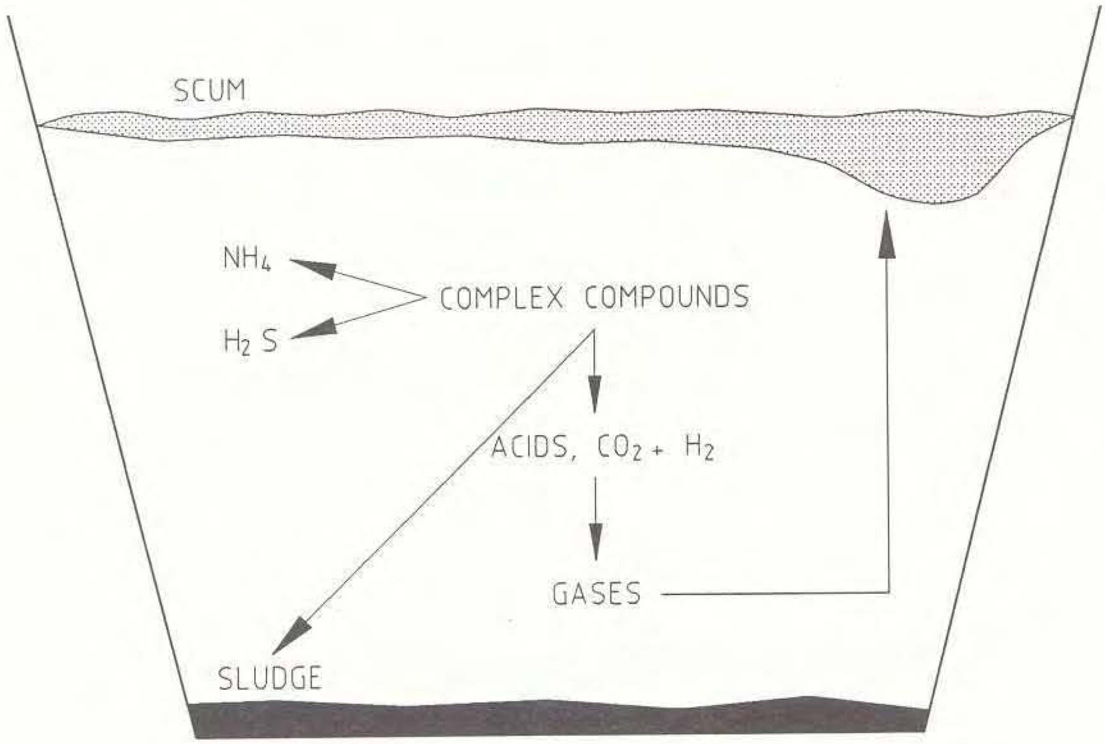
Photosynthesis

$CO_2 + H_2O$ $\xrightarrow[\text{micro-organisms}]{\text{Photosynthetic}}$ Photosynthetic micro-organisms + O_2

OVERALL PROCESS:
Anaerobic

Organic material $\xrightarrow[\text{micro-organisms}]{\text{Anaerobic}}$ Anaerobic micro-organisms + CO_2 + CH_4 + H_2O

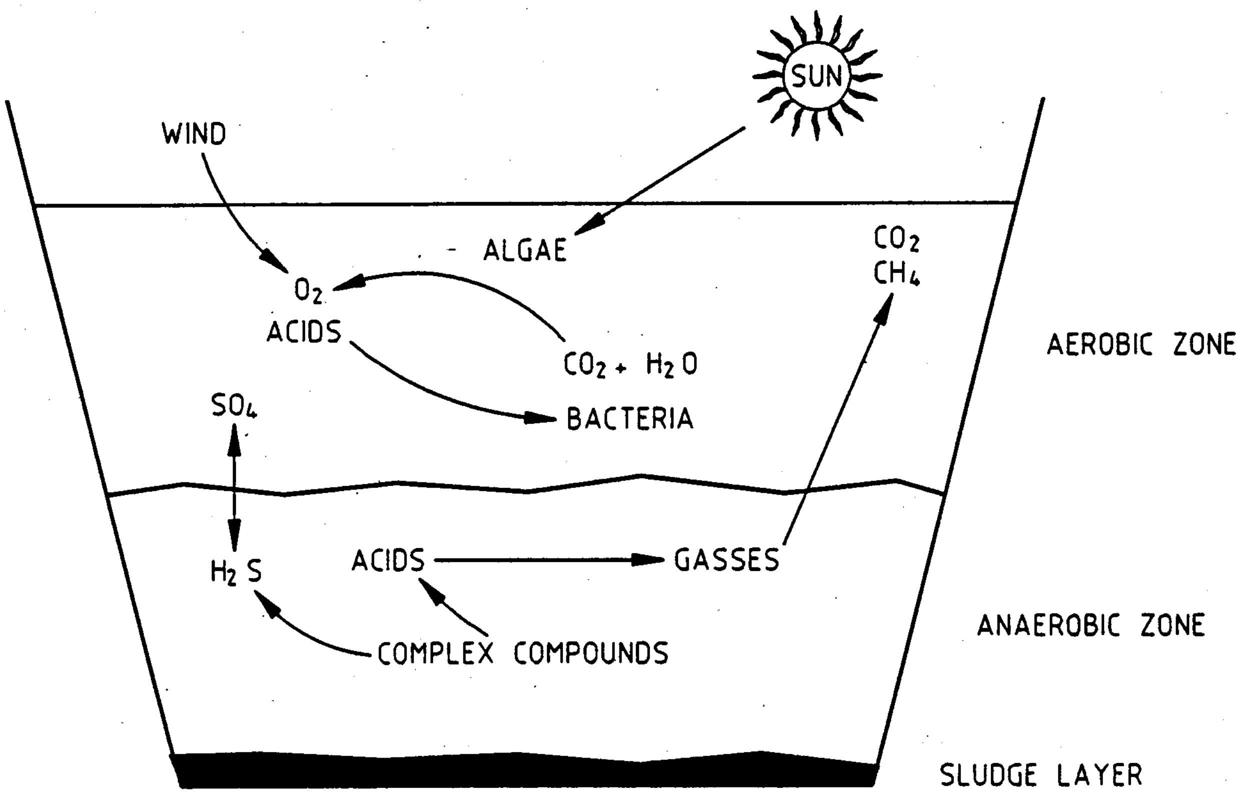
Figure 2.2 Anaerobic Lagoon Characteristics



CHARACTERISTICS:

- Anaerobic Environment
- Deep
- Two-Stage Microbial Process
- Major Products are Gases
  - Methane
  - Carbon Dioxide
- Effluent contains mainly reduced compounds
- Low Biomass Formation

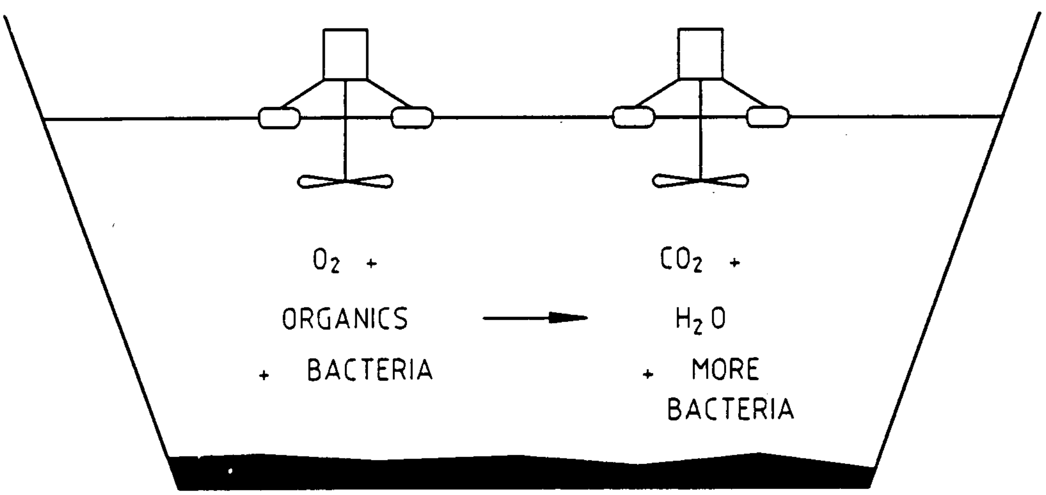
Figure 2.3 Facultative Lagoon Characteristics



CHARACTERISTICS:

- "Two-Zone" Environment
- Deeper than an Aerobic Lagoon
- Most Microbially Diverse
- Medium Load Capability

Figure 2.4 Aerated Lagoon Characteristics



TWO TYPES:

- Completely mixed
  P/V 2.3 - 3.9 W/m^3
- Facultative
  P/V 0.8 - 1.2 W/m^3

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CHAPTER 3

TASMANIAN LAGOONS

Contents

Tables

3.1 Introduction

Lagoon systems have become a popular way of treating wastewater in the small to medium size communities within Tasmania; approximately 60% of all the permitted (*licensed) treatment plants in the State are sewage lagoon systems. Because of their low construction and operating costs they offer these communities a significant financial advantage over other methods of treatment provided the cost of land is not prohibitive. The source of wastewater treated by these plants is mainly domestic. (* Under the new Resource Management and Planning System of Tasmania, what were formerly licenses are now referred to as permits.)

Lagoons in the State are also presently being used for treating industrial wastewater and mixtures of industrial and domestic wastewater amenable to biological treatment. An example of this is the George Town WWTP, which has two aerated basins followed by a facultative lagoon, two aerobic lagoons, and a wetland to treat industrial wastewater from Tempco (a metallurgical industry) and domestic wastewater from George Town. Lagoon installations are also serving such industries as abattoirs (Longford), rendering plants, vegetable processors (Smithton), dairies and milk product manufacturers, and poultry processing plants (Sorell).

3.2 Tasmanian Lagoon Systems

There are 88 permitted wastewater treatment plants in Tasmania of which 55 have lagoon systems associated with them. Most of these lagoon systems have been designed to provide the full treatment, i.e., receive untreated domestic or industrial wastewaters and provide the primary, secondary and polishing parts of the process. Some, however, are being used to provide only part of the treatment within a plant such as treating only the primary or secondary treated effluent, or excess (activated) sludge. Other lagoon systems are being used for only primary treatment to reduce non-filtrable residue (NFR) (and the biochemical oxygen demand (BOD) and the concentration of disease-causing agents) to comply with the performance NFR limit in the 1974 Environment Protection (Water Pollution)

Regulations for discharge to coastal waters. Refer to Tables 3.1 and 3.2 for a list of the lagoon systems in the State and details about each one.

Under the now rescinded Tasmanian Environment Protection Act (EPA) 1973 all treatment plants with normal dry weather flows greater than 25 kL/d were required to be licensed. With the commencement of the Environmental Management and Pollution Control Act 1994 on the 25 January 1996 only those plants with design capacity to treat average dry weather flows of 100kL/d or greater are classified as level 2 activities and require a permit to operate. Those treatment plants with design capacity to treat average dry weather flows less than 100 kL/d are classed as level 1 activities and as such be regulated by Councils. The Environmental Management and Pollution Control Board may, however, determine that it is necessary for the Board to regulate any of these level 1 plants if the environmental need arises. The Board is assisted by Environment Tasmania through the Director of Environmental Management.

The following statistics on Tasmanian lagoons can be determined from Table 3.2:

Lagoon Type: 65.2% are passive lagoon systems, 21.7% have aerated primary lagoons, 8.7% have secondary lagoons associated with mechanical/biological facilities, 2.2% have wetlands attached, and 2.2% have chlorination facilities attached;

Lagoon Cell Number: 6.5% single cell, 63% 2 cells, 15.1% 3 cells, 4.4% 4 cells, 4.4% 5 cells, 4.4% 6 cells, 2.2% 7 cells;

Baffles: 52.4% wooden, 19.0% corrugated fibro-cement sheet, 9.5% prefabricated plastic, 14.3% square aluminium sheet, 4.8% concrete block;

Primary Inlet: 71.4% central, 26.2% side, 2.4% multiple side inlets;

Connection between cells: 62% one-way flow, 38% sub-surface two-way flow;

Lagoon Depth: 17.5% 1 metre, 42.1% 1.2 m, 3.5% 1.3m, 3.5% 1.4m, 10.5% 1.5m, 7.1% 1.8m, 10.5% 2-3m, 3.5% greater than 3m;

Average equivalent persons (EP) connected to a lagoon system is 1280;

Average Dry Weather Flow entering lagoons is 365 kL/d;

Average BOD_5 is 69.5 kg/d;

Average Primary Load is 69.9 kg/ha.d.

Table 3.1 Tasmanian sewage lagoon systems and their outfall points

Council	WWTP Location	WWTP Type	Outfall Point	Receiving Waters	Licensed Flow(kL/d)
Break O' Day	Fingal	AL	South Esk River	Inland	125
	St Helens	L	Georges Bay	Bay	690
	St Marys	L	Break O' Day River	Inland	190
	Steiglitz	L	Intended for irrigation	NA	59
Brighton	Brighton	AL	Jordan River	Inland	200
Burnie	Ridgley	PDLC	Pet River	Inland	110
Central Coast	Turners Beach	L	Forth River	Estuarine	400
Central Highlands	Bothwell	L	Clyde River	Inland	155
	Hamilton	L	Clyde River	Inland	40
	Ouse	L	Ouse River	Inland	40
Circular Head	Smithton	AL	Pelican Pt Duck Bay	Bay	5200
	Stanley	L	Unnamed Creek	Inland	276
Clarence	Cambridge	L	Barilla Rivulet	Inland	125
	Rokeby	ASL	Ralphs Bay	Bay	1330
Dorset	Bridport	L	Andersons Bay	Ocean	500
Glamorgan/Spring Bay	Bicheno	L	Old Mines Lagoon	Irrigation	450
	Orford	L	Quarry Point	Coastal	473
	Swansea	L	Water Creek	Irrigation	200
	Triabunna	L	Vicarys River	Estuarine	210
George Town	George Town	APLW	Tamar River	Estuarine	3600
Huon Valley	Dover	L	Esperance Bay	Bay	175
	Ranelagh	L	Huon River	Inland	525
Kentish	Railton	L	Redwater Creek	Inland	600
	Sheffield	L	Dodder Rivulet	Inland	324
Kingborough	Margate	AL	North-West Bay	Bay	167
Latrobe	Port Sorell	L	Eddies Point	Estuarine	961
Launceston	Lilydale	L	Rocky Creek Pipers River	Inland	135
Meander Valley	Carrick	AL	Liffey River	Inland	500
	Deloraine	L	Meander River	Inland	568
	Prospectvale	L	Dalrymple Creek	Inland	1400
	Westbury	L	Quamby Brook	Inland	600
Northern Midlands	Campbell Town	L	Elizabeth River	Inland	325
	Cressy	L	Back Creek	Inland	240
	Evandale	LC	Boyes Creek	Inland	375
	Longford	AL	Back Creek	Inland	1700
	Perth	L	South Esk	Inland	450
Southern Midlands	Bagdad	AL	Bagdad Rivulet	Inland	75
	Kempton	AL	Green Ponds Rivulet	Inland	135
West Coast	Queenstown	EAL	Queen River	Inland	1100
	Strahan	AL	Manuaka River	Inland	240
	Tullah	L	Lake Rosebery	Inland	243
	Zeehan	L	Little Henty River	Inland	214
West Tamar	Beaconsfield	LW	Brandy Crk, Tamar	Inland	324
	Exeter	L	Stony Creek to Tamar	Inland	115
	Ilfraville	AL	West Arm Port Dalrymple	Estuarine	540
	Legana Industrial Park	L	Bernard Pt Tamar River	Estuarine	540

Legend:

Lagoon Type

AL	Aerated Primary Lagoon
APLW	Aerated Ponds followed by Lagoons and Wetlands
ASL	Activated Sludge followed by Lagoons
EAL	Extended Aeration Tank followed by Lagoon
L	Passive Lagoon System
LC	Passive Lagoon System followed by Chlorination
LW	Passive Lagoon System followed by a Wetlands
PDLC	Pasveer Ditch followed by Lagoons and Chlorination

Lagoon systems included in the study

Table 3.2 Design details on Tasmanian lagoon systems

Council	WWTP Location	Average Connected Equ. Persons	WWTP Type	Lagoon Cell Number	Baffle Number	Primary Inlet Type	Connection Between Cells	Primary Surface Area(m2)	Lagoon Depth (m)	Total Surface Area(m2)	Unit Flow (Vp(d)	ADWF (ktid)	Boo Load (kg/d)	Average Primary Load (kgfba.d)	Compliance With Regulation Limits 1994195: BOO	Faecal Collforms
Break O' Day	Fingal	350	AL	2	1 W	SC	SSFE	4200	1.2	6300	240	84	245	58.3	N/N	N/N
	St Helens	2000	L	2	0	SSS	OW	20600	1.3	31980	240	350	02	49.5	NN	N/V
	St Marys	450	L	2	W	SC	SSFE	7450	1,2	11150	240	08	31 5	42.3	N/N	N/N
	Steiglitz	2000	L	2	0	SSS	OW	18500	1.5	27000	240	90	263	14.2	ER	ER
	Brighton	1120	AL		0	SSC	NA	6 00	1.2	6100	205	250	63	103.3	N N	N/N
Brighton	Ridgley	-	PDLC		0	NA	OW	NA	I(S)						Y/Y	Y/N
Burnie	Turners Beach	-	L	2	W ICB	SSC	SSFE	10350	1					76.6	Y/V	YN
Central Coast	Bothwell	350	L	3	1CFCS	SSC	SSFE	4680	1.2	23700		213	49	52.4	YN	N/N
Central Highlands	Hamilton	164	L	2	0	SS	OW	1815	1.8(P)&2.7(S)	7000	240	84	245	63.4	VN	YIN
	Ouse	172	L	3	0	SS	NA	3720	1.2	5135	240	51	5	32.3	YN	Y/N
	Smithton	4500	AL	1	0	SSS	OW	37100	4.6(P)&1.2(S)	3720	240	40	2	973	Y/Y	Y/Y
	Stanley	870	L	7	0	SSC	OW	0000	1.2	145300	250	3160	3610	63.0	NN	N/N
Circular Head	Cambridge	-	L	3	0	SS	OW		1.2	17500	230	253	63		Y/Y	VN
	Rokeby	6000	ASL	3		NA	OW		1(S)			25	3 3		VN	N/N
	Bridport	620	L	2		SSC	OW		1	8094		400	350	152.6	NA	NA
Clarence	Bicheno	600	L	2	0	SC	SSFE	8 92	1.2	12288		500	25	39.6	ER	ER
	Orford	500	L	2	1W	SSC	NA	0600	1.8	15650		44	42	152.2	NA	NA
	Swansea	500	L		0	SSC	SSFE	2300	1.2	2300	240	120	35	53.8	ER	ER
Dorset	Triabunna	1000	L	2	1CFCS	SSC	OW	6500	1.2	9400	240	20	35	64.2	Y/Y	YIN
Glamorgan/Spring Bay	George Town	-	APLW	2	0	NA	OW	10900	3(P)&2.5,1.5,2(S)	5800	240	240	70		Y/Y	YN
	Dover	430	L	4	0	SSC	SSFE		1.2		240			39.2	YIN	N/N
	Ranelagh	2330	L	2	1 W	SSC	SSFE		1.2			95	20	77.7	Y/Y	N/N
	Railton	1000	L	2	W	SSC	OW	5100	1	7970		5592	398	38.2	Y/Y	Y/N
	Sheffield	1200	L	2	0	SSC	OW	8000	1.2	27000	220	275	688	82.5	YN	N/N
George Town	Margate	700	AL	2	0	SSS	OW	18000	1.2	27000	240	330	825	82.0	YIN	YIN
Huon Valley	Port Sorell	1625	L	2	0	SSC	SSFE	0000	1.2	15000	275	64	0	37.8	N/N	N/N
	Lilydale	675	L	2	1 W	SSC	SSFE&OW	5000	1.2	10000	275	20	503	71.8	VN	V/N
	Carrick	1750	AL	2	1CFCS	SSC	SSFE	3300	1.5	20000		155	388	62.9	YIN	N/N
Kentish	Deloraine	2300	L	2	1W	SSC	OW	5400	1.4	8050		350	88	48.6	V/N	V/N
	Prospectvale	4800	L	2	0	SS	OW	14000	1.5	21000	275	450	3	140.8	N/N	YIN
Kingborough	Westbury	1292	L	4	0	SSC	SSFE	23250	1.4	35400	230	200	300	47.6	YIN	N/N
Latrobe	Campbell Town	800	L	2	1W	SC	OW	2 300	1.8 (P) & 1.5 (S)	62900	200	280	70	46.7	VN	VN
Launceston	Cressy	600	L	6	0	SSC	OW	14700	1	19100	210	192	56	65.6	VN	YIN
Meander Valley	Evendale	1000	LC	2	0	SSC	SSFE&OW	12000	1.2	16400	250	44	42	102.2	V/V	V/N
	Longford'	-	AL	3	1CFCS	SS	OW	6400	2.4(P),1.8,&1.0	2900	2 0	240	70	1595	N/N	N/N
	Perth	1550	L	6	0	SSC	OW	6850	1	15230	240	1466	973	66.1	N/N	N/N
	Bagdad	235	AL	2	0	SSC	SSFE&OW	6100	1	91200	240	372	109	62.5		-
	Kempton	440	AL	2	1PP	SSC	SSFE	6500	1	33000	240	50	125	60.8		-
	Queenstown	-	EAL	2	1 W	SSS	OW	2000	5(P)&2(S)	4000	220	486	122			N/N
Northern Midlands	Strahan	665	AL	2	0	SSC	SSFE&OW	2000	1.2	3000	240			147.8	Y/Y	N/N
	Tullah	225	L	3	PP	SSC	OW	800	1.2	2685	240			25.0	YN	YN
	Zeehan	1500	L	2	0	SSC	OW	2700	1.2	8100	1 0	66	39 9	41.0	VN	Y/Y
	Beaconsfield	1010	LW	2	0	MSSS	SSFE&OW	5600	1.2(P)&1.3(S)	8400		56	14	94.7	Y/Y	N/N
	Exeter	650	L	5	4SAS	SSC	SSFE&OW	23000	1.2(P)&1.5(S)	34500	250	377	943	70.9	N/N	YIN
	Ilfraville	1640	AL	5	W& SAS	SS	OW	6400	1.2	16900	250	242	606	92.0	YIN	YN
Southern Midlands	Legana Industrial Park	1570	L	2	0	SSC	SSFE&OW	5500	1.3	1300	250	156	39	102.4	N/N	YN
				3	1SAS			10700		18700	240	394	984		YN	
								9200		21200	240	377	942			
West Coast											240					
											240					
West Tamar																
Average		1280						10162		20915	236	364	69 5	69.9		

Legend:

(A) Baffle Type
- CB — Concrete Brick
- CFCS — Corrugated Fibro- Cement Sheet
- SAS — Square Aluminium Sheet
- W — Wooden
- PP — Pre-fabricated Plastic

SSC

(B) Lagoon Type
- AL — Aerated Primary Lagoon
- APLW — Aerated Ponds followed by Lagoons and Wetlands
- ASL — Activated Sludge followed by Lagoons
- EAL — Extended Aeration Tank followed by Lagoon
- L — Passive Lagoon System
- LC — Passive Lagoon System followed by Chlorination
- LW — Passive Lagoon System followed by a Wetlands
- PDLC — Pasveer Ditch followed by Lagoons and Chlorination

(

(C) Inlet Type
- MSSS — Multiple Sub-Surface Side
- NA — Inlet into mechanical treatment facility
- SC — Surface Central
- SS — Surface Side
- SSS — Sub-Surface Side
- Sub-Surface Centel

(D) Internal Outlets
- SSFE — Sub-Surface Free Exchange
- OW — One Way

(E) Lagoon Depth
- P — Primary Cell
- S — Secondary Cell

(F) Unit Flow' — Normal theoretical flows of 200 to 250 Up/d are used

(G) ADWF — Average Dry Weather Flow

(H) Compliance N — No compliance
- Y — Yes, compliance
- ?/? — 50th/90th percentile
- ER — Effluent Reuse
- NA — Not Applicable

(NFR not regulated due to algae in effluent)

(I) Lagoon systems in the study

3.3 Monitoring

Most of the permits for treatment plants require that a 24 hour composite sample be obtained once per month and be tested for at least the following if they are discharging to inland waters, bays or estuaries: 5 day biochemical oxygen demand (BOD5), non-filtrable residue (NFR), and faecal coliforms. The need for a 24 hour composite sample at the outfall is to be reviewed by Environment Tasmania in light of information generated by the Sewage Lagoon Performance Improvement Program (SLPIP) which showed only a slight difference in effluent quality between those samples taken by composite and those by grab methods. Refer to Chapter 8, Section 8.3, for detail on monitoring aspects.

Both the outfall point and the nature of the receiving waters dictate the regulated effluent quality to be achieved by the treatment process. Table 3.3 summarises the standards of the Environment Protection (Water Pollution) Regulations 1974 for BOD5, NFR, faecal coliforms, oil and grease and the dissolved oxygen level. This table also details the phosphorus, nitrite, nitrate, and ammonia levels listed in the regulations.

Table 3.3 Sewage treatment plant discharge limits under the Environment Protection (Water Pollution) Regulations, 1974.

	BOD_5 (mg/L)	NFR (mg/L)	Faecal Coliforms (orgs/100mL)	Oil & Grease (mg/L)	Ammonia (mg/L)	Nitrate &/or Nitrite (mg/L)	Phosphorus (mg/L)
Receiving Waters							
(1) Inland Waters							
(i)	20	30	200	10	0.5	10	2
(ii)	40	60	200	10	0.5	10	2
(2) Bays & Estuarine Waters							
(i)	20	30	1000	10	0.5	10	2
(ii)	40	60	1000	10	0.5	10	2
(3) Coastal Waters	N/A	200	N/A	N/A	N/A	N/A	N/A

Notes:
(a) For "Inland Waters" and "Bays and Estuarine Waters", (i) represents where the flow of the receiving waters is less than 50 times the rate of the flow of the emission, and (ii) is where the flow of the receiving waters is greater than 50 times the rate of the flow of the emission.
(b) Oxygen content in the receiving waters shall not be reduced to below 50% saturation.
(c) The effluent should be visually free of oil and grease.
(d) Where algae are visually detectable (i.e. the chlorophyll "a" and phaeophytin is greater than 100 ∝g/L) in the effluent there is no limit on the NFR level and the BOD level is increased to 40 mg/L for inland waters, bays, and estuaries.

The regulation, however, states that these nutrient levels are not applicable to sewage treatment plants which include sewage treatment lagoon systems. With the commencement of the new State Policy for Water Quality Management (see Chapter 5), anticipated by late 1996, the quantification of nutrient levels in the effluent and the receiving waters, as part of a catchment management approach, will be required.

If algae are detectable in the lagoons, i.e., the chlorophyll "a" and phaeophytin is greater than 100 ∝g/L then the NFR performance parameter is not applicable. In this situation the BOD is, if at a regulation level of 20 mg/L, lifted to 40 mg/L. If discharging to coastal waters then only compliance with the NFR performance parameter is required. The effluent should also be visually free of oil, grease, solids and unnatural discolouration.

The Environmental Management and Pollution Control Act 1994 (EMPCA) uses the 1974 Water Pollution Regulations as an interim arrangement pending the completion of the Tasmanian Sustainable Development Policy (State Policy) on Water Quality Management and the establishment of site specific ambient water quality objectives. Plants will then be required to perform in compliance with the specific ambient requirements identified for the particular receiving waters, which will reflect the protected environmental values of those waters. Any upgrades to sewage treatment plants will require the use of acceptable modern technology, information on the dispersion and dilution rates of the receiving waters, and include programs of wastewater minimisation.

The monitoring data provided by Councils as a requirement of their permit conditions is held on a spreadsheet database held by Environment Tasmania.

3.4 Compliance

The permits for plants include conditions that require the premises be maintained and operated in such a manner that the final effluent complies with the relevant standards as set down in the 1974 Water Pollution Regulations. These conditions require that every effort is to be taken to operate the premises such that the discharge is less than the respective regulation limits for the performance parameters (e.g. BOD, NFR, faecal coliforms).

In practice, when Environment Tasmania assesses the performance of the plants to determine compliance with the Regulation levels, the mean, maximum and 50th and 90th percentile data are used. An extract of the monitoring data and compliance assessment for 1994 for the Turners Beach Plant (as an example) is detailed in Table 3.4.

Premises are deemed to be in practical compliance with BOD and NFR regulatory limits provided the absolute permit limit is exceeded no more than 50% of the time and 1.5 times the absolute permit limit is exceeded no more than 10% of the time. This takes account of the inherent variability of lagoon water quality (lagoons essentially being functional ecosystems) and the consequential practical constraints of maintaining water quality below an absolute figure 100% of the time.

Practical considerations also lead to the operational faecal coliform limit being taken as 5 times the absolute regulation limit, based on the fact that initial dilution in receiving waters would be at least 5 fold which would take the coliform count to below 150 faecal coliforms per 100 mL, a level suitable for primary contact (i.e., full body immersion and/or partial intake).

Due to the algal blooms in lagoons, the NFR performance parameter is generally not considered when determining compliance. The presence of algal cells confound the measurement of suspended material. Some jurisdictions (e.g. NSW) at times use a deeming formula to recalculate NFR taking the level of algae (measured as chlorophyll "a") into account. However, this practice is not unequivocally accepted, and is not used in Tasmania.

Up to December 1995, 46 permitted lagoon systems in the State for which Environment Tasmania has regulatory responsibility were assessed for compliance for the year in accordance with the assessment approach detailed above: 17 plants (44%) and 30 plants (77%) did not comply with the BOD and the faecal coliform performance levels respectively, and 44% complied with neither parameter. Refer to table 3.2 for more compliance details.

Although percentile data provide a practical measure of compliance, the 1974 Regulations mean that the design of wastewater treatment plants should be based on achieving compliance with the absolute standards (i.e. not percentiles) set down in those regulations .

With the soon to be finalised State Policy on Water Quality Management, which will be replacing the 1974 Water Pollution Regulations, effluent discharge limits will be able to be set based on the ambient water quality objectives of the receiving waters. The setting of these objectives will involve community consultation to determine the desired environmental values of the receiving waters to be protected. Plants would be required to operate in a manner which would ensure that the objectives are met, and compliance will be assessed accordingly.

3.5 Environmental Improvement Programs

A 1992 review of lagoon performance led to the then Division of Environmental Management (now Environment Tasmania) assisting Councils to prepare what are now known as Environmental Improvement Programs (EIPs) for their non-complying lagoons. These programs were designed to assist councils to understand the systems under their authority, and enable them to optimise the management of the waste streams entering and leaving their site(s) and improve plant performance.

The submitted EIPs detailed information on plant management (operator training, expertise available, management regimes, communication framework between those parties involved in the plant management), lagoon characteristics, monitoring programs, operation and maintenance, effluent and sludge disposal, waste minimisation, major lagoon upgrade (not mandatory), and the time frame for the implementation of the program.

This manual will further assist councils to implement those EIPs.

Table 3.4 Example extract of monitoring data provided as required by permit

Client Name:	Central Coast	
Premises:	Turners Beach	
Map:	Forth (1:100000)	
Grid Ref.:	4365E5442 7N	
Treatment Type:	L	Divided Lagoon (oxidative) Polishing Lagoon
Treatment Level:	S	
Lic. Flow(kL/d):	400	
Outfall:	Forth River	
Receiving Waters:	Estuarine	

Lic.Levels:	BOD(mg/L)	NFR(mg/L)	c.f.u(/100ml)
	40	60	1000

Monitoring Data:

Sampling Date	BOD (mg/L)	NFR (mg/L)	c.f.u (/100ml)	Town Flow (kL/wk)
1994				
19 Jan	37	38	100	2909
9 Feb	34	65	400	2978
9 Mar	30	130	100	2762
6 Apr	29	110	100	2708
4 May	31	82	200	2715
1 June	23	60	200	3140
20 July	28	64	100	2282
17 Aug***	30	65	1000	2495
14 Sept	26	74	600	2294
12 Oct	15	10		
9 Nov	32	30		
14 Dec	25	22		
Sample No	12	12	9	9
Mean	28	63	311	2698
Maximum	37	130	1000	3140
Minimum	15	10	100	2282
Compliance of				
Mean	YES	NO	YES	
Maximum	YES	NO	YES	

where the mean =< licence limit (for BOD and NFR)
=< 5xlicence limit (for faecal coliforms)

and the maximum =< 2xlicence limit (BOD and NFR)
=< 10xlicence limit (faecal coliform)

1994	BOD(mg/L) More than 40	More than 60	NFR(mg/L) More than 60	More than 90	faecal coliforms (orgs/100ml) More than 5000	More than 7500
Jan	0	0	0	0	0	0
Feb	0	0	1	0	0	0
Mar	0	0	1	1	0	0
Apr	0	0	1	1	0	0
May	0	0	1	0	0	0
June	0	0	0	0	0	0
July	0	0	1	0	0	0
Aug	0	0	1	0	0	0
Sept	0	0	1	0	0	0
Oct	0	0	0	0	0	0
Nov	0	0	0	0	0	0
Dec	0	0	0	0	0	0
No. > =	0	0	7	2	0	0
%> Limit	0	0	58	17	0	0

Compliance within Percentile	BOD	NFR	cfu
50	YES	NO	YES
90	YES	NO	YES

where 50th percentile
= 50% of samples <=licence limit (for BOD and NFR)
<= 5xlicence limit (for faecal coliforms)

and 90th percentile
= 90% of samples < = 1.5x licence limit (for BOD and NFR)
<= (5x1.5)xlicenc e limit (for faecal coliforms)

3.6 References and Bibliography

1. TASMANIAN DEPARTMENT OF ENVIRONMENT AND LAND MANAGEMENT, *"Environment Protection (Water Pollution) Regulations"*, Gov. Printer, 1974.

2. TASMANIAN DEPARTMENT OF ENVIRONMENT AND LAND MANAGEMENT, "*Draft State Policy, Setting new standards for Water Quality"*, Gov. Printer, 1996.

3. TASMANIAN DEPARTMENT OF ENVIRONMENT AND LAND MANAGEMENT, *"Environmental Management and Pollution Control Act"*, Gov. Printer*,* 1994

4. TASMANIAN AUDIT OFFICE, "*Special report No 8 Wastewater Management in Local Government"*, Gov. Printer, October 1994

CHAPTER 4

FACTORS AFFECTING LAGOON TREATMENT

Contents

Tables

Figures

4.1 Introduction

This study investigated lagoons that were typical of the majority of lagoon systems within the State. The lagoons chosen were aerobic/facultative systems with depths ranging from 0.95m to 1.8m. The sewage treated by each was predominantly from domestic sources with limited commercial connections. The lagoons studied had varying cell design and performance, which allowed for many factors affecting lagoon treatment to be reviewed.

Monthly measurements and samples were taken from each lagoon system. Flow-proportional composite sampling and flow measurement of the influent wastewater were undertaken to assess the concentration and load of pollutants. Grab samples were taken at the outlet of each lagoon to determine their individual performance within the system. The final discharge from each lagoon system was sampled using 24 hour composite samples throughout the study, however it was found that grab samples taken during the morning gave similar results to composite samples.

The following parameters were used to assess the performance of each of the lagoon systems:

ammonia
biochemical oxygen demand
chemical oxygen demand
chlorophyll "a"
dissolved oxygen
faecal coliforms
hydraulic load
nitrate
nitrite
non-filtrable residue
ortho-phosphate
pH
soluble biochemical oxygen demand
temperature
total nitrogen
total phosphorus

Detailed methodologies for the parameters used are explained in Chapter 8, Sections 8.3.2. and 8.3.3.

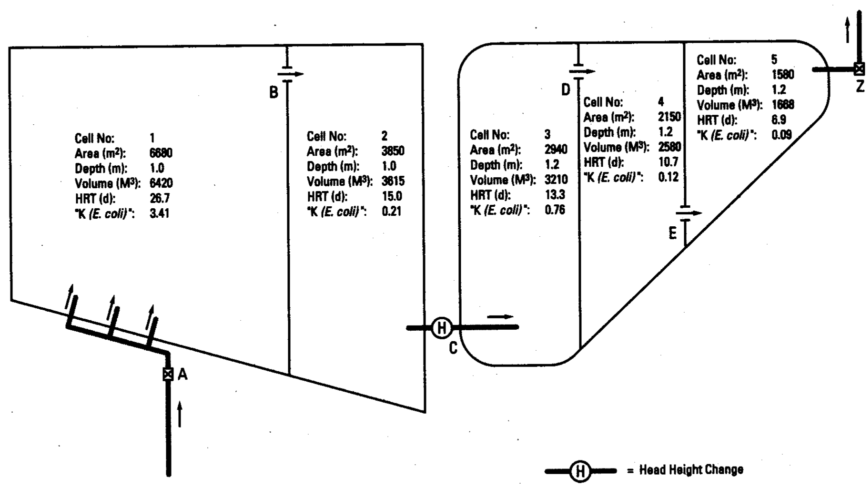
A submersible data logger was used at all lagoon systems on a monthly basis to monitor the level of dissolved oxygen, pH, temperature, conductivity and turbidity at various depths over transects in the lagoon systems. A grid of profiles for each lagoon generated information on their status over several seasons. The levels measured for these parameters and the detected occurrence of stratification in the lagoons appeared to explain some of the characteristics of lagoon performance.

The lagoons monitored in the study were distributed throughout Tasmania. As the State is small, the temperature ranges for each lagoon were fairly similar with the water temperatures ranging from approximately 5^{o}C to 25^{o}C for each lagoon system over a calendar year. Despite Tasmania being located in the Roaring Forties wind latitude, some of the studied lagoon systems were quite protected from wind action due to local topography, tree placement and high embankments around the lagoons. This allowed for the general effect of wind action on the lagoon system to be observed.

4.2 Outline of Lagoon Systems Studied

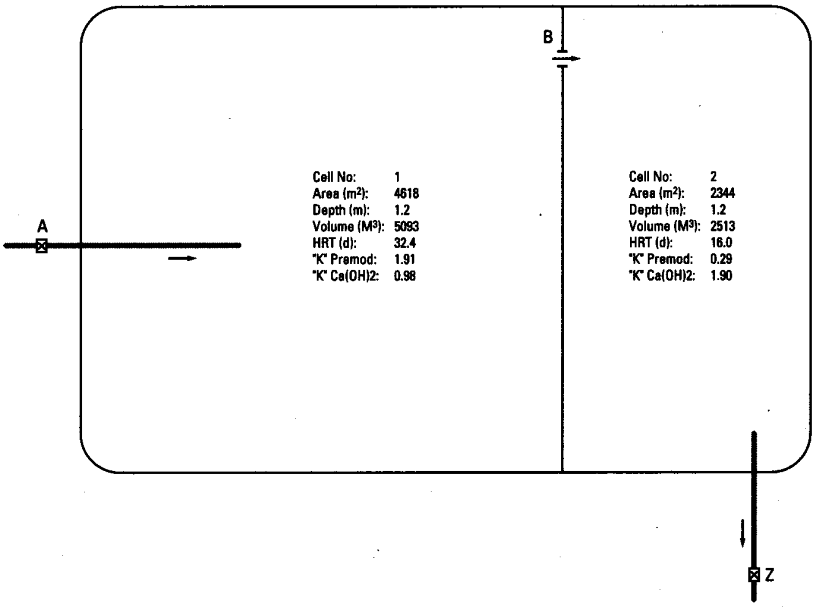
The following gives a schematic layout for each of the lagoon systems studied. Below each layout is site specific information to help visualise the system under consideration.

Beaconsfield



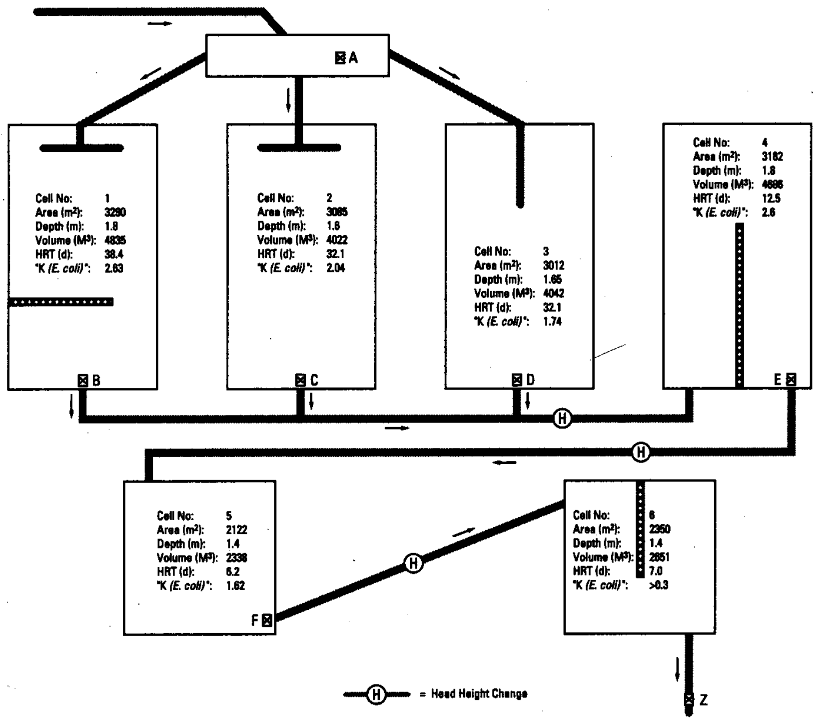
The Beaconsfield lagoon system is relatively sheltered by topography and forested areas, and thus is minimally affected by wind action from many directions. It is situated approximately 5 km from the coast. The lagoons have concrete wave walls and complete baffles made of solid aluminium sheets separate the 1st and 2nd, and the 3rd, 4th and 5th cells. The flow within the system is through subsurface outlets with weirs separating the 2nd & 3rd cells and at the outfall of the lagoon system. The 3rd, 4th, and 5th cells are quite small. The sewage enters the lagoon by gravity and is distributed by a multiple inlet.

Bothwell



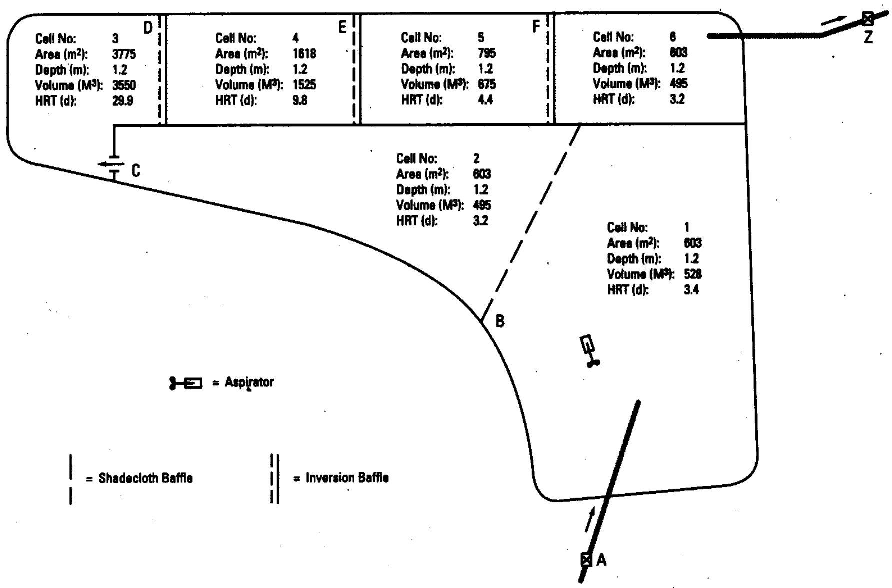
The Bothwell lagoon system is located in the Central Highlands of Tasmania. The site is well exposed to the elements and experiences some sub zero air temperatures overnight during winter. The lagoon has concrete wave walls and a complete baffle made of solid concrete sheets. The influent is pumped and enters the primary cell centrally via a bell mouth. The flow within the system is through a subsurface outlet between cells and the final discharge is via a weir. In the latter part of the study a lime ($Ca(OH)_2$) addition program was trialed at this site to determine the beneficial effects it may have on removal rate within the system.

Campbell Town



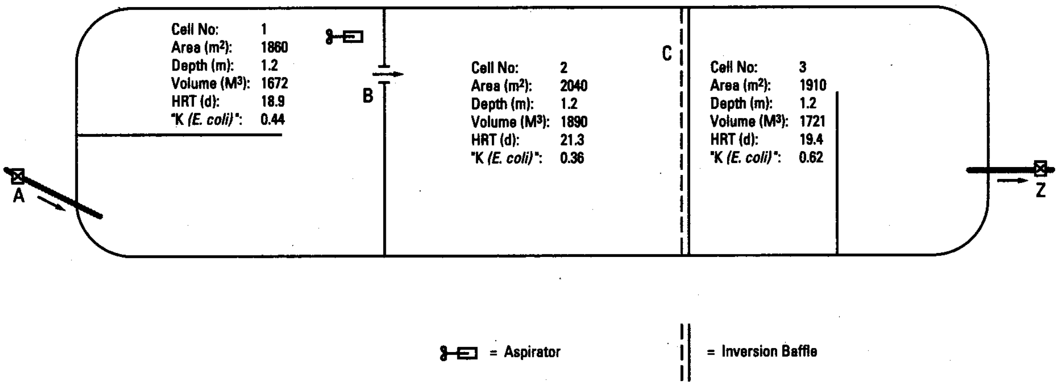
The Campbell Town lagoon system is a unique system with all three primary cells having virtually the same dimensions and aspect. This situation enabled performance comparison of primary cells which were retrofitted differently, i.e., multiple to single inlet comparison and partial baffled to non-baffled cells. The system is well exposed to the wind and is located in the Northern Midlands of Tasmania. The average depth of the lagoons is 1.8 m deep which is considerable deeper than other systems studied, which ranged from 1 to 1.2 m. The influent is pumped and enters centrally or via multiple inlets. All discharges are from the surface. The lagoon has earthen wave walls which are eroding. A pilot sand filter was trialed at this site to determine the potential removal capacity of such units used in this way.

Dover



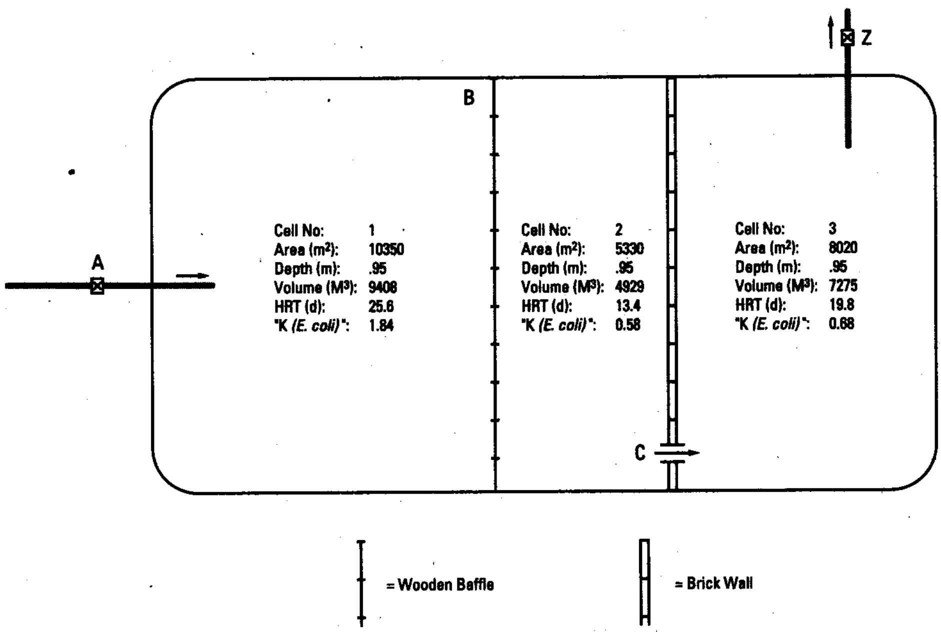
The Dover lagoon system is located in Southern Tasmania. The system is moderately protected from the effects of wind by topography and forests. The lagoon has concrete wave walls and a wooden baffle which required retrofitting (prior to the commencement of the study) with an impermeable liner to prevent water movement throughout the baffle wall. An intermittently operated, aspirator aerator was installed in the primary cell. Also, as part of the upgrade of this system prefabricated synthetic baffles with chain to weight them down were used to create the additional cells, 3 to 6. The influent is pumped and gravity fed into a central inlet. The flow within the system is surface and sub-surface, and the final discharge is via a weir. Short circuiting from the primary to the 5th and 6th cells was detected late in the study using an air compressor, but could not be effectively prevented. Ground water via a spring was infiltrating into the primary cell during wet periods.

Fingal



The Fingal lagoon system is situated on the east coast of Tasmania. There is significant interference to wind action due to high banks (designed for prevention of the encroachment of flood waters), tree locations and topography. The lagoon has concrete wave walls with prefabricated synthetic baffles to create lagoon cells 1 and 2. An impermeable liner was used to cover the existing wooden baffle which separates cells 2 and 3. A prefabricated partial baffle was created within cell 3. The influent is gravity fed and enters the primary cell diagonally opposite its outlet. The flow is subsurface throughout the system with a weir at the outfall. The lagoon is very narrow compared to other systems in the State. An aspirator (aerator) was installed in the primary cell due to the high aerial load created by reducing the original cell by approximately half.

Turners Beach



The Turners Beach lagoon system is situated on the north coast of Tasmania. The site is well exposed to wind action. The lagoon has concrete wave walls with a wooden baffle between cells 1 and 2 which enables movement of wastewater through its full length. The brick wall separating cell 2 and 3 is solid (full integrity) with only one submerged outlet diagonally opposite the outfall. The influent enters the primary cell centrally.

4.3 Flows And Loading

4.3.1 Study findings

In any process design the designer needs to assess the influent loads as accurately as possible. The result of the design calculation will only be as accurate as the inputs. Lagoon design is no exception.

In this study, flow measurements and flow-proportional composite samples were taken over 24 hour periods to assess the hydraulic and pollutant loads at each site. A brief summary of the monitoring results is set out in Table 4.1. The procedures used are outlined in Chapters 6 and 8.

Table 4.1 Influent flow measurement at six Tasmanian lagoon systems

LOCATION	Actual EP *	Average Dry Weather Flow per EP per day (L/EP.d)	Dry Weather BOD_5 Concentration (mg/L)	E.coli organisms/100ml
Beaconsfield	1010	194	264	$1.0x10^{7}$
Bothwell	350	420	194	$3.6x10^{6}$
Campbell Town	800	471	175	$8.1x10^{6}$
Dover	430	300	129	$2.9x10^{6}$
Fingal	350	258	298	$4.6x10^{6}$
Turners Beach	1500	235	215	$8.2x10^{6}$

* Source: Environment Improvement Programs 1993 (DELM)

The figures in Table 4.1 show a large variation in dry weather flow/EP.day and BOD concentration between the sites. This finding highlights the need to accurately measure the influent loading to the lagoon system. If the design values of loading are grossly different from the actual values the design will be less effective. For example, a lower design BOD load than the actual load will result in under-sizing the first lagoon for optimal BOD removal. This may result in the generation of odours. A lower design flow than the actual will also cause a reduction in the hydraulic retention time (HRT) and an associated higher bacterial count at the outfall than predicted. On the other hand, if a facultative lagoon is oversized, the stabilisation effectiveness of this unit will be reduced (Bliss 1976).

Thus, it is highly recommended that for future upgrading work on an existing site an accurate assessment of the flows and associated pollutant loads be carried out prior to design commencing. When it is not possible to measure influent load data the designer will need to make the lagoon system conservative and/or flexible to accommodate the actual loadings at a later date if they are found to be different to the original estimates.

4.3.2 Flow Measurement Issues

With flow measuring devices, it is desirable for the influent and effluent flows to be easily and accurately measured to keep an effective check on the hydraulic and organic loads, infiltration into the sewer, lagoon seepage and evaporation. The devices adopted can be low cost and require minimal maintenance, but this will depend on the inlet arrangement of the lagoon system. For example, a pumped system, such as Campbell Town, can have hour-run meters at the pumping station which are regularly recorded. The flows are calculated by multiplying the hours-run by the calibrated flows of each pump. In other situations, such as the Beaconsfield lagoon system, which are fed by gravity a portable electronic flow meter can be used to measure flow. The system adopted for this study, which is described in Chapter 8, Section 8.3.2.1, is an adaptation of a mag-flow meter into a portable unit, easily inserted within existing pipe work. It is recommended as a reasonably cheap, accurate and easy to use device.

Once the flow data is obtained an audit of the sewerage system can be carried out. This is particularly beneficial if the flows or pollutant levels are unacceptably higher than designed for. Reduction of loads can lead to significant cost savings. If there is a high dry weather flow per person connected to the sewerage system it may be worthwhile taking measures to reduce infiltration, or introducing minimisation strategies for water consumption. These may include the monitoring of commercial connections, introduction of "user pay" charges for water and/or educational campaigns to reduce water consumption. These strategies can be highly effective in deferring the need for new capital investment.

Also, with the introduction of a flow measuring device, increases in flows during periods of wet weather due to infiltration and inflow can be quantified. The percentage increase in the flow due to infiltration can be used by Council to determine the associated costs for the treatment of this additional hydraulic load or its reduction.

If a problem with high water usage, infiltration, or inflow has been recognised the portable electronic flow meter can be used on the up-stream branches of the sewer system. This allows for areas and sources responsible for high flows to be determined.

4.4 Temperature And Sunlight

4.4.1 Heat flows in lagoons

Solar radiation is the major source of heat for lagoon systems. Biological activity within lagoons increases with temperature to an optimal level. Algae and aerobic bacteria can function within a temperature range of 5°C to 40°C (US EPA 1983) which encompasses the temperature range found for Tasmanian lagoons in this study.

Another source of heat in the primary lagoon is the influent. In sewerage systems having no major infiltration problems, the influent temperature is higher than that of the lagoon contents during the cooler months. During the study it was found that the temperature of influent could range from 9 to 20 ^{o}C. With lagoon temperatures lower than the influent temperature it is feasible that a slick of warmer raw sewage may develop on the surface of the primary cell potentially leading to some short circuiting of flow. Mr. Jonathan Crockett (Gutteridge, Haskins, Davey Consultant Engineers) and Dr Mike Johns (Chemical Engineering, Qld Uni.) have observed slicking in some lagoons on the mainland of Australia (pers.comm.). However, this event was not detected during the profiling work in the study which may suggest that the slicking was mixed with the main water body of the lagoon through wind action and/or through surface cooling or rapid heat dispersion. Cooling influences within lagoon systems are exerted by evaporation, transfer of heat to the atmosphere, convection and conduction, and contact with cooler soil and ground.

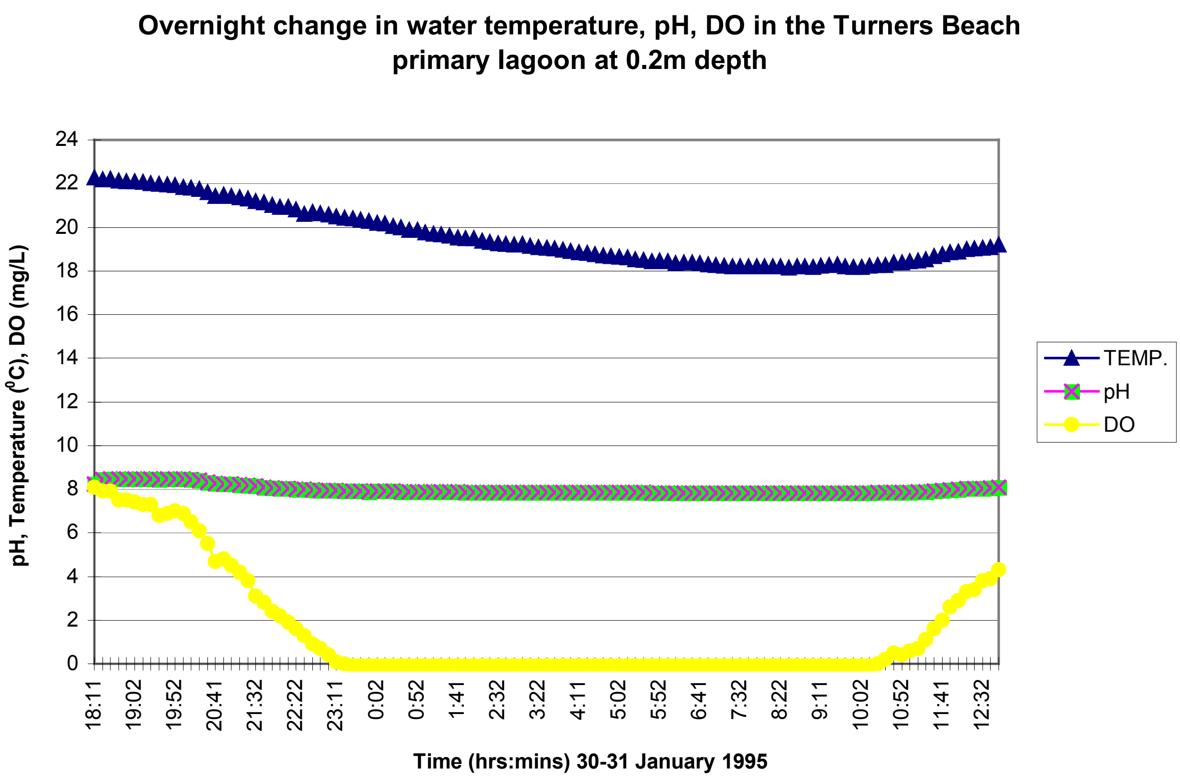
4.4.2 Temperature

In order to monitor the diurnal fluctuations in temperature and other parameters a submersible data logger (SDL) was suspended 20cm below the lagoon surface in the middle of each studied lagoon. Typical summer and winter plots of temperature, pH, and DO versus time for primary and secondary lagoon cells are illustrated in Figures 4.1 and 4.2.

The summer diurnal temperature fluctuation is significant, being 4^{o}C at the Turners Beach lagoon system, which illustrates a large amount of heat transfer occurring at the lagoon surface; at the Beaconsfield lagoon system the diurnal summer range was up to 5^{o}C. There is less temperature fluctuation over the winter months, approximately 1.5^{o}C for Turners Beach; at Beaconsfield lagoon system the diurnal winter range was 3^{o}C, illustrated in Figure 4.3.

Figure 4.1 A typical temperature, pH, and D.O. versus time chart during summer for both primary and secondary lagoon cells

(a) Primary Cell:



(b) Secondary Cell:

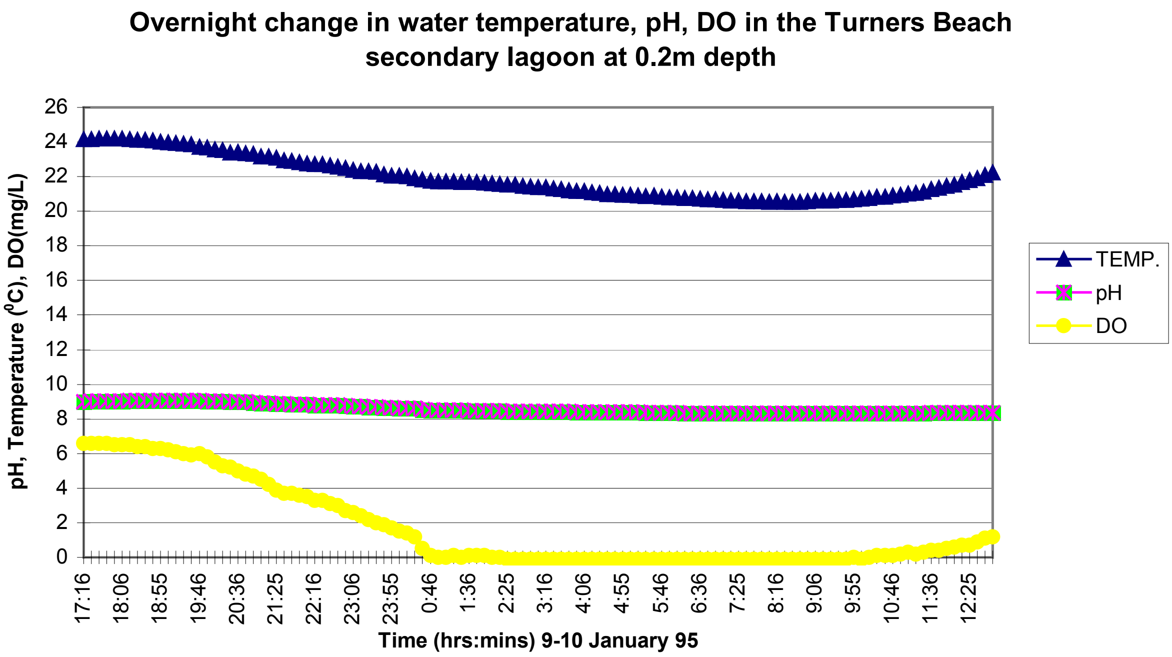
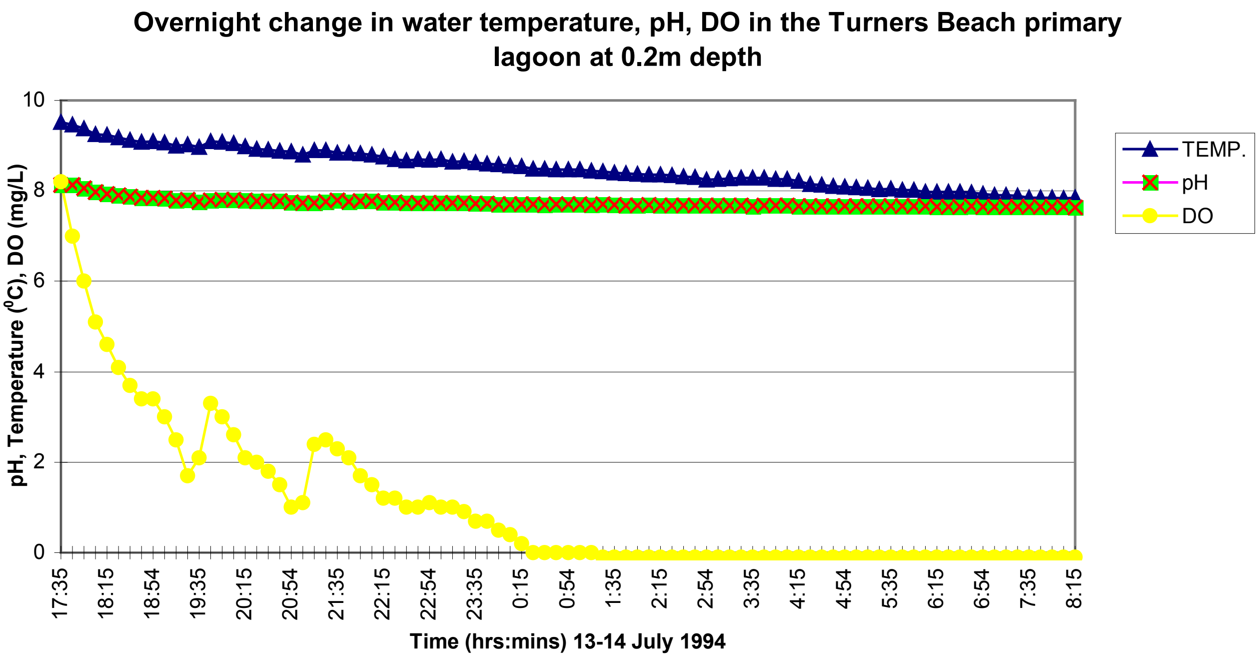
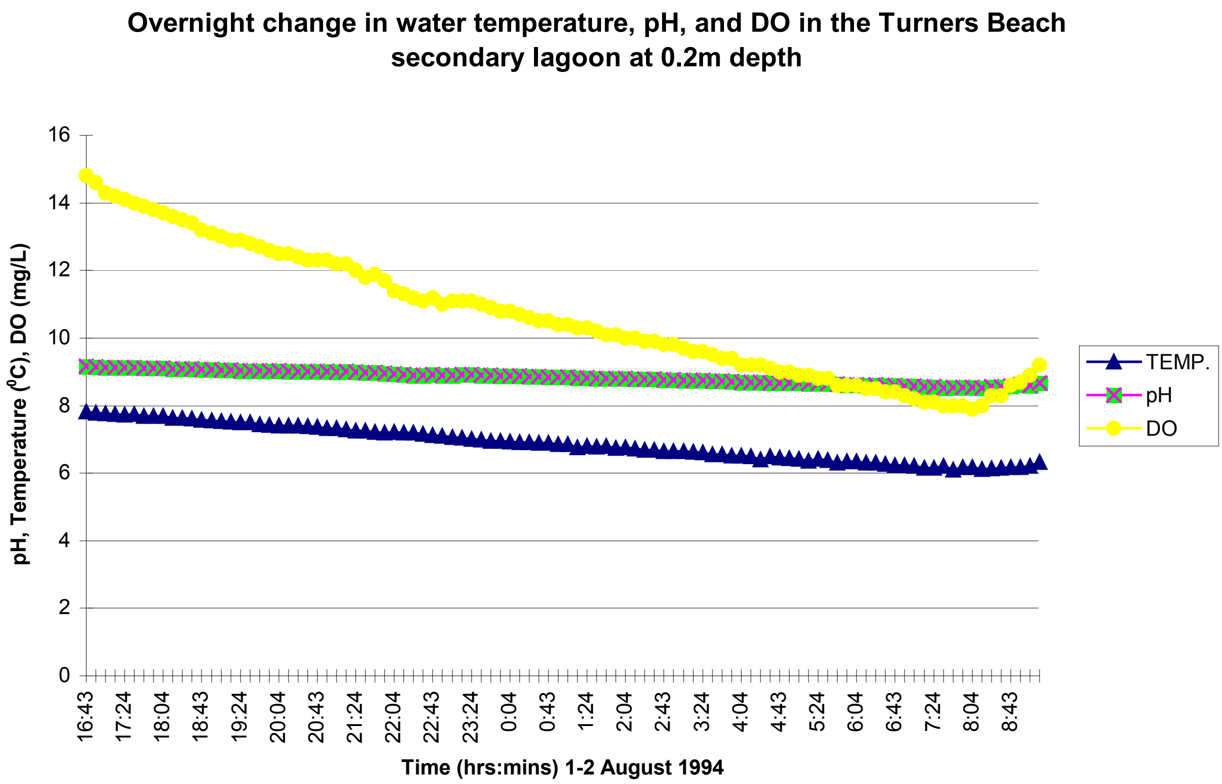


Figure 4.2 A typical temperature, pH, and D.O. versus time chart during winter for both primary and secondary lagoon cells

(a) Primary Cell:

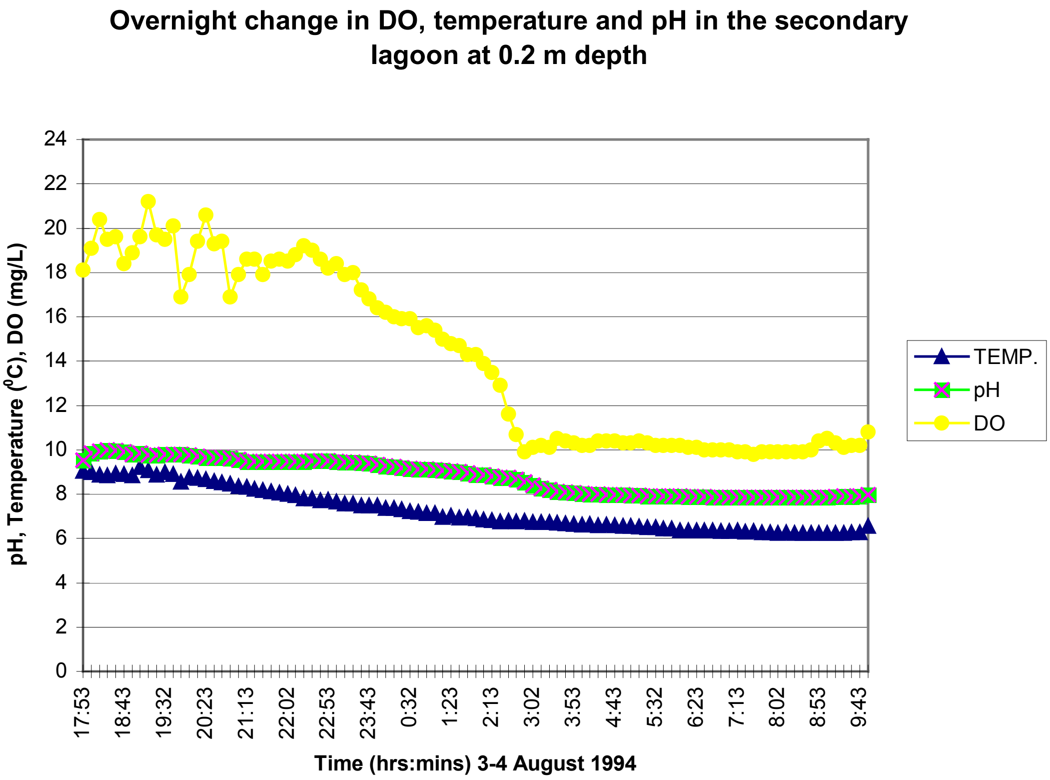


(b) Secondary Cell:



Also illustrated in Figure 4.3 is the diurnal DO change in the Beaconsfield secondary lagoon. The change that occurred up to approximately 2:30 AM clearly shows the break-up of stratification in this lagoon cell. Figure 4.7 shows that this secondary lagoon on the 3 August 1994 was strongly stratified before this destratification event.

Figure 4.3 The DO, temperature and pH versus time chart during winter for the Beaconsfield secondary lagoon cell



Figures 4.4 and 4.5 illustrate how closely the changes in lagoon temperature follow maximum air temperatures over 12 months. These lagoons had total depths of 1 and 1.2 m respectively. Similar results were found for the other lagoons. Even though the air temperature was only measured over two consecutive days using a min/max thermometer (while conducting the monthly tests at each lagoon site) the similar trends are quite marked. Significantly, in lagoons which are generally fully mixed a high proportion of the time, the lagoon temperature during the day generally corresponds to the maximum air temperature as would be expected from first principles. With lagoons which stratify, however, only the top water layer generally corresponds with the maximum air temperature and the bottom water layer temperature is several degrees lower, again as would be expected from first principles. In both situations (i.e., fully mixed or stratified) the correspondence between minimum air temperature and lagoon temperature, whether measured at the top or the bottom of the water column, is poor, suggesting that the use of minimum air temperatures for design is both inappropriate and over conservative. This fact has a significant effect on design if rate constants are used.

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Figure 4.4 Air and lagoon temperature variations for a fully mixed lagoon

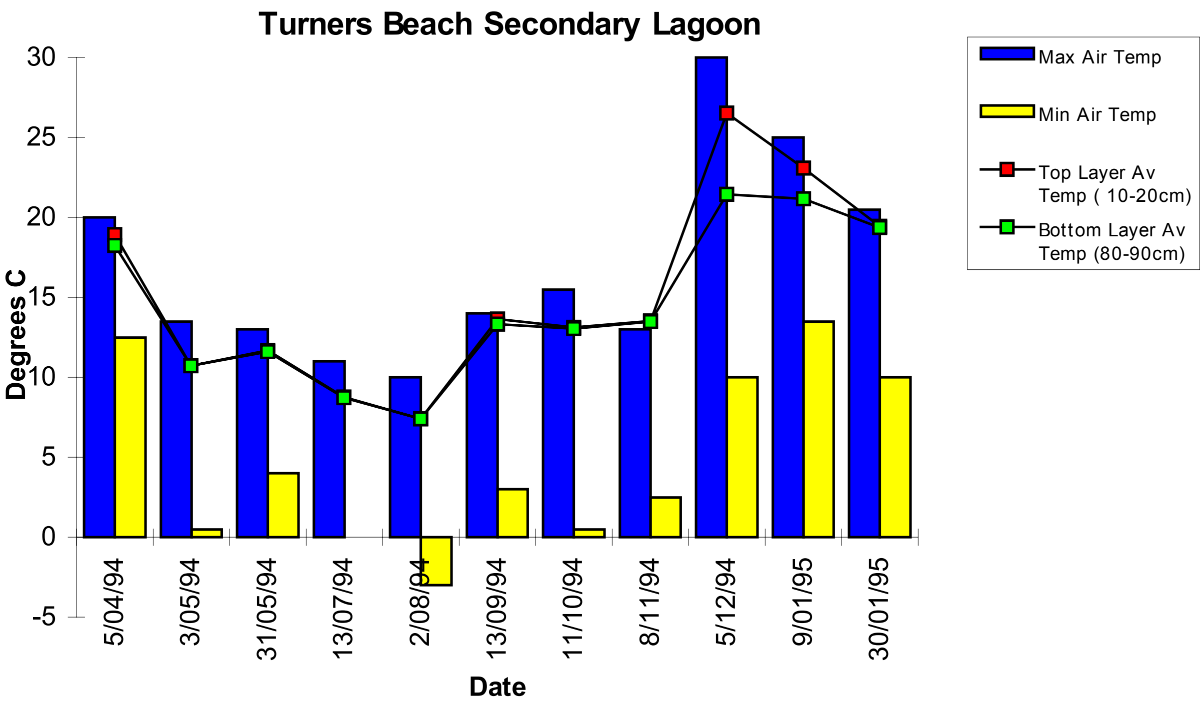
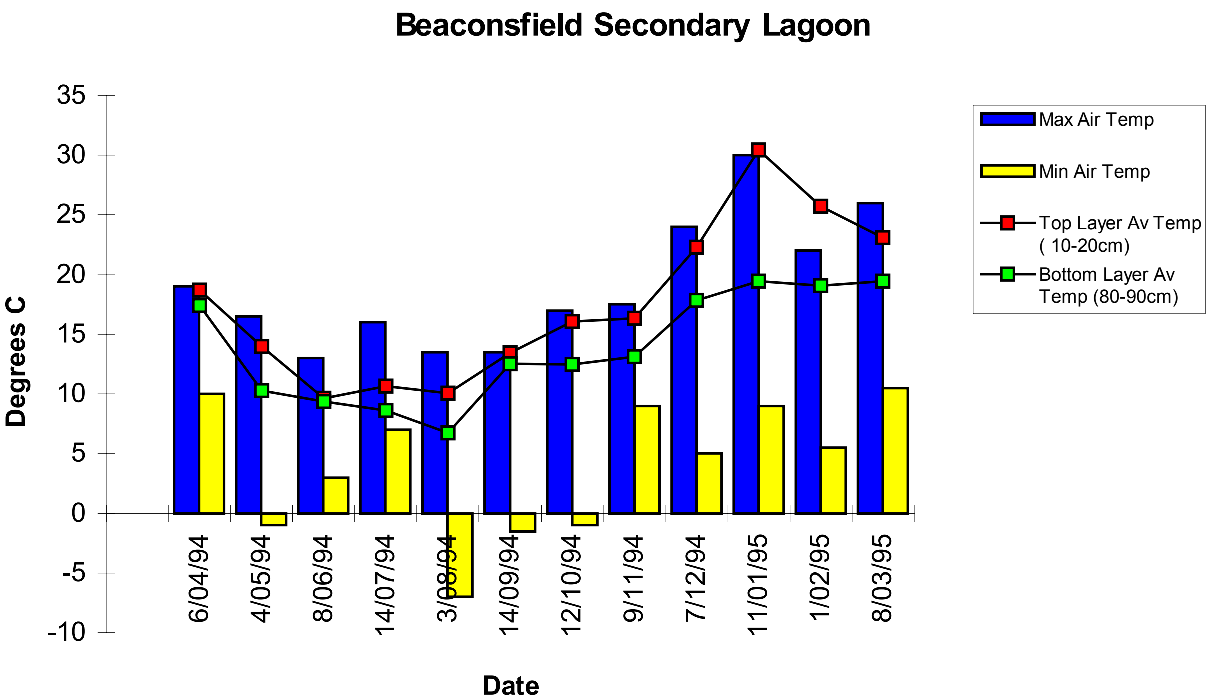


Figure 4.5 Air and lagoon temperatures variations for a stratified lagoon

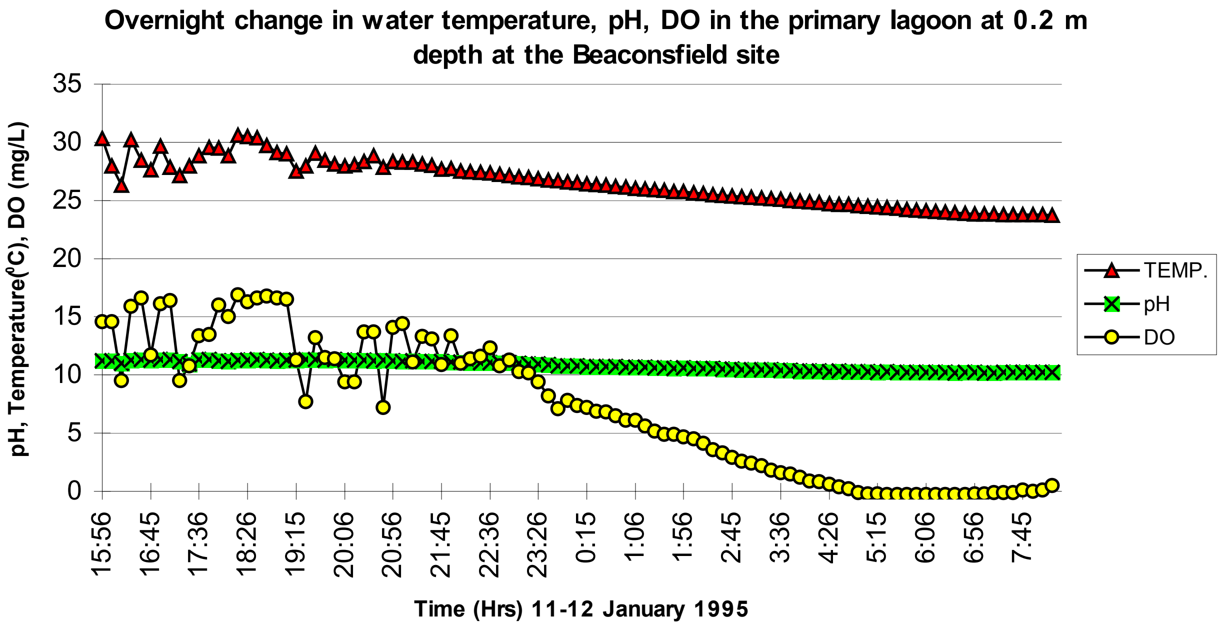


It is interesting to note the cooler bottom layers in a stratified lagoon may be part of the reason for inhibitory bacterial die-off rate in such lagoons compared to well mixed lagoons. Refer to section 4.7 for more detail on bacterial die-off rate.

From the Figures 4.1, 4.2 and 4.3 on diurnal temperature changes in lagoons at 0.2m depth and information in Figures 4.4 and 4.5 on ambient and lagoon profile temperatures, it is evident that lagoon temperatures change accordingly with changes in the ambient air temperatures. Whether a lagoon system is stratified or fully mixed will dictate the effect this change in ambient temperature will have. In a well mixed system the heat transfer would be fairly evenly distributed. However, in a stratified system such heat transfer can be concentrated in the surface layer during a heat uptake period over the day and be distributed in a complex way within the profile during heat loss over night when stratification breaks down.

Figure 4.6 shows the fluctuation in water temperature and DO as the Beaconsfield primary lagoon cools down during the late afternoon and night in summer. The fluctuations were not wind induced as the wind over this period was only weak.. From Figure 4.7 it is evident that the lagoon was strongly stratified over the day prior to the recorded night data. The temperature of the surface layer overnight approaches the temperature of the bottom layer (detailed in Figure 4.5 for 11 January 1995). It is proposed that the surface cools due to heat loss to the atmosphere and this denser cooler surface water is being replaced with warmer sub-surface water. This mechanism of thermal mixing is termed penetrative convection. This heat loss induced inversion mechanism occurs repeatedly until an equilibrium of mixed states is reached within certain profile depth ranges. As the density of the water within the lagoon approaches uniformity the whole lagoon contents are more prone to mixing by wind action. Figure 4.6 suggests that inversion commenced at about 7 pm, followed by approximately 3 hours of instability and then a steady decline in temperature.

Figure 4.6 Heat loss overnight through lagoon inversion due to density changes



The fluctuation events of DO and temperature are similar. This may in part be due to the sub-surface warmer water layers (with depleted DO levels) replacing the cooler (elevated DO) surface layers. The DO changes may also be due in part to the cessation of photosynthesis within the algal population due to lack of sunlight.

4.4.3 Sunlight

It is well established that UV radiation from the sun assists in bacterial reduction in wastewater. This mechanism for disinfection in lagoon systems operates to varying levels of effectiveness throughout the system depending on lagoon conditions such as turbidity and suspended solid levels in the wastewater. The high organic load in the primary lagoon and its normally high turbidity level maintain an environment for bacteria to thrive. The study, however, did showed good bacterial reduction in the primary cell of the pathogen indicator organisms, E.coli and faecal streptococci. This was probably due to a large proportion of bacteria being killed by UV light near the surface and particulate adsorbed bacterial settling and being digested through anaerobic activity in the lagoon sludge.

Although incident light flux was not measured during this study, it is well accepted that the available light determines, to a large degree, the level of photosynthetic activity, and hence oxygen production. Oxygen availability to the aerobic bacterial organisms is vital to ensuring optimal organic carbon stabilisation and odour prevention.

Figures 4.1 and 4.2 illustrate how significantly a primary and secondary lagoon can become depleted in oxygen overnight, particularly in summer, when the rates of bacterial action, which continues at night, are high due to the warmer temperatures. Continued high oxygen consumption by aerobic bacteria and algae, coupled with an absence of photosynthesis by algae at night deplete the oxygen levels. This oxygen sag would then lead to a significant reduction of the activity of the aerobic bacteria and their rate of breakdown of the organic material for the remainder of the night. With a severe depletion in oxygen, a lagoon can become odorous during the night where there is no aerobic layer to oxidise the gases arising from anaerobic bacterial activity which may dominate during this period.

4.5 Seasonal Effects

The seasonal effects, which are detailed in Table 4.2, were determined by using the percent removal of pollutants over the lagoon systems studied. The percent removal is the percent reduction in concentrations of parameters from the influent to the outlet of the primary cell and the outfall of the lagoon system.

The seasonal effects on lagoon performance, for example in terms of biochemical degradation, were not easy to quantify over the two year study (i.e., at best two repetitions only of each season was possible). Some of the variations detected in lagoon performance could be attributable solely to general fluctuations, and not related to particular seasonal variation. The expected drop-off in lagoon performance during the winter months was only apparent with some performance measures and the trends were not always clear cut. It was nevertheless observed that if a facultative or aerobic lagoon system did deteriorate to sustained anaerobic conditions over a winter period it could take several months to recover.

If the study had been conducted over a longer period it may have been possible to clarify the seasonal trends of some of the performance parameters which did not appear to illustrate strong trends.

4.5.1 BOD removal

Lagoon systems which are well exposed to the elements, such as the Turners Beach site, are well mixed and the BOD removal performance is consistently high fluctuating between 80 - 90 % regardless of the season. The Fingal site, which had an aerator, was also unaffected significantly by seasons. In systems which have a tendency to stratify, however, BOD removal performance is reduced. For example, the Beaconsfield lagoon system, with its frequently occurring stratified states, performed optimally during the summer months, up to 90% BOD removal (in February 1994 and 1995), and decreased in performance over autumn and winter months to 60% (in June 1994).

Table 4.2 Seasonal impact on the % removal for primary cells and over the lagoon system

% Removal for Primary Cell

Parameters	Season	Sewage Lagoon Sites											
		Beaconsfield		Bothwell		Campbell Town		Dover		Fingal		Turners Beach	
		1994	1995	1994	1995	1994	1995	1994	1995	1994	1995	1994	1995
BOD	Summer	75	83	78	82		64		70	85	79	77	60
	Autumn	75	73	72	80		75	56	69	83	86	68	78
	Winter	82	89	83	78		85	83	72	81	90	69	
	Spring	73		81	80	76	79	71		81	79	72	
SOL BOD	Summer		88		82		72		87	88	80	72	
	Autumn	93	92	82	83		84	80	91	87	86	71	85
	Winter	93	95	90	80		90	84	88	87	92	69	
	Spring	87		84	85	89	80	89		74	93	67	
NFR	Summer	61	62	25	69		49		-33	78	76	70	34
	Autumn	26	15	71	73		48	-38	21	57	74	49	60
	Winter	64	72	59	80		70	70	54	85	82	53	
	Spring	51		77	72	30	44	24		81	66	51	
E.coli	Summer	99.9660	99.5888	93.3463	92.6070		96.6722		91.7991	99.0388	96.6359	99.1607	91.5461
	Autumn	99.7080	99.0166	96.0324	93.4202		98.7400	83.2634	96.4047	96.1553	96.7240	98.4121	97.5047
	Winter	98.3669	99.6960	95.2931	87.2100		97.8434	99.0888	93.3054	96.3956	97.5970	98.3894	
	Spring	99.6960		96.8456	96.3035	98.6067	99.9612	98.6363		88.4658	96.0351	93.8524	
Total N	Summer		49		46		50		60		49		
	Autumn		32		28		35		57		48		
	Winter		55		15		31		45		34		
	Spring	43		45		43	30	39		38	33	27	
Ammonia	Summer	89	60	80	55		74		100	86	61	54	
	Autumn	83	54	13	15		35	45	87	73	50	33	
	Winter	54	50	9	-4		24	21	39	37	23	11	
	Spring	48		33		69	40	42		21	50	15	
ortho-PO4	Summer	62	4	-23	-9		-8		53	71	10	17	
	Autumn	49	-26	-52	-5		3	-67	78	57	22	15	
	Winter	45	44	7	31		17	3	24	23	14	-18	
	Spring	21		-9		12	18	31		-7	35	-4	
Total P	Summer	30	-14	-16	-3		-8		19	61	15	11	
	Autumn	16	-21	-13	7		3	-39	70	49	45	1	
	Winter	36	52	15	27		17	10	38	42	42	5	
	Spring	14		6		12	18	20		27	28	-7	

Note: * Those primary lagoons which are baffled.
For each plant there may be results for 1994 and/or 1995.

% Removal over whole lagoon system.

Parameters	Season	Sewage Lagoon Sites											
		Beaconsfield		Bothwell		Campbell Town		Dover		Fingal		Turners Beach	
		1994	1995	1994	1995	1994	1995	1994	1995	1994	1995	1994	1995
BOD	Summer	85	86		82		64		67	84	85	85	87
	Autumn	82	79	67	86		85	57	64	85	87	86	87
	Winter	68	73	84	89		90	84	74	85	89	85	
	Spring	77		82	79	74	95	70		83	83	86	
SOL BOD	Summer	90	92	93	85		83		58	86	86	84	86
	Autumn	93	94	83	90		89	88	90	90	93	91	84
	Winter	88	82	91	91		95	88	88	89	92	92	
	Spring	92		84	88	93	94	90		73	96	89	
NFR	Summer	64	52		71		26		-16	76	81	67	66
	Autumn	37	49	65	76		71	-22	26	75	77	51	86
	Winter	58	63	51	87		89	65	55	89	83	66	
	Spring	61		78	69	37	98	23		90	72	77	
E.coli	Summer	99.9942	99.9946	99.6303	99.9113		99.9980		93.8540	99.5434	99.8678	99.8837	99.9493
	Autumn	99.9893	99.9881	99.7462	99.6932		99.9976	95.9088	98.0288	99.7705	97.0764	99.9962	99.9955
	Winter	99.9338	99.9978	98.3957	99.6119		99.9952	96.2808	86.9827	97.3567	99.7357	99.9924	
	Spring	99.9928		99.4776	99.9926	99.9982	99.9976	96.9006		97.0412	99.8438	99.9962	
Total N	Summer		72		54		61		55		57		63
	Autumn		63		44		68		54		58		28
	Winter		57		33		41		43		38		
	Spring	69		48		56	39	41		39	47	62	
Ammonia	Summer	98	96	96	74		98		99	89	74	89	79
	Autumn	97	83	18	49		82	49	89	76	67	85	32
	Winter	95	66	13	4		28	23	42	37	32	52	
	Spring	82		32		78	13	41		58	67	55	
ortho-PO4	Summer	76	51	55	10		10		25	76	26	21	-73
	Autumn	65	-18	-34	45		27	-65	82	60	44	40	-128
	Winter	42	43	15	50		-19	-4	36	22	21	44	
	Spring	43		-5		1	-4	30		0	50	26	
Total P	Summer	57	30	-11	12		4		5	61	17	0	-49
	Autumn	42	2	-15	25		32	-42	71	45	46	13	-34
	Winter	45	43	18	47		9	10	44	43	43	52	
	Spring	31		6		12	20	24		25	39	23	

The primary cell BOD removal performance for generally well mixed cells appeared independent of seasonal variation, while the removal performance of primary lagoons which stratified improved slightly over winter.

In general, the % removal of soluble BOD appeared to fluctuate non-seasonally between 80 and 95%.

4.5.2 NFR removal

The % removal of NFR decreased over late summer and/or autumn for the lagoons studied to as low as approximately 30% for some months. The % removal performance, however, improved to approximately 90% for some lagoons during winter and/or early spring months. The aerated Fingal site had a less dramatic depression in summer/autumn performance at approximately 65% from a peak performance in winter/spring of 90%.

This apparent decline in NFR removal over summer and early autumn could be due to the confounding effect of algal cells, which grow significantly over the summer months, on the measurement of suspended material, and not reflect the underlying trend in the non-algal NFR.

4.5.3 E.coli removal

The % removal of E.coli generally declined over the cooler months, however those lagoon systems studied which were well mixed, such as Turners Beach, and had three or more cells, such as Beaconsfield and Campbell Town showed only slight reduction in performance.

4.5.4 Total N removal

The % removal of total nitrogen was optimal over summer and decreased over autumn and winter. The performance of a stratified lagoon system, such as Beaconsfield system decreased from 75% (in January 1995) to 55% (in June 1995), while the fully mixed lagoon systems, such as Turners Beach decreased from 70% (in December 1994) to 30% (in May 1995). Poor performance in winter is most likely due to poor ammonia volatilisation rates at the lower lagoon pH values, generally between 7 and 8 during this period: summer pH values generally ranged between 9 to 10.

The % removal of ammonia was optimal over summer at approximately 90%. It declined over autumn and winter to as low as 60% for stratified lagoon systems and to 10-30% for moderate to well mixed systems.

4.5.5 Phosphorus removal

The % removal of ortho-phosphate and total phosphorus over the stratified lagoon systems appeared to be optimal in warmer months at 80 and 60 % respectively, and decreases during autumn and/or winter months. Other, non-stratified, systems generally fluctuated erratically in a non-seasonal manner. At times there was either no phosphorous removal or an increase in phosphorus over the lagoon systems in all cases.

The % removal of phosphorus in the primary cell in some of the lagoons studied is highest over autumn/winter. These data suggest that the phosphorus accumulating in the sludge settling over autumn/winter is being released in the warmer months when the sludge is being more actively digested.

4.5.6 Lagoon turnover

The turning over of lagoons in spring was frequently observed in the lagoon systems studied. It is generally understood that turnovers develop when the lagoon warms up and the level of biological activity increases. The gas produced via bacterial fermentation under anaerobic/anoxic conditions in the sludge becomes trapped in the sludge causing it to float to the lagoon surface. Methane and hydrogen sulphide gases are subsequently released to the atmosphere in large quantities potentially causing offensive odours.

Furthermore, algae growth in lagoons would go through stages of exponential growth and then suffer large die-off because of nutrient and/or oxygen deficiency or cold temperatures particularly in autumn. Decomposing algae would create significant oxygen sags in the lagoon and result in fermentation gases being released. Under aerobic conditions in the surface layers of the lagoon cell, these gases are normally effectively oxidised. However, in severely oxygen depleted conditions, gases produced by anaerobic reactions are not oxidised and odours can develop. Refer to Chapter 8 Table 8.3 for detail on controlling odours through chemical dosing and aeration, and other trouble shooting measures with lagoon system operation and management issues.

4.6 Wind Mixing And Stratification

Lagoons are often defined as either facultative or aerobic lagoons: facultative lagoons have depleted oxygen in the bottom layer and aerobic lagoons have oxygen throughout the lagoon depth. Such classifications in practice can be misleading as the study showed that nearly all lagoons are capable of being either facultative or aerobic depending on the amount of mixing and light penetration. Primary and secondary cells alternate between aerobic and facultative states especially over a 24 hour period although primary cells do normally become facultative more quickly than secondary lagoons due to their higher organic load. Figures 4.1 and 4.2 in Section 4.4 illustrate, for example, that a normally completely mixed aerobic lagoon system during the day alters to a facultative (or anoxic) system at night.

What is important when assessing lagoon performance is how often a lagoon becomes facultative and/or stratified. Stratification is where there is a defined abrupt change in the level of a parameter such as DO and/or temperature with depth. The real dangers of DO stratification include localised depletion of the aerobic zone, typically near the inlet during the night, and widespread oxygen depletion over much of the lagoon (sometimes indicative of severe overloading). Both situations lead to odour formation due to the activities of anaerobic bacteria.

Aerobic layers within lagoons are driven by wind action. With the promotion of wind assisted mixing, the interaction of oxygen, waste solids and bacterial communities is enhanced. Whether or not fully mixed or stratified conditions are desirable depends on the pollutants the lagoon cell is designed to remove. If the cell is to optimally remove organic load and reduce bacterial levels then a fully mixed reactor is desirable. If, however, optimal nutrient removal is desired then alternating stratified and fully mixed states were found in the study to result in the greatest removal.

During the monitoring of the Beaconsfield lagoons on consecutive days it was found that a cell can destratify and stratify within a 24 hour period due to wind action.

The study data showed stratification often occurred due to the surface layer being a higher temperature than the bottom layer. This situation is stable as water density decreases with increased temperature. Therefore the warm water floats on top of the denser cold water. The profiling work, using the submersible data logger (SDL), showed stratification occurring with the top layer being up to 9^0C warmer than the bottom layer. The boundary between the two layers is typically very sharp, i.e., within 5-10 cm. As mentioned previously in this chapter, the surface of a lagoon frequently drops in temperature up to 5^0C overnight. In this situation, the top layer is likely to drop to similar temperatures to bottom layers and thus as the densities are becoming similar minimal wind energy would be required to mix the two layers.

On occasions it was observed that the surface layer had significantly higher DO than the bottom layer despite there being no detectable difference in temperature between the two layers. This suggested that stratification in DO levels can occur simply from lack of mixing. A possible reason for this is that algae in the surface layer absorb most of the light and produce oxygen through photosynthesis. The algae in the bottom layer are starved of light and they therefore do not produce as much oxygen or they may have a net oxygen consumption through respiration.

The SDL used in the study was also fitted with a spectro-fluorometer which enabled *in situ* tracing of a dye (Rhodamine WT) to ascertain the fluid dynamics in lagoon cells. The dye tracing illustrated that the lagoons are homogeneous in nature and under stratified states movement is preferential along certain strata.

Figure 4.7 and 4.8 present the profiles of the DO levels versus depth over 12 months at two sites, Beaconsfield and Turners Beach. The Turners Beach site is well exposed to the elements and this is observable from Figure 4.8 which mostly shows a completely mixed lagoon state on those occasions when the site was monitored. However, the Beaconsfield site with its moderate interference to wind action had a high incidence of stratification when monitored, and on several occasions stratification lead to zero DO below approximately 0.5 m depth during the day.

Table 4.3 summarises the comparison of wind speeds experienced at each of the lagoon sites over the period of the study and the occurrence of lagoon stratification. It can be seen from the data that it is reasonable to use the Turners Beach lagoon as indicative of the wind speed necessary to destratify a lagoon cell which has full exposure to wind action: approximately 2.8 m/s, provided the wind action is not limited by barriers around the perimeter of the cell or local topography.

Table 4.3 Destratification wind speeds

Location	Wind barriers	Range within which [A] destratification wind speed (DWS) occurred (m/s)	Percent occurrence of at least [B] the DWS being reached over the period of the study (%)	Percent occurrence of stratification (%)
Beaconsfield	Mod	3.4 - 5.2	56 - 41	44 - 59
Bothwell	Min	1.5 - 2.1	68 - 59	32 - 41
Campbell Town [C]	Min	2.6 - 5.9	67 - 32	33 - 68
Dover	Mod	2.1- 2.9	67 - 43	33 - 57
Fingal [D]	Maj	Occasionally at 1.5, 2.6	Rarely	Often
Turner Beach	Min	2.8	93	7

Note: **A**: Range exists because of insufficient profiling data at the critical wind speed for the exact point of destratification.

B: The percent occurrence is the percentage of times over the length of the study that the DWS or greater is reached. This can be equated to the % occurrence of stratification, e.g. Turners Beach lagoon system with a 93% percent occurrence of the DWS or greater being reached would result in the lagoon being stratified only 7% of the time.

C: Insufficient data in the critical range, however 62% of the time the wind speed is at or greater than 2.85 m/s.

D: Primary Lagoon aerated, however secondary lagoons were strongly stratified.

Table 4.4 summarises the removal performance of lagoon systems with different mixed states. With moderate interference around the lagoon system to wind action, such as at Beaconsfield, the lagoon cells stratify fairly frequently, approximately 50% of the time, during the warmer months when stratification

Figure 4.7 Dissolved oxygen v depth for the Beaconsfield lagoon system

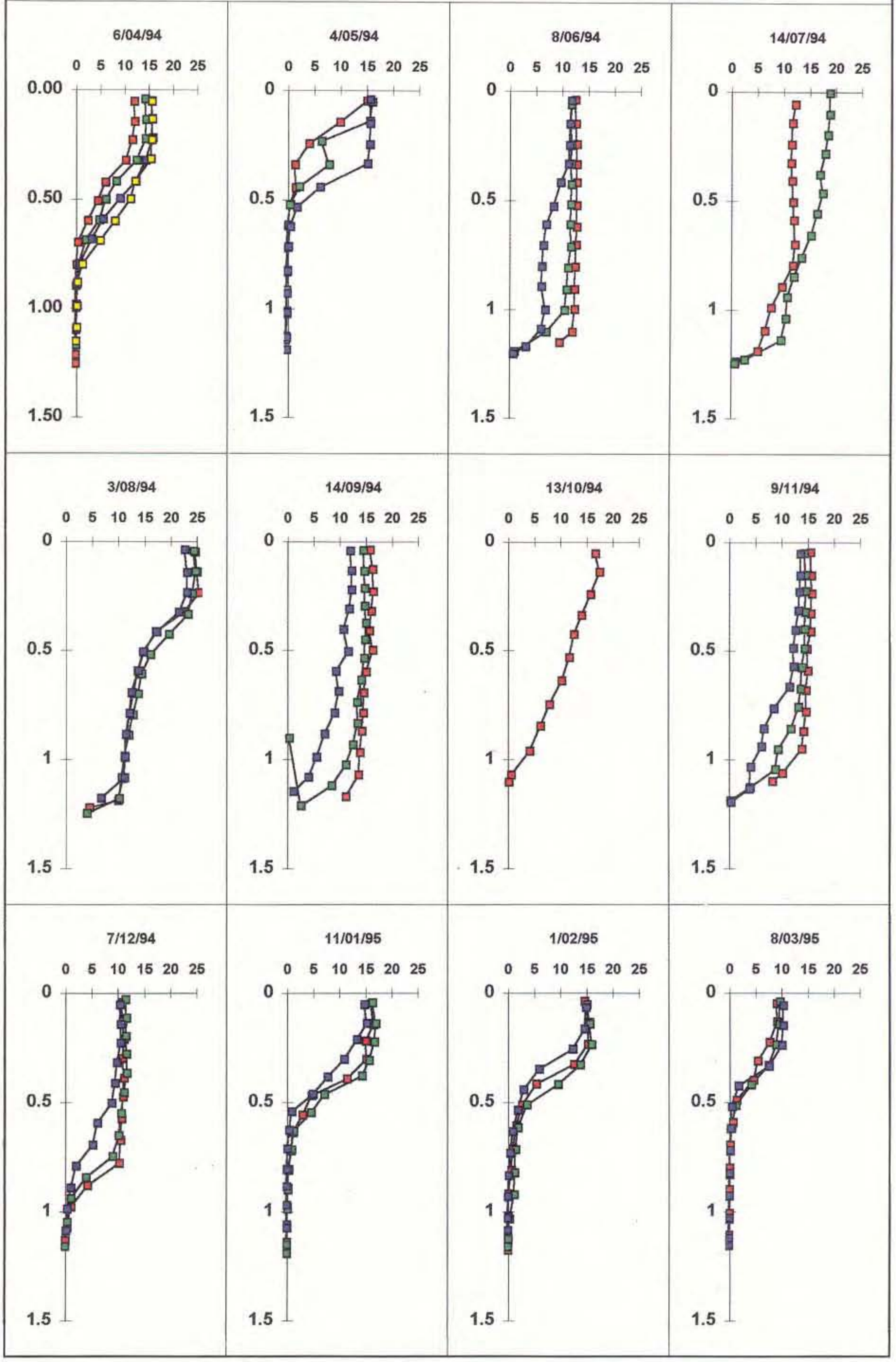
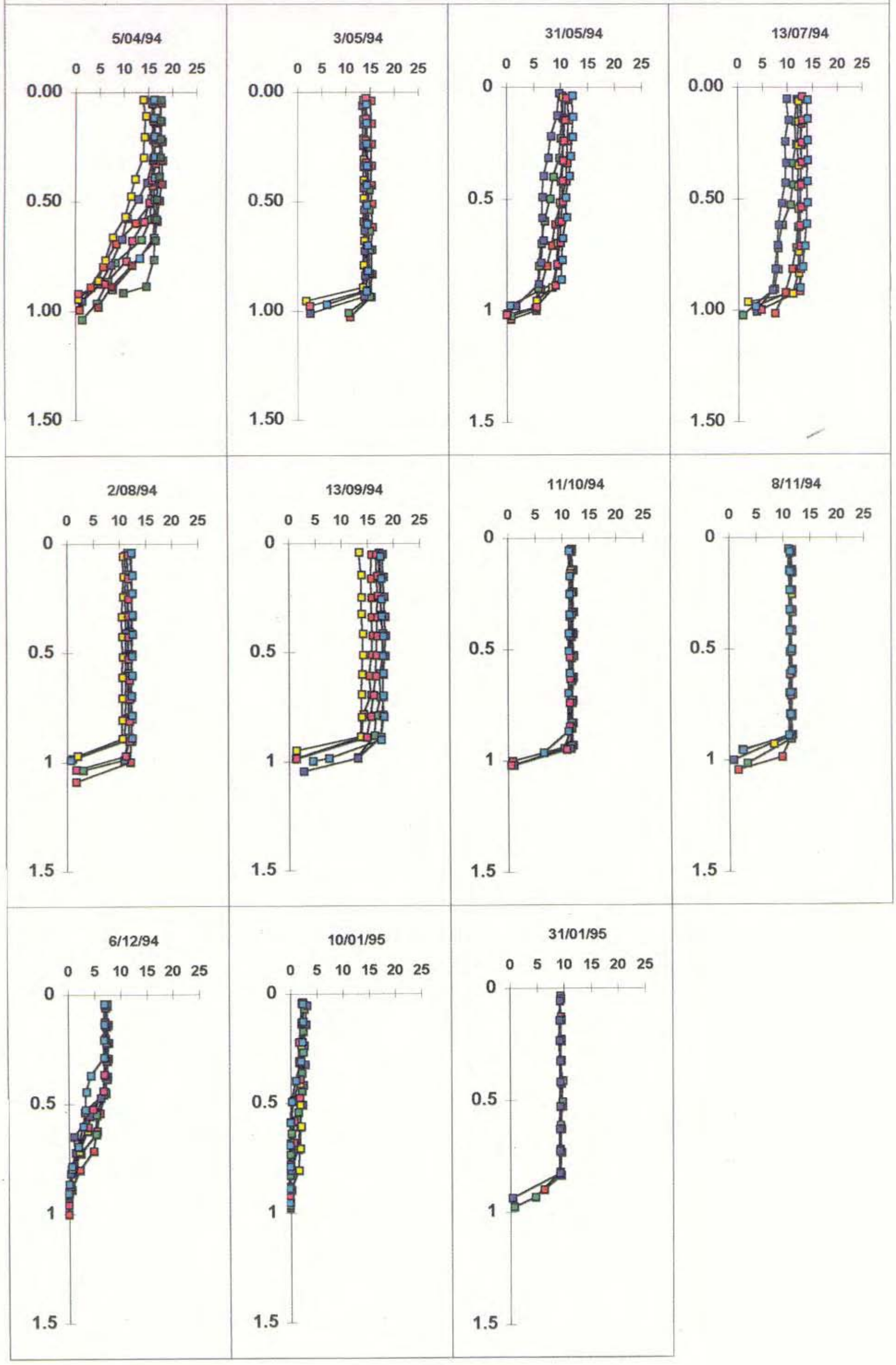


Figure 4.8 Dissolved oxygen v depth for the Turners Beach lagoon system



is significant. This situation culminates in poorer removal of BOD and NFR (and E.coli; refer to Figure 4.9) for the system.

Table 4.4 The mean percent removal of pollutants over a 50 contiguous day period.

Parameter	Locations and Mixing Status				
	Beaconsfield	Fingal [3]	Bothwell	Campbell Town	Turners Beach
	Poorly Mixed	Poorly Mixed	Mixed	Mixed	Well Mixed
BOD	54	70	71	64	73
SOL. BOD	62	73	77	73	75
NFR	37	66	58	49	56
Total N.	46	42	39	44	49
NH_3	61	57	30	53	60
PO_4	29	32	12	3	3
Total P	24	32	11	12	9

Note:
1. % NO_x removal not determined as influent values should be zero.
2. This data is derived from determining the percent removal over one day for each of the systems studied and then multiplying by 50 to represent 50 days HRT. This approach is idealised for comparative purposes only.
3. The Fingal primary lagoon is aerated and fully mixed, while the secondary cells are stratify strongly and poorly mixed

Tables 4.3 and 4.4 indicate that, for lagoons (such as Beaconsfield) that have a moderate tendency to stratify, approximately 45-55% of the time, there is a decreased removal performance in BOD, soluble BOD, and NFR compared to lagoon systems with infrequent periods of stratification. However, stratified lagoons have significantly greater phosphorous removal. The removal of nitrogen in both situations appears to be comparable.

4.7 Factors Affecting Bacterial Die Off

4.7.1 E.coli k values

The k value is the rate constant used in die off equations. The higher the value of k the greater the rate of die off and hence, generally, the better the performance of a lagoon. The k value for each study lagoon cell was calculated from fitting study data to the first order die off equation, i.e., the Marais Model for a single cell,

$$N = \frac{N_o}{kt + 1}$$

Refer to Chapter 6, Section 6.4.4, for further information and a description of this equation and its terms.

The average bacterial die off performance for the studied primary cells is illustrated in Table 4.5 along with the factors that could impact on this bacterial die off.

Table 4.5 E.coli k values for the primary cells (first cells) studied with some factors that influence them

Location	BOD_5 Load (kg/ha.d)	HRT (Days)	Mixing Status	Multi Inlet (Y/N)	Partial Baffle (Y/N)	E.coli k Value (0 0.5 1 1.5 2 2.5 3 3.5.)
Beaconsfield	71	26.7	S	Y	N	3.4
Bothwell	56	32.4	M-S	N	N	1.9
Campbell Town 1	70	38.4	M	Y	Y	2.6
Campbell Town 2	70	32.1	M	Y	N	2.0
Campbell Town 3	70	32.1	M	N	N	1.7
Fingal	69	18.9	A	N	N	0.44
Turners Beach	75	25.6	M	N	N	1.8

Note:

BOD_5 Load: The BOD_5 Load on the primary cell.
HRT: The Hydraulic Retention Time of the cell.
Mixing Status: The mixing status is a representation of how well the lagoons are mixed.
A = Aerated, M = Stratifies less than 25 % of the time. S = Stratifies more than 50 % of the time. S-M = Stratified Between 25 and 50 % of the time.
Multi Inlet: The primary cell has a multiple inlet for the raw sewage
Partial Baffle: The primary cell contains a baffle that travels part way across the lagoon to prevent short circuiting from the lagoon's inlet to outlet.
E.coli k Value: The E.coli k value calculated from the average concentrations of E.coli (orgs/100mL) sampled for each cell.

The bacterial die off rates in the primary cells (Table 4.5) are generally quite high when compared to those observed in secondary cells (Figures 4.9-4.11). The high proportion of solids settling in the primary with the associated high concentrations of adsorbed bacteria is considered to be a major factor for this enhanced bacterial reduction. The settled solids are reduced through anaerobic/anoxic digestion with the pathogen indicator aerobic faecal bacterial number being substantially less at the primary lagoon outlets. The reason for differences in bacterial performance between the primary cells is not well understood. It may be related to the extent of mixing within the lagoons. With those cells less mixed compared to other cells, through poorer wind action, such as the Beaconsfield site, settling is enhanced.

The worst performing primary cell is Fingal which is highly loaded and needs an aerator to keep it aerobic. This poor performance may be due to the suspended material (and the associated high attached bacterial biomass) being carried over into the next cell for clarification. In the other primary cells, which are not heavily loaded and aerated, the level of suspended partially treated organic load reaching the primary outlets would be significantly less.

Considering all the primary cells, except Fingal as it is mechanically aerated, and accepting that shorter resident time should result in lower k values if the influent organic loads are generally similar, it is interesting to note that the frequent development of a stratified state through poor mixing within the primary cell (at Beaconsfield) seemed to improve the bacterial performance of the primary cell. Settling probably, however, is one of the main factors that influences E.coli levels at the primary lagoon outlet.

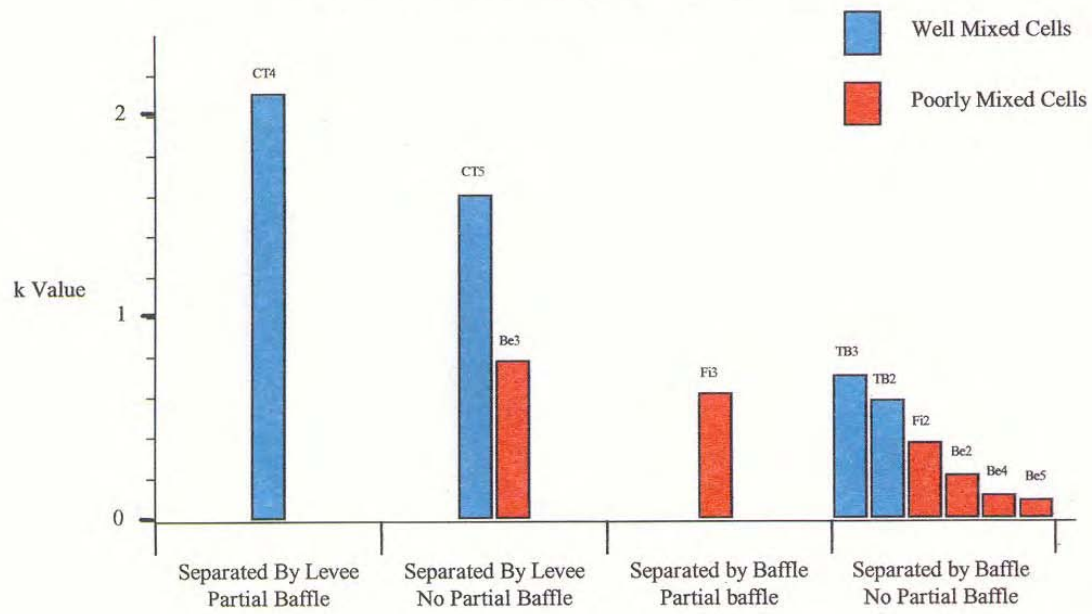
Different inlet and baffle configurations were developed for the three Campbell Town primary lagoon cells to determine what effect the modifications would have on bacterial die off rates. This comparison was possible as the lagoons had basically the same dimensions and were subjected to the same external influences. Campbell Town (CT) 1 was modified to have a multiple inlet and partial baffle arrangement while CT 2 just had multiple inlets. CT 3 was left with its original surface "third of the lagoon length" inlet. From the data in Table 4.5 it is evident that multiple inlets and partial baffling of the primary cell to minimise short circuiting can improve bacterial removal. Even though the theoretical resident time of CT 1 with the baffle is 20% more due to it being deeper than CT 2 without the baffle, the benefit of including a partial pre-fabricated baffle is observed.

From the data summarised in Figures 4.9-4.11, the effects of different design and environmental conditions on the E.coli k value for secondary lagoons in the study can be observed. The data suggest that the three significant factors are stratification, type of separation between cells, and partial baffling within cells.

4.7.2 Effect of cell mixing

Figure 4.9 shows that the amount of mixing within lagoon cells has a large impact on E.coli die off. All cells that were well mixed had improved performance over cells that were not as well mixed with the same type cell separation and internal baffling. It is therefore important to establish the mixing status, or the likely mixing status of lagoons when assessing their performance or designing a lagoon.

Figure 4.9 Comparison of E.coli k values for well mixed and poorly mixed cells (codes refer to location and cell number, e.g. CT4 = Campbell Town cell 4)

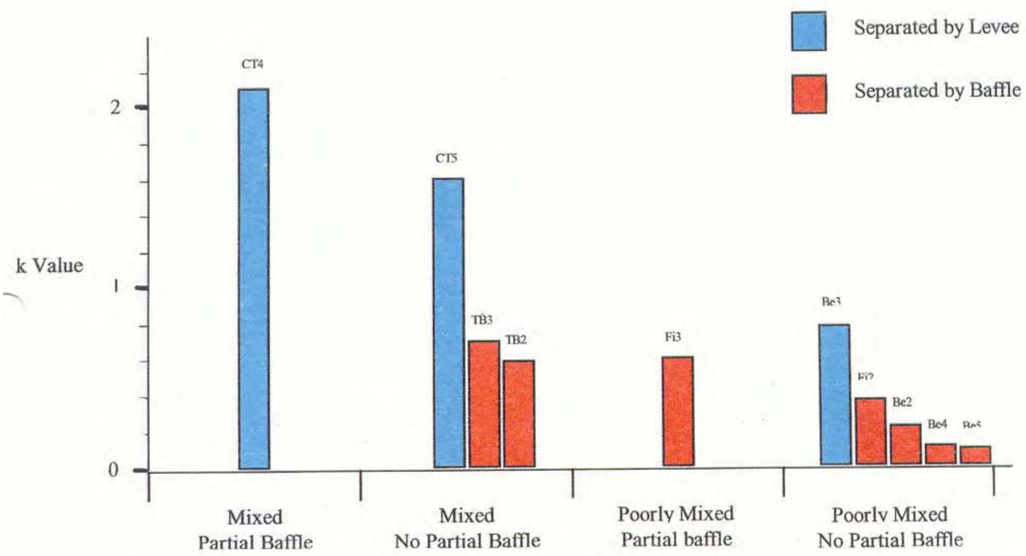


4.7.3 Effect of different types of cell separation

The way in which two cells are separated was found to impact significantly on the performance of the later cell. It was found that cells separated by a levee which altered the water level between two lagoon cells increased the performance of the later cell substantially. Decreased performance was observed for cells separated by baffles and walls without a drop in water level even if they were impervious, e.g., the situation of the Turners Beach lagoon system with a cement brick wall separating the second and third lagoons with only one subsurface opening connecting them.

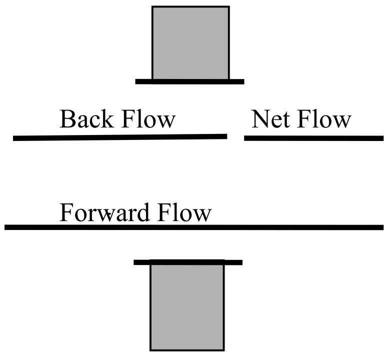
Figure 4.10 shows the effect of cell separation on the E.coli k values. It is evident that lagoon cells separated by levees, i.e. one way flow direction, have a much higher k value than the cells separated by walls and baffles, i.e., sub-surface free exchange in both directions between cells.

Figure 4.10 Comparison of E.coli k values for cells separated by levees or separated by baffles.



One of the possible reasons why levees perform better than baffles was recognised at this Turners Beach lagoon system. It was observed, when the second and third cells had substantially different colouring due to algae, that the water was actually flowing back and forth between the cells. Refer to Figure 4.11 to illustrate the flow pattern.

Figure 4.11 A schematic of water flow through an inlet in a cell wall or baffle.



The mass balance over the inlet gives:

Net Flow (NF) = Forward Flow (FF) - Back Flow (BF)

It follows where no back flow exists, i.e., BF = 0, then FF = NF. If back flow occurs, then FF > NF. For cells separated by levees, with a change in water level the forward flow is the same as the net flow due to there being no back flow.

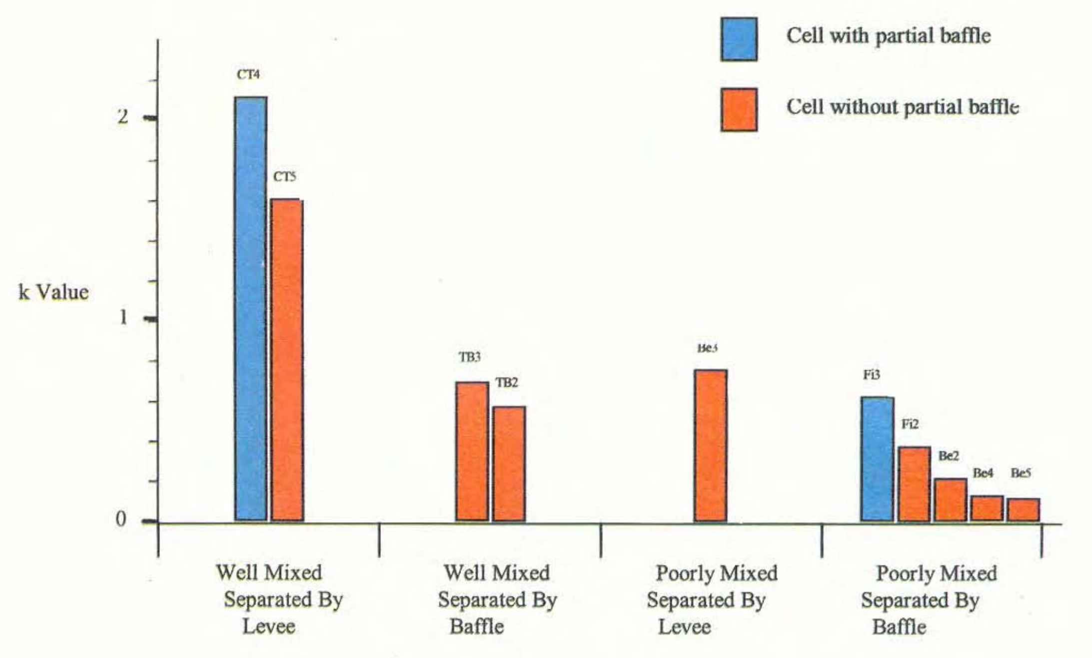
For cells separated by walls or baffles back flow would transport lower concentrations of bacteria than forward flow and the net flow must always add bacterial load to the receiving cell. The higher the back flow, the higher the loading. For a given net flow, an increase in (low bacteria) back flow must be balanced by an equivalent increase in (high bacteria) forward flow and a greater bacterial load would be added to the receiving cell than if the back flow was less.

4.7.4 Effect of partial baffles

The data in Table 4.5 and Figure 4.10 suggest that cells containing a partial baffle have increased E.coli die off. The two cells with partial baffles within them, have higher k values than other cells with the same type of cell separation and mixing status. It is believed that this improvement is due to the minimisation of short circuiting within the cells and possibly the enhancement of settling.

Figure 4.12 compares cells that contain internal partial prefabricated synthetic baffles to inhibit short circuiting with cells that do not. It is evident that a slight increase in k value may be achieved with a partial internal baffle.

Figure 4.12 Comparison of E.coli K values for cells containing a partial baffle or without such a baffle.



4.8 Organic Removal

4.8.1 Correlation of COD and BOD_5 in lagoon water

The main method of determining organic removal in Tasmanian lagoons is to use five day biochemical oxygen demand (BOD_5). As BOD is a parameter influenced by other factors such as algae, nitrification, toxic substances and anaerobic organisms, care needs to be given to the interpretation of the results. Total chemical oxygen demand (COD) was also determined and compared to total BOD. In the raw influent entering the lagoon system there was good correlation between the COD and BOD; refer to Figure 4.13. The total COD was approximately 2.1 times the total BOD.

Figure 4.13 Raw influent total COD v. total BOD for all the lagoon systems studied

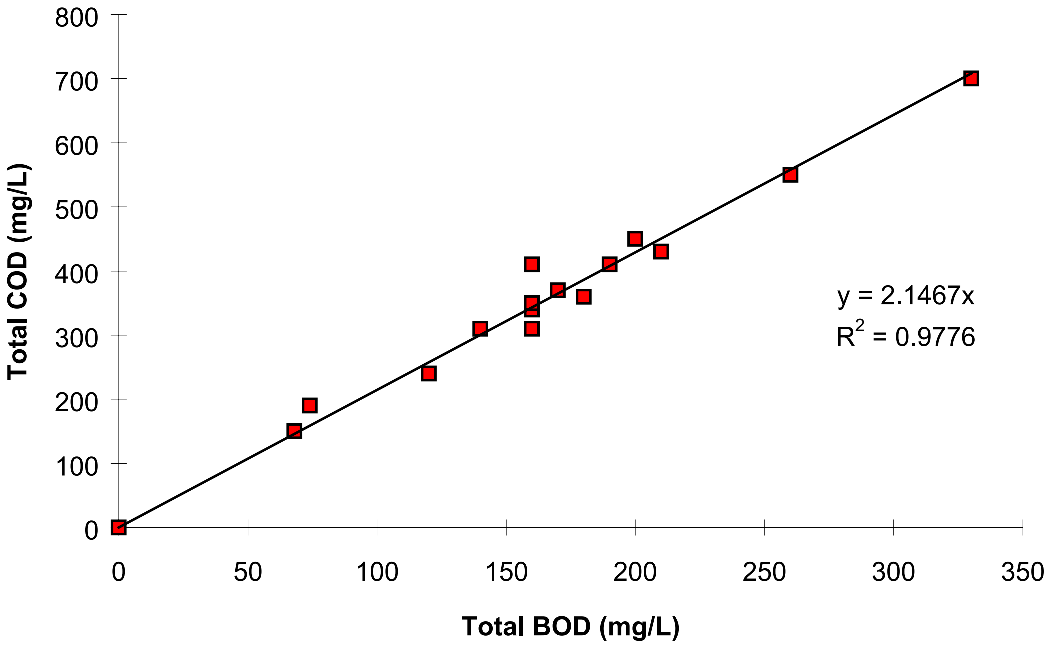
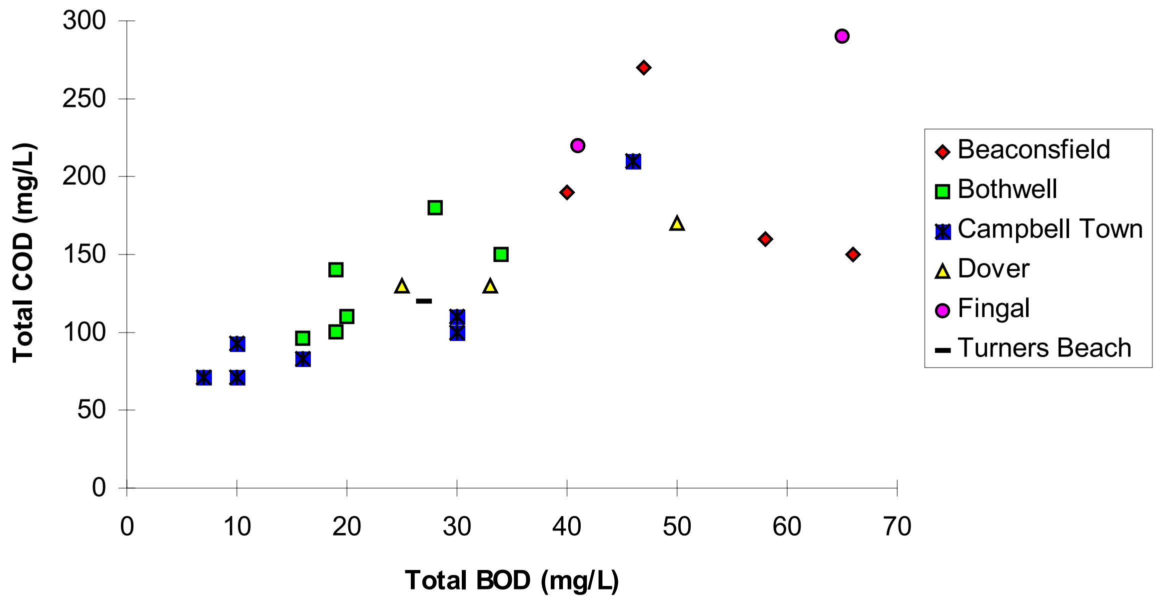


Figure 4.14 shows that there is correlation between COD and BOD in the final lagoon effluent, however it is poor particularly with the data for Beaconsfield which deviates from the trend substantially. The possible explanation for this poor correlation is that the BOD test measures the oxygen demand by carbonaceous oxidation (which is the oxygen required by bacteria to consume the organic material) plus the oxygen demand for nitrification (which is the oxidation of ammonia to nitrite and nitrate). Effluent from lagoon systems is not normally treated in the BOD test to inhibit nitrification, as with the influent, there is an expectation that it does not occur. However, since some nitrification does occur at times the BOD test can give misleadingly high values. It is thus recommended that with any BOD testing of lagoon effluent nitrification is inhibited in the sample. The method for BOD determination is detailed in Standard Methods for the Examination of Water and Wastewater (1995 Ed 19).

In most situations the nitrogenous oxygen demand will not have any significant environmental impact unless possibly discharging to a small water body. The nitrogenous oxygen demand (NOD) in a lagoon is beneficial as it illustrates that nitrification is an active process and there is an avenue for the nitrogen to be removed through denitrification before discharge.

Figure 4.14 Final lagoon effluent Total COD v. Total BOD



4.8.2 Correlation of COD to NFR

Figure 4.15 shows a strong correlation between COD and NFR in the effluent. As NFR is nearly entirely algae in the lagoon systems studied, as established by microscopic examinations, this suggests the main pollutant is due to the algae biomass leaving the lagoon system.

Figure 4.15 Final lagoon effluent Total COD v. Total NFR

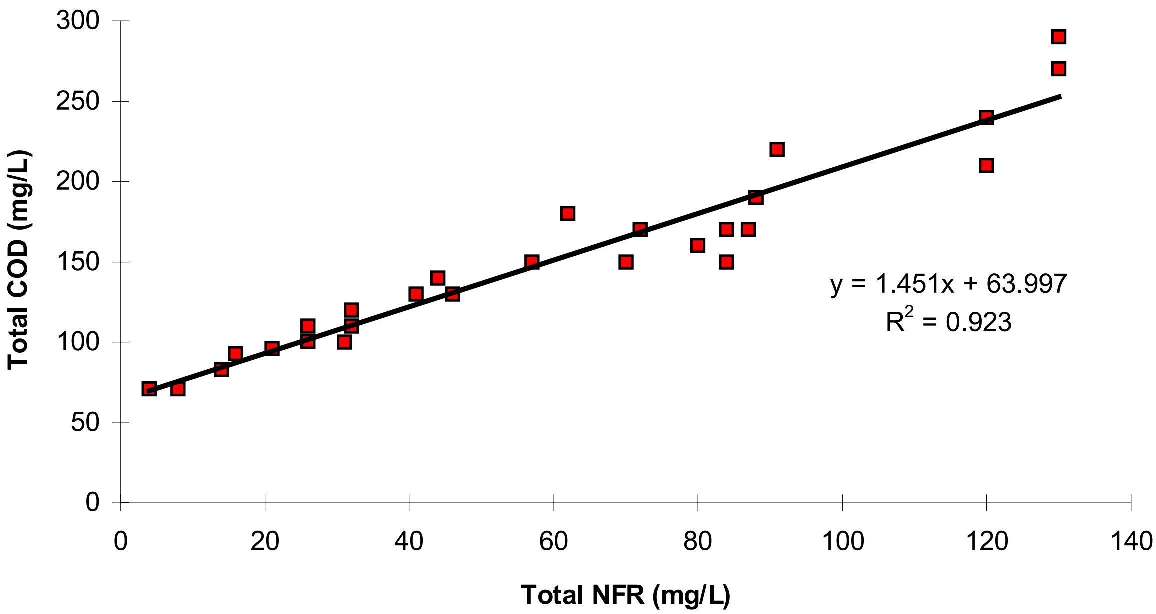
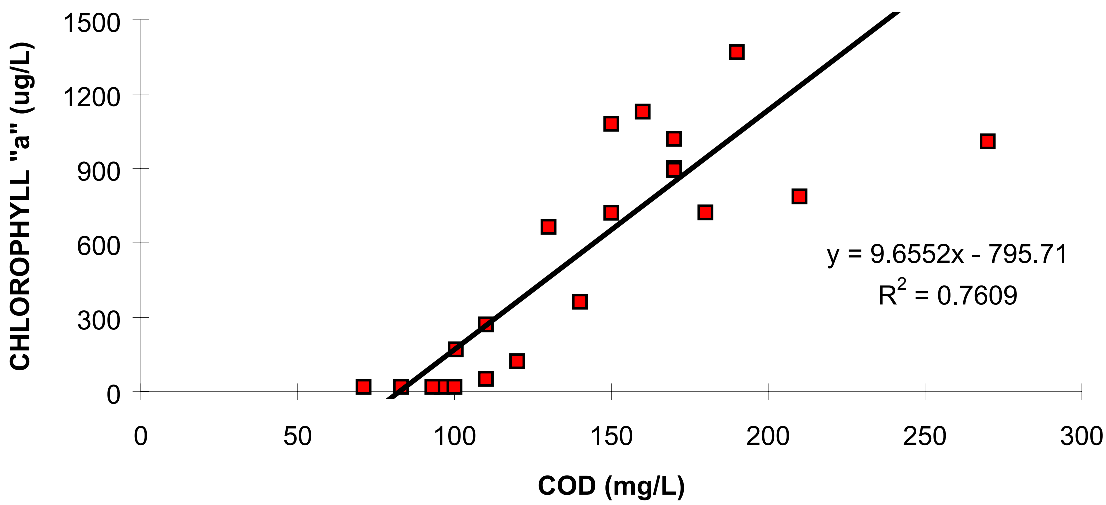


Figure 4.16 shows a less clearly defined correlation between chlorophyll “a” and COD, where at less than 100µg/L of chlorophyll “a” the correlation is very poor.

Figure 4.16 Final lagoon effluent chlorophyll “a” v. COD



4.8.3 Affect of algae on organic load

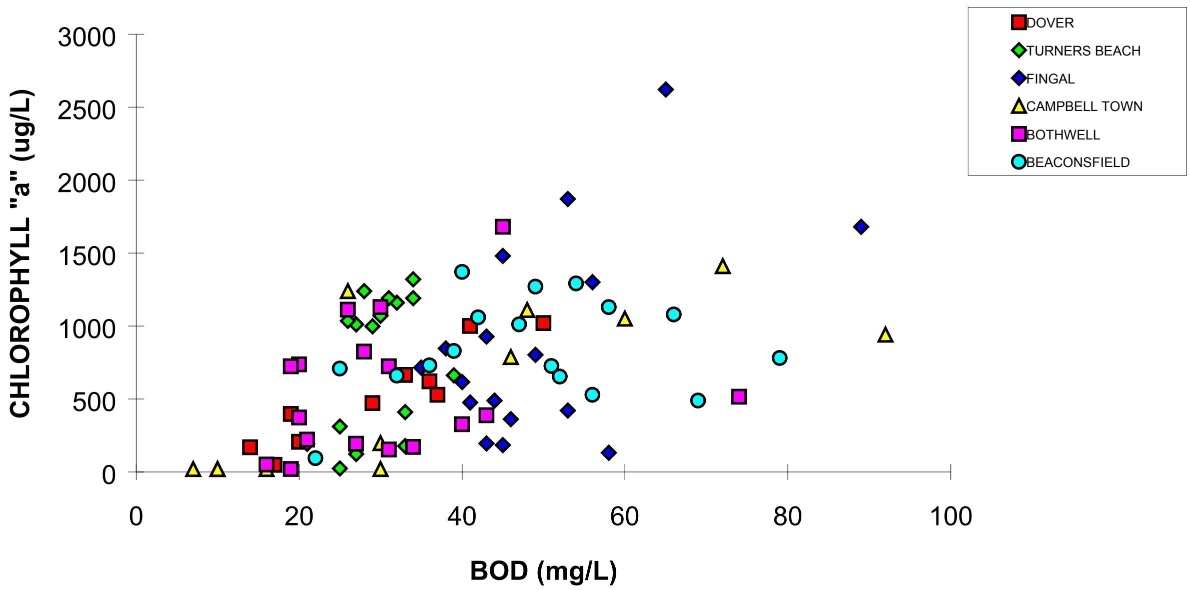
It can be seen from the summarised data in Table 4.6 that the average effluent chlorophyll “a” level ranges from 500 to 900 µg/L, with the generally warmer less cloud affected areas of the State, such as Beaconsfield, Fingal and Turners Beach, having higher figures than the generally cooler more cloud affected areas, such as Bothwell and Dover.

Table 4.6 Average chlorophyll “a” level in lagoon effluent

Location	Sample Number	Average Chlorophyll “a” (µg/L)	SE
Beaconsfield	17	848	81
Bothwell	17	549	110
Campbell Town	12	570	163
Fingal	17	889	167
Dover	10	513	104
Turners Beach	16	757	117

Figures 4.17 and 4.18 show there is very poor to poor correlation of chlorophyll “a” to BOD and NFR respectively. However, Table 4.6 and Figure 4.17 illustrate within the range of average effluent chlorophyll “a” the present Tasmanian BOD regulation discharge level of 40 mg/L, if the chlorophyll “a” and phaeophytin levels are greater than 100 µg/L, cannot be consistently met.

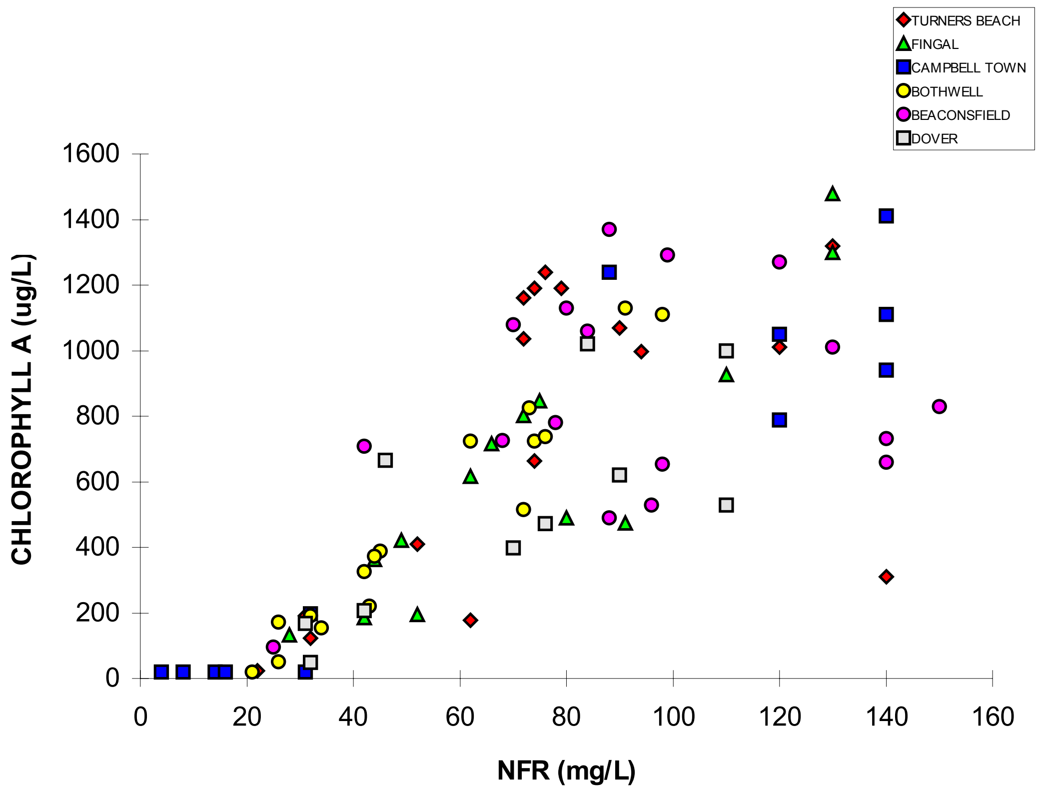
Figure 4.17 Chlorophyll “a” v. BOD for the lagoon systems studied



Additional tertiary treatment, such as sand filtration discussed in Chapter 6, Section 6.7, would be required to consistently achieve the 40 mg/l BOD limit. As previously discussed (section 4.8.1) the standard BOD test may give elevated measurements when applied to sewage lagoon effluent due to nitrogenous oxygen demand, which itself is not always directly related to the potential environmental harm which could be caused by the release of the effluent. Depending on the nature of the effluent and receiving waters, a more environmentally appropriate tests may be the nitrification inhibited BOD test, COD test or a COD v. Chlorophyll a test.

Figure 4.18 shows that with lagoon effluent from a primary/secondary lagoon treatment process a low NFR level cannot be consistently met. The present Tasmanian regulations do not set any limit on NFR if chlorophyll "a" and phaeophytin levels are greater than 100µg/L, however if this situation was to change then tertiary treatment would be necessary.

Figure 4.18 Chlorophyll "a" v. NFR for the lagoon systems studied



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CHAPTER 5

REGULATORY DISCHARGE REQUIREMENTS

Contents

Tables

5.1 New Legislation

The Tasmanian Government has through legislation over the last two years put in place a new Resource Management and Planning System. This new legislation is an integrated package which includes:

(a) *State Policies and Projects Act 1993.(SPPA)*
(b) *Land Use Planning and Approvals Act 1993.(LUPAA)*
(c) *Resource Management and Planning Appeal Tribunal Act 1993.(RMPATA)*
(d) *Environmental Management and Pollution Control Act 1994 (EMPCA).*

The aim is to provide a more integrated approach to the assessment and regulation of issues associated with land or water based developments, and the use of resources.

The legislation has the common objective of sustainable development in Tasmania (see Schedule 1 of EMPCA). A consolidated approval process replaces the separate planning permit and the environmental licensing systems.

The *State Policies and Projects Act 1993* has three key functions:

- to outline the State objectives and provide for Tasmanian Sustainable Development (TSD) Policies on key environmental management issues, such as water quality management and waste management (thus supporting EMPCA).

- to outline the criteria for activities or projects deemed to be of State Significance (i.e. Level 3 Activities under EMPCA). All projects need to be assessed under Environmental Impact Assessment (EIA) principles as described by Section 74 of EMPCA.

- to provide for State of the Environment Reporting.

The *Land Use Planning and Approvals Act 1993* provides, as the title suggests, for land use permits and planning matters, while the *Resource Management and Planning Appeal Tribunal Act 1993* provides a mechanism of appeal against any planning and/or environmental issues.

The *Environmental Management and Pollution Control Act 1994* replaced the *Environment Protection Act 1973* (EPA) as the principal Tasmanian environmental legislation.

5.2 Environmental Management and Pollution Control Act 1994 (EMPCA)

EMPCA, which was passed through Parliament in 1994 and proclaimed in January 1996 provides the framework for environmental management in the State.

A central principle of EMPCA is to prevent significant "environmental harm". Environmental harm is defined as "any harm or potential harm to the environment and includes environmental nuisance". It is an offence to cause material or serious environmental harm or an environmental nuisance.

EMPCA has a number of features which make it a more effective tool than the EPA (1973) for achieving good environmental management. These include:

- it is part of an integrated package of environment and planning legislation;
- a series of objectives which make the legislation outcome oriented;
- capacity to move away from end-of-pipe standards to ambient objectives;
- no Ministerial Exemptions, but the Board of Environmental Management and Pollution Control can require non-complying activities to implement an Environmental Improvement Program which has a three year time frame and associated fee structure;
- incentives for better performance, e.g. Environmental Agreements, Performance Guarantees and Financial Assurances;
- more flexibility and more effective enforcement tools, e.g. Environmental Protection Notices, Environmental Infringement Notices, Environmental Audits;
- environmental impact assessment principles.

Environmentally relevant activities under EMPCA are classed as Level 1, 2 or 3 activities. Under EMPCA, a Level 1 is an activity/development/use requiring a permit under LUPAA and which may cause environmental harm. These are smaller scale activities, which are below those production thresholds for Level 2 activities detailed in Schedule 2 of EMPCA, and are assessed and regulated by Local Government unless called in by the Board of Environmental Management and Pollution Control. Activities which are considered more likely to be significant sources of pollution are defined as "Level 2 Activities", e.g. sewage treatment works with a design average dry weather flows greater than 100 kL/d. Most Level 2 activities require a permit under LUPAA. These activities will be subjected to an integrated assessment process based on the Local Government planning system. There will be no separate environmental licence. The assessment of environmental issues will be carried out by the Board of Environmental Management and Pollution Control, and the Board's decision and conditions are implemented by the Local Government body through the development permit. The Board's assessment is in accordance with the Environmental Impact Assessment principles. Level 2 activities will continue to be regulated by the Environment Tasmania. Level 3 activities are projects deemed to be of State significance according to the SPPA (1993). Assessment of such proposals is performed by the Sustainable Development Advisory Committee under Environmental Impact Assessment (EIA) principles.

EMPCA provides a new range of environment enforcement mechanisms. With the concept of "environmental harm" caused by pollution, the extent of harm adversely affecting the environment can be broadly defined. "Serious" or "material" harm depends on the impact and scale of the effect, while "environmental nuisance" is defined as the emission of a pollutant that unreasonably interferes with, or is likely to unreasonably interfere with, a person's enjoyment of the environment.

Management and enforcement tools include the following:

- *Environmental Agreements* recognise performance beyond that required by the Act and provide for incentives such as fee reductions.

- *Environmental Audits* are either mandatory or voluntary and include compliance, management, waste and contaminated sites auditing. Mandatory auditing is called for by the Board and voluntary auditing is, as the name suggests, undertaken voluntarily by an operator. Information generated in a voluntary audit formally acknowledged by the Board before the commencement of the audit, cannot be used to prosecute for any contravention of the Act. However, the operator can certainly be requested to address, through improvement programs, any deficiencies that may result in environmental harm.

- *Emergency Authorisations* are used by the Director in exceptional circumstances to authorise an act or emission that would otherwise constitute a contravention of the Act.

- *Financial Assurances* for an activity are bonds supported by a guarantee, a security or a specified pecuniary sum approved by the Board in situations where the risk of environmental harm is high, there is a history of failure to comply, or the potential nature of the contravention is serious.

- *Environmental Improvement Programs (EIPs)* specify a need to reduce environmental harm in order to comply. A program to achieve compliance is documented, and completed within a 3 year time frame. A fee structure exists for this type of program.

- *Environmental Protection Notice (EPN)* require specified actions to be undertaken. Costs incurred by the Department are recoverable from the person on whom the notice was served, and EPNs are publicly registered. EPNs are used to prevent or remedy environmental harm, give effect to State Policy, and to vary the environmental conditions attached to a permit.

- *Civil Enforcement Proceedings* can be initiated by the Director, Council, or any person demonstrating a proper interest. It is handled by the Resource Management and Planning Appeals Tribunal.

- *Environmental Infringement Notices* are issued when an authorised officer is satisfied that a prescribed offence has been committed. Penalty fines are associated with this notice.

5.3 Tasmanian Sustainable Development Policy On Water Quality Management

5.3.1 Present regulations

The same regulations that supported the EPA 1973 have been carried over to the new EMPCA legislation as an interim arrangement. The EPA Regulations (1974) on water pollution and atmospheric pollution (for example) will be progressively replaced with Tasmanian Sustainable Development Policies, which have statutory force. These are often referred to as State Policies. The Environment Protection (Water Pollution) Regulations 1974 will be replaced with a State Policy on Water Quality Management in 1996/97. The relevant emission limits specified in these regulations will remain as the limits attached to permits in force for point sources of pollution until the regulatory authority reviews the emission limits in accordance with the policy detailed below.

The principal function of the present Water Pollution Regulation (1974) is to define standards for the (point source) emission of pollutants to water bodies (see Table 5.1).

Table 5.1 Sewage treatment plant discharge limits under the Environment Protection (Water Pollution) Regulations 1974.

		BOD_5 (mg/L)	NFR (mg/L)	Faecal Coliforms & (orgs/100mL)	Oil & Grease (mg/L)
Receiving Waters					
(1) Inland Waters					
	(i)	20	30	200	10
	(ii)	40	60	200	10
(2) Bays & Estuarine Waters					
	(i)	20	30	1000	10
	(ii)	40	60	1000	10
(3) Coastal Waters		N/A	200	N/A	N/A

Notes:

(a) For "Inland Waters" and "Bays and Estuarine Waters", (i) represents where the flow of the receiving waters is less than 50 times the rate of the flow of the emission, and (ii) is where the flow of the receiving waters is greater than 50 times the rate of the flow of the emission.

(b) Oxygen content in the receiving waters shall not be reduced to below 50% saturation.

(c) The effluent should be visually free of oil and grease.

(d) Where algae are visually detectable (i.e. the chlorophyll "a" and phaeophytin is greater than 100 μg/L) in the effluent there is no limit on the NFR level and the BOD level is increased to 40 mg/L for inland waters, bays, and estuaries.

Table 5.1 specifies the levels of pollutants permitted to be discharged into inland waters, bays and estuarine waters, and coastal waters from wastewater treatment plants (WWTP) under the 1974 Regulations. If the pollutants in an emission complies with the standards, the emission is not deemed to be pollution. If the standards are exceeded then the emissions are a breach of the Act. The standards for most pollutants are more stringent for inland waters, recognising more limited dispersion and dilution. In contrast there are very few specified standards for the emissions to coastal waters.

5.3.2 State Policies

The currently draft State Policy on Water Quality Management follows from the model administrative structure outlined in the National Water Quality Management Strategy. This strategy was developed by Australian and New Zealand Environment and Conservation Council (ANZECC) and the Agricultural and Resource Management Council of Australia and New Zealand (ARMCANZ). A series of documents were produced by these organisations, including the Australian Water Quality Guidelines for Fresh and Marine Waters.

The vision statement of the State Policy on Water Quality Management is:

> "To achieve the sustainable use of Tasmania's surface water and ground water resources by protecting and enhancing their quality while maintaining economic and social development".

The major points of the State Policy are as follows:

- establishing a framework for setting water quality objectives;
- setting out strategies for managing point source and diffuse sources of pollution;
- specifying policies and programs to achieve and maintain water quality objectives;
- defining the responsibility for and approaches to water quality monitoring to determine whether the desired environmental values are being protected. Refer to Table 5.2 for a list of Protected Environmental Values identified in the Australian Water Quality Guidelines;
- establishing integrated catchment management as a means of evaluating whether objectives are being achieved, and implementing corrective action where necessary.

Table 5.2 Protected Environmental Values

The water quality objectives for surface and ground water in Tasmania will be determined on a case by case basis by evaluating which of the following protected environmental values and uses need to be protected.

A. Protection of Aquatic Ecosystems
  (i) Pristine or nearly pristine ecosystems
  (ii) Modified ecosystems
    (a) from which edible fish, crustacean and shellfish are harvested
    (b) from which edible fish, crustacean and shellfish are not harvested

B. Recreational Water Quality and Aesthetics
  (i) Primary contact
  (ii) Secondary contact
  (ii) Aesthetics only

C. Raw Water for Drinking Water Supply
  (i) Subject to coarse screening only
  (ii) Subject to coarse screening plus disinfection

D. Agricultural Water Uses
  (i) Irrigation
  (ii) Stock watering

E. Industrial Water Supply
  The specific industry type for which the water is to be used must be specified to identify appropriate guidelines.

5.3.3 Water Quality Objectives

The environmental values to be protected in specific waterways and segments of the coast (and ground waters) are proposed in the draft State Policy to be set through a consultative process with the involvement of all stakeholders as part of an Integrated Catchment Management process in Tasmania. The Environmental Management and Pollution Control Board will determine the water quality guidelines for the key indicators nominated to protect the environmental values. The lowest level of each indicator to protect these values will be required.

If the environmental values to be protected have not formally been set, the Board or a planning authority, with the Board's approval, may nominate the environmental values to be protected and use these to set the interim water quality objectives. Before the values are nominated all reasonable measures to consult with the parties that have interest in the water body will be taken.

5.3.4 Diffuse Sources of Pollution

The key principle to manage diffuse sources of pollution from such areas as agricultural run-off, urban run-off, road construction, maintenance and drainage is to use Best Practice Environmental Management. This is defined by EMPCA as:

> "the management of the activity to achieve an ongoing minimisation of the activity's environmental harm through cost-effective measures assessed against the current international and national standards applicable to the activity"

5.3.5 Point Sources of Pollution

The key principles of the management of point sources of pollution are the maintenance of recognised environmental quality objectives and waste minimisation.

The management of point sources of pollution will require the following:

A) Regulatory authorities (the Board of EMPCA for Level 2 activities and planning authorities for Level 1 activities) are to ensure that pollution from potential sources of pollution is controlled by limiting the rate and quantity of pollutants which may be discharged to surface or ground water in accordance with the provisions of the Policy.

B) Emission limits for discharges of waters to surface and ground water from point sources of pollution are set in accordance with the following key principles:

   (i) The discharge limit must be designed to ensure that recognised water quality objectives of the receiving waters are met and maintained.

   (ii) Waste discharges to the environment should be reduced to the maximum extent that is reasonable and practicable and provides potential environmental benefit.

C) Regulatory authorities are to ensure that all reasonable and practicable measures are taken to reduce waste discharges to surface and ground water in accordance with the following hierarchy of waste management (arranged in decreasing order of desirability).

   (i) Waste Avoidance
   (ii) Recycling/Reclamation
   (iii) Waste Reuse
   (iv) Waste treatment to reduce potentially degrading impacts
   (v) Waste Disposal

D) A permit to discharge waste water to inland waters will only be issued by the relevant regulatory authority if it is satisfied that:

   (i) the pollutant load of the waste water has been minimised in accordance with the hierarchy of waste management, described above;
   (ii) connection to a sewerage system with appropriate treatment capacity is not reasonable or practical;
   (iii) land application of the wastewater in an environmentally acceptable and sustainable manner is not reasonable or practicable;
   (iv) any unavoidable discharge will not prejudice the achievement of the recognised water quality objectives of the receiving waters.

The discharge of wastes to surface waters will continue to be the only option for some situations. In accordance with point (B) above, regulatory authorities will need to set an emission limit which will ensure that the recognised water quality objectives for the receiving water are maintained. To do this it is necessary to:

(a) identify the protected environmental values of the water body receiving the discharge;

(b) identify components in the emission with the potential to degrade water quality;

(c) establish quantitative water quality objectives for these components (i.e. the maximum tolerable levels of these components in the ambient environment consistent with protecting the nominated environmental values) in accordance with guidelines based on human health criteria (recommendations in the most recent NH&MRC guidelines, unless otherwise recommended by the Director of Public Health) and the Australian Water Quality Guidelines (or more recent authorised guidelines) supplemented with site specific information, where practicable;

(d) establish the background levels of these components in the ambient environment; and

(e) calculate the amount of waste which may be discharged without causing the levels of critical components to exceed the levels required by the water quality objectives.

(d) and (e) will require considerable information and resources, and would be difficult to implement in full when dealing with discharges from small-scale activities. In such cases, a simple approach, in which emission limit could be based on those achievable using "acceptable modern technology" or “best practice environmental management” to reduce wastes as far as is reasonable and practicable. This technology must provide sufficient treatment such that the emission levels do not threaten the protected environmental values of the receiving water.

To protect some environmental values the maximum tolerable level of some contaminants is very low. In many cases these levels could not be achieved in wastewater using presently available acceptable modern technology. However, usually it should be possible to achieve the required levels within a short distance from the end of the discharge pipe following dilution and dispersion of the effluent. This would be referred to as the mixing zone. Clearly, the size of a permitted mixing zone must be both practical and consistent with community needs.

The EMPC Board may designate a mixing zone in respect of a discharge from level 1 and level 2 activities to surface waters. Water quality objectives must be achieved at the boundary of the mixing zone.

Mixing zones must be in accordance with the following:

(a) The location and size of the mixing zone, and the indicators to which it applies, must be clearly defined in a permit;

(b) The mixing zone, either alone, or in combination with other mixing zones

  (i) should not occupy a significant proportion of the receiving waters designated for a given protected environmental value;
  (ii) should not detract from the values and uses of the surrounding waters.

(c) The mixing zone should not generally be specified in waters which:

- (i) receive significant and regular use for primary contact recreation;
- (ii) are recognised as of significant values for spawning or nursery areas;
- (iii) are close to areas used for aquaculture;
- (iv) are close to potable water supply intakes;
- (v) within waters where pristine aquatic ecosystems are to be protected.

(d) Mixing zones must not create a significant barrier to the migration of fish or other aquatic organisms.

(e) Mixing zones are to be set with regard to low flow conditions.

Within mixing zones the emission limits are to be set such that the emission does not cause:

(a) objectionable odours which may affect the use of the surrounding environment;

(b) objectionable discolouration at the surface of the mixing zone;

(c) visible floating foam, oil, grease, scum, litter, or other objectionable matter;

(d) mortality of fish;

(e) fish or other aquatic organisms used for human consumption to become unacceptable for such as determined by the Tasmanian health standards.

5.4 References and Bibliography

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CHAPTER 6

DESIGN PRINCIPLES FOR TASMANIAN SEWAGE TREATMENT LAGOONS

Contents

Appendices

Tables

Figures

6.1 Introduction

Sewage lagoons in Tasmania have primarily been designed to reduce organic and inorganic pollutants and pathogens to acceptable levels prior to discharge into receiving waters. From Chapter 3 it is apparent that the majority of the lagoon systems in the State produce an effluent with levels of biochemical oxygen demand (BOD_5) and/or faecal coliforms that do not comply with the Tasmanian water pollution regulations. All lagoon systems with only 2 cells (i.e., 63% of the lagoon systems in the State) do not comply.

Designers may face one of two situations: the upgrading of a system (to meet statutory regulations or handle a greater load) and the design of a new system. The retro-fitting or upgrading of a system enables the designer to use past performance and influent load data for each specific system. However, the design of a new system in many circumstances requires the use of theoretical design values derived by researchers locally or world-wide.

Bacterial reduction within lagoons using modifications of Marais' work (1974), as outlined in this chapter, was found to be effective for design purposes.

Removal of BOD_5 was modelled using many design equations ranging from areal loading rate through to linear and empirical equations, and various kinetic models. As a result of the study the design method recommended for the primary cell is the areal loading rate method. This method is easy to apply as it only requires the determination of a few variables. When the values of Marais & Shaw 1961 and 1964 were averaged, however, the estimated value was within approximately 10% of the observed value for BOD in the primary effluent of non-aerated lagoon cells.

An additional requirement imposed on lagoon systems may be the removal or reduction of nitrogen, phosphorus and algae levels. In this study, observations of the nutrient levels from the effects of passive mixing in lagoons were made and the effects of a sand filter and lime dosing (within a secondary lagoon) were trialed. It was found that some reduction of these performance variables was achievable using these avenues of optimisation.

Finally, the designer should not underestimate the significance of hydraulic design. Lagoon system size, shape, baffle placement, baffle design and environmental conditions all influence performance. The study indicated the fluid bodies for different lagoons differed in how frequently they mixed as a result of that wind influence and fluid temperature changes. Stratification during warmer months could develop within 24 hours, resulting in anoxic and anaerobic conditions in the bottom layer, short circuiting and reduced hydraulic residence times. The monitoring of this stratification during the warmer months is of use in predicting likely lagoon performance in terms of bacterial and BOD_5 removal.

Integral to any lagoon design is actual data on influent flow, load and bacterial concentration. However, if these data cannot be obtained then they should be conservatively estimated from data acquired from similar design situations. Values for performance variables used in design work have been derived and summarised in this chapter.

6.2 Measurement And Estimation Of Load

Determination of pollutant loads to a lagoon requires the measurement of influent (raw sewage) flow rate and associated pollutant concentrations. However, it is also possible to estimate the load from the population size using empirically determined flow and pollution generation per "equivalent person". As part of the study, loadings estimated by this method were compared to actual measurements to ascertain the accuracy of the EP calculated loadings.

6.2.1 The methods for load determination

Sewage is normally delivered to the lagoon site via gravity or pumped flow. Gravity flow is considered to be part to full pipe flow, while pumped flow via a rising main usually results in full pipe flow.

In the study, gravity flow was measured by adapting a magnetic flow meter, which is normally installed into permanent pipe work, into a portable unit. This inexpensive unit, once calibrated, recorded the instantaneous and total flow to within ±5%.

Pumped flow was measured using hour-run meters at the pumping station and calibrating the flow in the rising main by carrying out a drop and fill test in the pump well. The total flow is found by multiplying the number of pump hours run by the duty of the pump. This method was estimated to be accurate to approximately 5 %.
To measure pollutant concentrations, a flow proportional composite sample was taken over a 24 hour period. Samples were analysed for a range of pollutants.

The organic and suspended solid loads are determined by the product of the flow data and corresponding concentrations of the organic material as shown in the equation below. The organic material is normally determined using biochemical oxygen demand (BOD_5); chemical oxygen demand (COD) and non-filtrable residue (NFR, also known as suspended solids). It is important that the load is calculated for each individual day the sample and corresponding flow measurements are taken. The average of these daily loads is used as the load on the lagoon, not the product of the average flow and average pollutant concentration.

For any day, the load is calculated by:

$$\text{Load (kg/d)} = \frac{\text{Flow (kL/d) x Pollutant Concentration (mg/L)}}{1000}$$

For detailed discussion of the methods refer Section 6.10 and to Chapter 8, Section 8.3.2.

6.2.2 Influent hydraulic load determination

Table 6.1 compares the measured flow and the EP estimated flow for the six Tasmanian lagoon systems investigated in the study. Table 6.2 compares average dry and wet weather flow.

Table 6.1 Comparison of the estimated and measured sewage influent flows

LOCATION	Estimated EP	[1]Estimated ADWF	[2]Measured ADWF			[3]Measured Average Flow			Measured ADWF per EP per day		Measured Average Flow/EP.d.	
	No.	kL/Day	n	kL/d	SE	n	kL/d	SE	(L/EP.d)	SE	(L/EP.d)	SE
Beaconsfield	1010	242	7	196	16	11	241	34	194	16	239	34
Bothwell	350	84	9	147	9	14	157	13	420	25	439	38
Campbell Town*	800	192	6	377	10	6	377	10	471	12	471	12
Dover	430	103	5	129	3	10	155	17	300	8	360	39
Fingal	350	84	5	90	7	6	89	6	258	21	253	18
Turners Beach	1500	360	6	347	6	15	367	11	235	3	245	7
Average									313		343	

Note:
- ADWF is the average dry weather flow.
- n and SE are the number of samples and standard error respectively.
- [1] 240 litres/person/day was applied as it is the most often used flow/EP in Tasmania; refer to Table 3.1.
- [2] Measured dry weather flows in the SLPIP were defined as flows measured when less than or equal to a total of 2mm of rain fell over the day prior and day during the flow measurements.
- [3] Measured average flows is the average flow measurements for the system.
- * No wet weather event was recorded for this site.

Table 6.2 Comparison of the average dry weather flow to average wet weather flow

LOCATION	ADWF			n	Average Rainfall		AWWF		Ratio
	n	kL/d	SE		mm	SE	kL/d	SE	AWWF:ADWF
Beaconsfield	7	196	16	4	12	2	321	79	1.64
Bothwell	9	147	9	5	4	1	175	35	1.19
Campbell Town	6	377	10	0	na	na	na	na	na
Dover	5	129	3	5	10	9	181	31	1.40
Fingal	5	90	7	1	8	na	81	na	≅ 1
Turners Beach	6	347	6	9	10	2	380	14	1.10

Note:
1) AWWF is the average wet weather flow over the period the individual plants were studied. The sample distribution for each plant may not be sufficient to determine the absolute AWWF for the plant.
2) Fingal AWWF is less than ADWF due to insufficient WWF samples being recorded and standard error.

There are some significant differences between the estimated and measured dry weather flows in Table 6.1. The data for the Campbell Town plant, with an estimated flow of 192 kL/d and a measured flow of 377 kL/d, demonstrates there would potentially be approximately a 100% error if only estimated data were used. This is a significant design error. The range of measured dry weather flow per EP at the sites studied was 194 - 471 L/EP.d. These values generally deviated considerably from the values of 220 - 250 L/EP.d mainly used for lagoon design in the State: refer to Chapter 3 Table 3.2 for more details on Tasmanian lagoon systems.

The flow data emphasise the need to quantify actual site specific flow using flow equipment when possible or, if not possible, recognising that when using estimates the margin of error may be significant, as illustrated in the above table for most of the sites studied.

The high measured dry weather flows per EP for several of the lagoon systems indicates high water usage by some communities. This is a major factor when considering lagoon performance and design. There is a possibility that high flows are due to high permanent infiltration into sewers in situations where sewers are below the water table and ingress is through damaged pipework. Both Bothwell and Campbell Town staff indicated permanent infiltration was unlikely. However, high flow may also be due to commercial activities and public facilities present in the towns. As the "EP" concept is a residential one, considering industries in EP terms does impose limitations.

Comparing the measured average dry weather flows with average wet weather flows and the average flows indicates that higher flows are probably also associated with stormwater infiltration and/or illicit connections. Lagoon systems can generally accommodate the increase in flow due to wet weather without significant loss in performance due to their long hydraulic residence time. However, design should allow for these occasions when infiltration is significant and frequent.

Regular flow measurements allow the integrity of existing sewer lines, and community water management practices to be evaluated and are integral to the ultimate design and optimisation of sewage lagoon systems. The quantification and reduction of flows has obvious economic benefits such as reduced pumping costs and potential postponement or minimisation of costly lagoon upgrades.

Errors in flow determinations have major ramifications for the performance of lagoon systems, e.g. bacterial die off performance. For example, if the EP estimates of flow in Table 6.1 were used for the Bothwell and Campbell Town lagoon systems, the actual bacterial count for a three cell system would probably be 6 fold higher than the predicted. These figures are derived from the Marais Equation which this study has found to be, with a few modifications, the most appropriate method of predicting bacterial die off.

6.2.3 Influent organic load determination

The measured average loads (determined by flow-proportional sampling) on the lagoon systems included in the study are briefly summarised in Table 6.3. Examples of concentrations used to estimate loads are detailed in Tables 6.6 and 6.7.

Table 6.3 BOD5 and NFR loads for the sewage lagoon influent flows.

LOCATION	EP	No. of Samples	Average BOD5		Average NFR		Measured Av. BOD_5 /EP Load		Measured Av. NFR /EP Load	
			(mg/L)	SE	(mg/L)	SE	(g/EP.d)	SE	(g/EP.d)	SE
Beaconsfield	1010	11	232	26	204	23	47	2.	40	2
Bothwell	350	14	165	13	181	23	73	7	81	11
Campbell Town	800	6	185	7	180	10	83	5	82	6
Dover	430	7	95	16	90	16	31	5	30	5
Fingal	350	6	303	19	397	59	77	7	110	21
Turners Beach	1500	15	207	8	221	6	51	1	55	2
Average			198		212		60		66	

Note:
1) The measured load per equivalent person is calculated from the product of the flow and the corresponding pollutant concentration and divided by the EP for the individual days sampled. The averages for each site are summarised above.
2) SE is the standard error

It can be seen from Table 6.3 that there are significant differences between sites for the average influent BOD5 and NFR concentrations and the average load per EP. From this table and Table 6.1, it is apparent that lagoon systems have their own particular flow, concentration and pollutant load per EP characteristics.

Since the hydraulic load influences the lagoon design for bacterial reduction (refer to section 6.4) and the organic load determines the primary lagoon size (refer to section 6.5), it follows from Tables 6.1 and 6.3 that the use of EP data and estimates alone are not ideal and may potentially lead to inaccuracies in the design of sewage lagoon systems. This may lead to poor performance, such as primary cell overloading and poor bacterial kill rate (due to reduced hydraulic residence time) or the uneconomic over design of lagoons.

It is recommended that hydraulic load and pollutant concentrations be measured, if possible, before the designing of new lagoons and upgrades. Measurements should also be taken from existing lagoons in order to properly assess design or upgrade options, should it be necessary. For lagoon systems where these measurements are not possible, the data generated in this document may be used as a guide for load estimation of similar sites proposed or existing. Refer to Tables 6.1 and 6.2 for this information and Chapter 3 (Table 3.2) to identify the study lagoon system most similar to that being designed.

6.2.4 Bacterial and nutrient levels in influent

As with BOD and NFR concentrations, average bacterial and nutrient concentrations were found to vary for each site studied.

The average bacterial number in the influent at the six Tasmania lagoon sites studied, shown in Table 6.4, was not dissimilar to the generally accepted level of 10^7 - 10^8 total coliforms/100mL (Metcalf and Eddy 1991). Nevertheless, influent bacterial levels at respective sites should still be measured or estimated if designing a primary cell. Refer to Chapter 4 Section 4.7 for discussion on E.coli k values (bacterial die off rates) for primary and secondary cells and Section 6.4 for design details for bacterial reduction. When redesigning the secondary cells it is recommended that bacterial levels firstly be measured at the outlet of the primary and the following cells. This information reduces the errors due to the variability in primary cell bacterial die off performance as detailed in Section 4.7.

Table 6.4 Influent bacteria levels at the lagoon sites in Tasmania

Location	Number of Samples	Total Coliform		E.coli	
		No/100ml	SE	No/100ml	SE
Beaconsfield	12	4.5×10^7	0.14	1.0×10^7	0.16
Bothwell	13	5.4×10^7	0.16	3.6×10^6	0.14
Campbell Town	11	2.3×10^7	0.14	8.1×10^6	0.14
Dover	6	1.3×10^7	0.44	2.9×10^6	0.44
Fingal	7	3.0×10^7	0.09	4.6×10^6	0.09
Turners Beach	16	8.1×10^7	0.18	8.2×10^6	0.14
Average		4.1×10^7		6.2×10^6	

Note: Standard Error (SE) expressed in log.

Average influent nutrient concentrations for different plants were also found to be variable; refer to Table 6.5. It is therefore advised that influent nutrient concentrations are also measured if a plant is to be designed for nutrient removal.

Table 6.5 Influent nutrient levels at lagoon sites in Tasmania

Location	Number of Samples	Total N		Number of Samples	NH4/NH3-N		Total P		PO4-P	
		(mg/L)	SE		(mg/L)	SE	(mg/L)	SE	(mg/L)	SE
Beaconsfield	10	44.4	3.9	12	25.4	2.7	8.1	0.7	4.9	0.6
Bothwell	9	27.9	1.0	15	14.2	1.2	6.1	0.4	3.4	0.3
Campbell Town	11	36.1	1.2	11	22.4	1.3	6.4	0.3	4.1	0.2
Dover	5	24.4	2.7	7	13.3	2.4	4.2	0.6	2.5	0.4
Fingal	7	52.4	3.2	7	31.8	2.0	12.9	0.9	7.2	0.4
Turners Beach	6	41.6	2.6	16	22.6	1.6	8.1	0.4	4.1	0.3
Average		37.8			21.6		7.6		4.4	

Note: Nitrite and nitrate are at minor levels in the influent ranging from 0.03-0.07 mg/L and 0.1-0.6 mg/L respectively

6.2.5 Estimation figures for design

Although the variability and site specific nature of hydraulic and influent pollutant concentrations makes accurate estimation of loadings to lagoons difficult it is sometimes necessary to use estimated flow/EP and influent load/EP. The data for the raw sewage measured from the six lagoons studied is summarised in Table 6.6.

Table 6.6 Raw sewage data from the six Tasmanian lagoons studied

Raw Influent Parameters	Average	Median	SE	Min	Max
ADWF per EP per day (L/EP.d)	313	279	45	235	471
BOD_5 concentration (mg/L)	198	196	28	95	303
BOD_5 Load per EP (g/EP.d)	60	62	8	31	83
NFR concentration (mg/L)	212	193	41	90	397
NFR Load per EP (g/EP.d)	66	68	12	30	110
E.coli enumeration ($x10^6$ orgs/100ml)	6.2	6.4	1.2	2.9	10.0
N-NH3/N-NH4 concentration (mg/L)	21.6	22.5	2.9	13.3	31.8
Total Nitrogen (mg/L)	37.8	38.85	4.3	24.4	52.4
P-PO4 concentration (mg/L)	4.4	4.1	0.7	2.5	7.2
Total Phosphorus (mg/L)	7.6	7.3	1.0	4.2	12.9

Table 6.7 is a brief summary from some of the literature values around the world.

Table 6.7 Typical composition of untreated domestic sewage

Contaminants	Unit	Typical Median Concentration	Source
BOD_5	mg/L	220	Metcalf and Eddy 1991
BOD_5	mg/L	316	Tebbutt 1971
BOD_5	mg/L	250	Bliss et al 1981
BOD_5	mg/L	200	Locke 1991
BOD_5	mg/L	196	SLPIP 1996
NFR	mg/L	220	Metcalf and Eddy 1991
NFR	mg/L	371	Tebbutt 1971
NFR	mg/L	250	Bliss et al 1981
NFR	mg/L	200	Locke 1991
NFR	mg/L	193	SLPIP 1996
Total Coliform	orgs/100ml	10^7-10^8	Metcalf and Eddy 1991
Total Coliform	orgs/100ml	10^7	SLPIP 1996
E.coli	orgs/100ml	10^6-10^7	SLPIP 1996
ADWF per EP per day	L/EP.d	270	DELM 1992
ADWF per EP per day	L/EP.d	279	SLPIP 1996

The median BOD_5 and NFR concentrations in the influent determined in the study are not that dissimilar from the values determined by Metcalf and Eddy (1991) and Locke (1991). However, from Tables 6.1, 6.3, 6.4 and 6.5 it can be seen that the range of measured ADWF and performance variable concentrations determined for the studied lagoon treatment sites is considerable.

6.3 Site Assessment

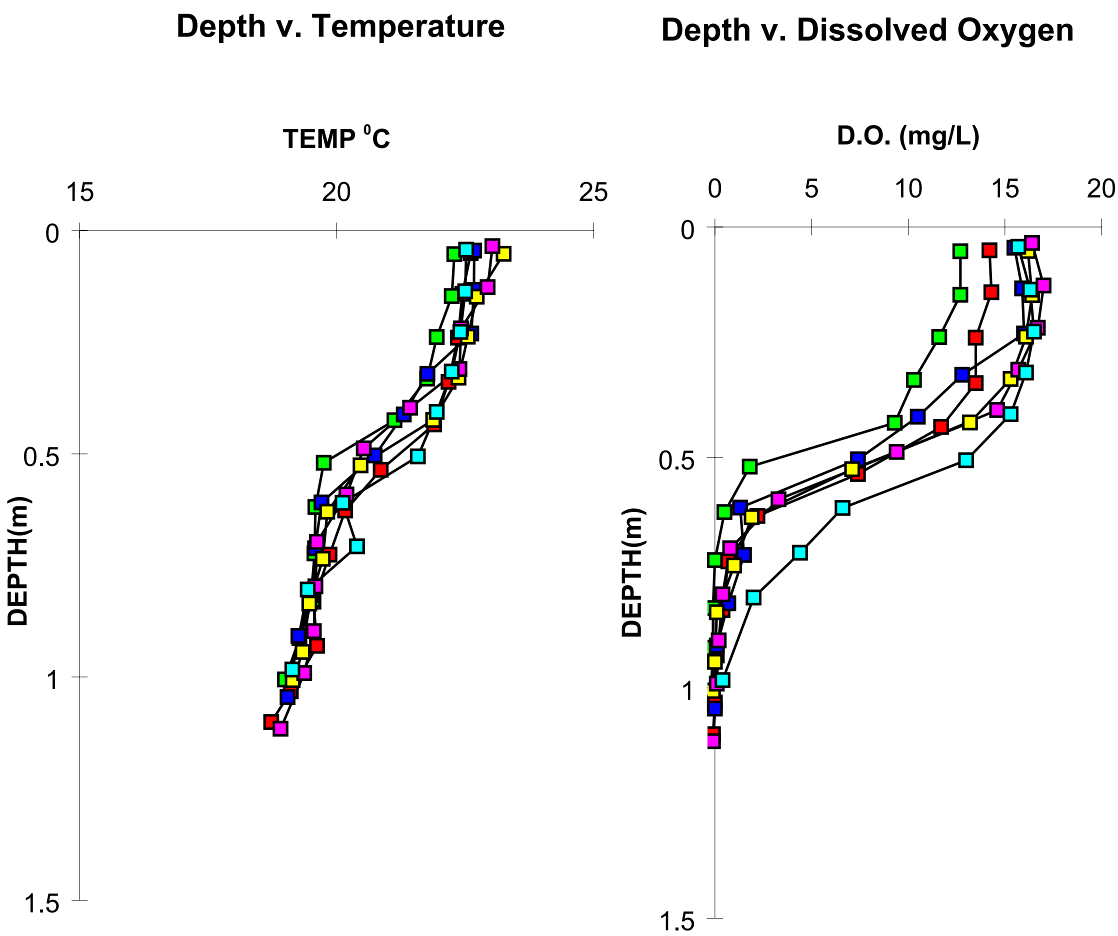
As mentioned in Chapter 4, local environmental factors influence most performance criteria for lagoon systems. The assessment of these factors is vital for optimising lagoon design. These factors are discussed as follows:

6.3.1 Stratification

During the study, monthly lagoon profiles were determined using a YEO-KAL 606 submersible data logger that measured dissolved oxygen(DO), pH, temperature, conductivity, turbidity and depth. It was found lagoons that stratified frequently had poor bacterial and BOD_5 reduction, but increased levels of nutrient removal. The results of this work are discussed in more detail in Chapter 4.

The major factors affecting the development and destruction of stratification are wind action and temperature changes. Stratification usually occurs during the warmer months, on relatively calm days, from around November to May, and can occur throughout the State. In lagoons it is usually caused by a less dense warm water layer sitting on top of colder, denser water. Stratification may be detected using a DO meter and/or thermometer where there is a change, normally abrupt, in DO and/or temperature at about 30 to 50cm depth. The study has shown the temperature differential between the layers may be up to 5 ^{O}C. Sometimes stratification in the DO concentrations occurs without a temperature change, indicating that oxygen depletion in the bottom layer can occur purely due to lack of mixing in the lagoon cell. A typical temperature and DO profile of a stratified lagoon cell is shown in Figure 6.1. These data were derived from profiling the secondary cell of the Beaconsfield lagoon system during a day in Summer.

Figure 6.1. The DO and corresponding temperature profile of a stratified sewage lagoon.



As wind influences most performance variables, the designer should attempt to predict from wind data the frequency of lagoon stratification. It was found in the study that cells separated by baffles or walls (i.e., with two way flow between cells), where stratification occurred more than 30 percent of the time, had a 60 percent reduction in the bacterial die off performance compared to well mixed lagoons, where stratification occurred less than 10 percent of the time. On these figures, a stratifying cell would need to be approximately 2.5 times its original size to achieve the same performance of a comparable well mixed cell. Also, for a lagoon cell separated by a levee, i.e., regardless of whether it is 30% stratified or not, there is a 140 percent increase in bacterial die off performance compared to a lagoon system separated by baffles that is rarely stratified (<10% of the time).

If a lagoon does (or is likely to) stratify frequently then the designer should allow for the decreased performance in organic removal and bacterial kill rate by increasing the size of the cells or incorporating more cells in the design. If this is not possible then changes are necessary to enhance mixing and the transfer of oxygen through improving wind action by removing obstructions around the perimeter of the lagoon system or by using aerators (or mixers). However, as mentioned, stratification improves the rate of nutrient removal and this may be a desired component of the treatment process (refer to Section 6.6).

6.3.2 Wind assessment

Due to Tasmania's location in the Roaring Forties many areas are generally windy. This potentially enables many lagoons to be well mixed reactors, but the extent of this wind-assisted mixing is really dependent on the local topography, surrounding vegetation, and lagoon design. If a lagoon is to be built the designer has to use their qualitative judgement to assess the likely amount of wind action on the lagoon. The exposure of the lagoons is probably the best indicator. Bureau of Meteorology data should be accessed and local knowledge is also helpful in assessing whether a site is windy. In difficult areas, where the wind is affected significantly by topography and/or vegetation, an anemometer could be installed on site for some baseline data on wind speed and direction.

The surrounding trees should be removed, particularly those upwind of the prevailing wind direction in warmer months when stratification is likely. Banks and baffles should be as low as practical to reduce wind interference and maximise wind speed across the lagoon surface.

If a lagoon has already been built the following approach can be used to assess whether a lagoon is well mixed:

- Local knowledge, such as, operators who visit the lagoons regularly. This is very subjective but invaluable.

- Measuring the DO and temperature at different depths with a DO meter during the warmer months (November to April) will detect if a lagoon is stratified. If this is done frequently it will give the designer a very good idea of the amount of mixing that a specific lagoon system may experience. Data collected in the study suggests that DO monitoring of one or two sites within each lagoon will give the designer a good indication of the stratification status of the whole system.

- If profiling data is limited it is still possible to generate information on likely lagoon stratification events for those periods which lack data. The profiling data measured is correlated with the local wind conditions obtained from a nearby weather station. The wind velocities and directions are related to measured incidences of DO and temperature stratification within the lagoon. It is then possible to predict the likely approximate percentage occurrence of stratification from the average wind speed for each day. However, the accuracy of the prediction is dependent on the distribution of information near the critical point where stratification is destroyed. Refer to Chapter 4, Section 4.6, for more details on how wind data may be used to indicate the frequency and timing of mixed and stratified systems.

6.3.3 Monitoring the present performance

If a lagoon system is going to be modified then monitoring of the present performance before modification is recommended. The BOD load entering the primary cell and the extent of BOD removal in this cell needs to be established through a flow proportional sample at the inlet and a grab sample at its outlet. For bacterial removal within a system and the determination of bacterial die off rates (E. coli k values) it is suggested that bacterial samples are taken after the primary cell and at the final outlet. The factors affecting bacterial performance, such as hydraulic residence time, type of cell separation and stratification, as discussed later in this chapter, should be taken into account when calculating the appropriate E.coli k values for the modified system.

6.4 Designing For Bacterial Disinfection

Aside from the advantages of well mixed over stratified systems for bacterial die-off rates, bacterial reduction is also affected by cell number, type of separation between cells and cell layout.

The study confirmed the applicability of the Marais equation in determining likely bacterial kill rate with certain lagoon configurations; refer to 6.4.4 for detail. However, it was necessary for the decay constant or bacterial kill rate (k) to be altered for various cell configuration and stratified states.

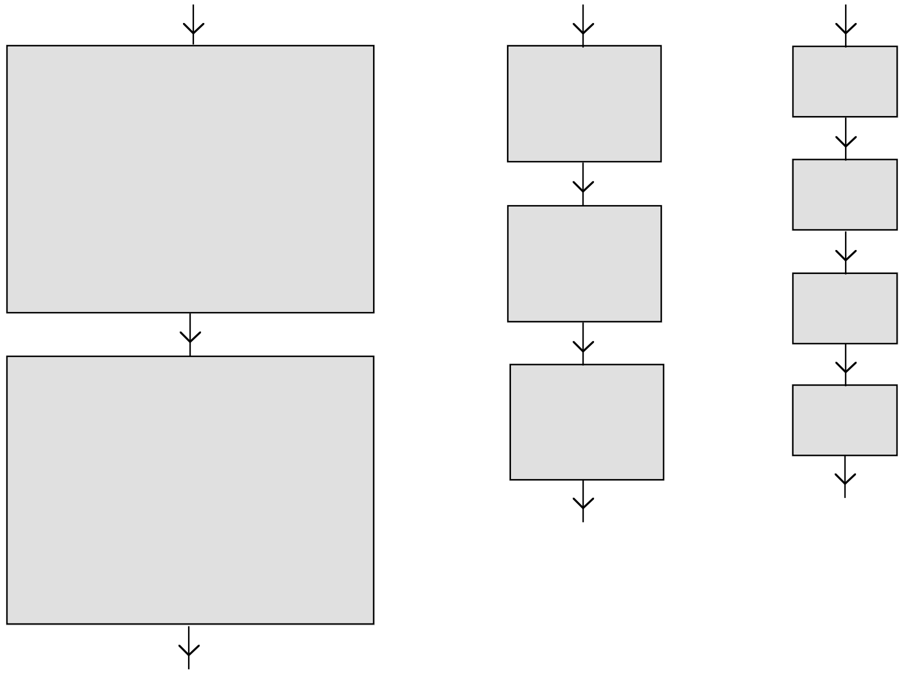
The lagoon configuration issues are discussed below, before dealing with Marais theory and the modified decay constant values.

6.4.1 Number of lagoon cells

From Chapter 3 it is evident that a majority (63%) of the permitted lagoon systems in Tasmania consist of two cells. None of these systems comply with the discharge requirements in the Environment Protection (Water Pollution) Regulations 1974 for faecal coliforms (i.e., E. coli ≈ 90% faecal coliforms). The E.coli die off rates measured in this study suggest that for an effluent to comply with the discharge limit into inland waters of 200 E.coli per 100 mL, a 200 days (or more) theoretical hydraulic residence time (HRT) in a two cell system would be necessary. None of these lagoon systems have been designed for this length of HRT.

It is well known that, for a given total lagoon volume, cells in series perform better than a single cell for bacterial removal (Marais 1966). Using equation 6.1 in Section 6.4.4, the following Figure 6.2 illustrates how important it is to review the desirable numbers of cells to cost effectively reduce the bacterial number. Note that this figure is a scale comparison of lagoon cells needed to theoretically produce the same effluent quality, using Marais equation. However, care should be taken to avoid making individual cells too small (ie less than 5-10 days HRT), otherwise short circuiting is likely and wind mixing can be significantly reduced due to the small fetch.

Figure 6.2 Scale comparison of lagoon cells (that will theoretically result in the same bacterial kill rate).



6.4.2 Separation of Lagoon Cells

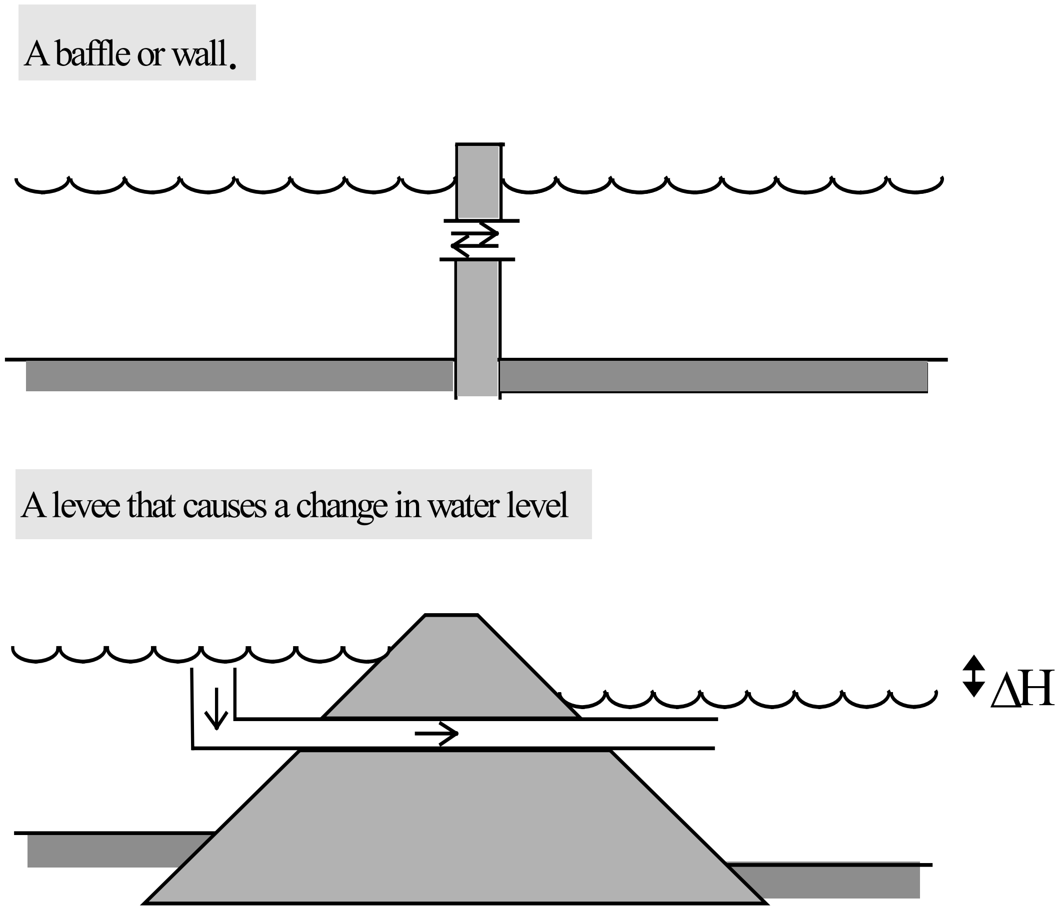
The study lagoon systems have cells in series separated by a baffle or wall, and/or levees. With baffles and walls free exchange of fluid occurs and the water level is unchanged between cells, while cells separated by levees permit only “one way” directional flow from one cell to the next and have changes in water levels: refer to Figures 6.3. The data collected, summarised in Tables 4.5 and Figure 4.10 in Chapter 4 (Section 4.7), showed that those cells in series separated by a baffle or wall had significantly poorer overall bacterial die off rates than cells separated by levees. The severity of the poor performance is significant, but can be altered by the general integrity of the baffle.

It was observed during the study that fluid exchanges in both ways through baffles and wall outlets. This exchange results in increased overall bacterial load on the latter cell; refer to Chapter 4, Section 4.7.2.

Despite cell separation by baffles or walls not being as effective as levees, they are still useful to the designer. A cell divided up with baffles will achieve a better overall performance than if it had not been. Their inclusion in an existing lagoon may be an economic alternative to improve the performance of the lagoon system. Figure 4.11 of Chapter 4 illustrates that a slight improvement in the bacterial die off rate may be achieved using prefabricated partial synthetic baffles within cells. It is considered that they may inhibit the occurrence of short-circuiting.

On a well made baffle that does not lack integrity, the construction of a one way valve to achieve the performance of a levee may be possible.

Figure 6.3 Baffle (or wall) and levee design



6.4.3 Cell layout

Comparative analysis of the six lagoon systems in the study has indicated that shape, type of cell separation, inlet (and outlet) all influence to varying degrees the performance of lagoon cells in terms of hydraulics and associated bacterial (and organic and nutrient) removal.

Extensive dye tracing studies using an *in situ* spectro-fluorometer revealed a homogeneity of fluid within cells on windy and calm days where stratification had not occurred. The tracer studies indicated that on non-stratified occasions influent disperses quickly throughout the cells. The cell performance, however, can be optimised with the following provisions.

It is recommended that the primary lagoon cell be designed with a regular square or slightly elongated rectangular shape to help create a complete mixed reactor. The subsequent cells should be rectangular in so producing a cell with characteristics between a complete mixed and plug-flow situation. It has been suggested that the length to width ratio should be 3:1 or greater (USA EPA Manual 1983). Optimally the corners of the lagoon cells should be curved. With irregular shaped cells hydraulic residence problems such as dead zones and short-circuiting can develop.

The study showed that only a slight improvement in the die off rate of E.coli in the primary cell was observed with multiple sub-surface side inlets compared to that of a single surface inlet approaching the centre of the lagoon; refer to Chapter 4 Table 4.5. However, it is important to note that the Campbell

Town lagoon system, where this trial was conducted, was well exposed to wind action and on the majority of occasions fully mixed.

To optimise a lagoon to account for those occasions when stratification exists the inlets should be sub-surface by approximately 0.6 m to avoid the stratification zones as detected in the study, and 5 to 10 m from the edge of the lagoon to prevent localised odour problems and difficulties in dispersing the influent. The path from the inlet to the outlet should be maximised. Wind under certain circumstances can lead to short circuiting within cells so alignment of the inlet to the outlet to the prevailing wind axis is not recommended.

An exposed windy site will ensure the lagoons act as intermittently mixed cells which will enhance the bacterial kill. The reader is referred to Chapter 4 for further discussion on the effects of wind and other factors which affect treatment.

Study observations indicate that the outlets should be designed to prevent the discharge of surface scum and minimise the transfer of algae by using a protective baffle. The depth the baffle needs to be from the surface is uncertain, however it is suggested that it should be approximately 300 mm deep. It is suggested that as 0.6 m/s (NSW Public Works) is the minimum velocity used to keep organic matter suspended in the sewer the protective baffle could be designed as a broad crested weir with a velocity less than 0.6 m/s.

The depth of the lagoons in the study ranged from 0.9 to 1.8 m. A conservative recommendation is that the depth should be a minimum of 1 metre for effective stabilisation, odour management, control of emergent weed growth and the effects of algal growth.

The inlet or whole area of the primary lagoon could be made deeper (1.5 to 2.5 m) to allow potentially for sludge settling and enhanced digestion. The SDL data and dye tracing did show some accumulation near the inlet but also showed sludge build up at other areas throughout the primary cells.

Free board (including a wave wall) width should be at least 0.5 m. The width will be dependent on the size of the lagoon and its exposure to the wind for wave development.

6.4.4 Predicting bacterial kill using the Marais Equation

Marais (1974) presented a consolidated theory of faecal bacteria death kinetics which covered previous work by Marais and Shaw (1961), Marais (1966), and Marais (1970). This theory incorporates the effect of temperature on specific death rate of bacteria.

The theory is based on the following assumptions:

1. Mixing in the lagoon is instantaneous and complete;
2. Reduction of bacteria takes place according to first order kinetics (Chick's law).

ie. $$\frac{dN}{dt} = -kN$$

Where N = concentration of faecal bacteria (number per unit volume) in the cell (No./100ml)
t = hydraulic retention time, days
k = decay constant in (day^{-1}) units.

Equations can be derived for a single lagoon or a series of lagoons. The general equation for a single lagoon is:

$$N = \frac{N_o}{kt + 1} \tag{6.1}$$

where N_o = concentration of faecal bacteria in influent (No./100ml).

and $t = \frac{V}{Q}$

where V = Cell Volume (m^3)
Q = Flow through the cell (kL/d).

An expression for series operation of i cells which assumes k is constant for each cell is given by:

$$N_n = \frac{N_o}{\prod_{i=1}^{n} (kt_i + 1)} \tag{6.2}$$

where t_i = influent retention time for each individual cell, days (where t can be calculated from V_i and Q_i as above)
Π= product of all $(kt_i + 1)$ where i =1,2,3.....
i.e., for two cells, $(kt_1 + 1)x(kt_2 + 1)$
N_n= concentration of faecal bacteria (number per unit volume) in the nth cell (No./100ml)

Experimental data show the effect of temperature on k value for E.coli to be:

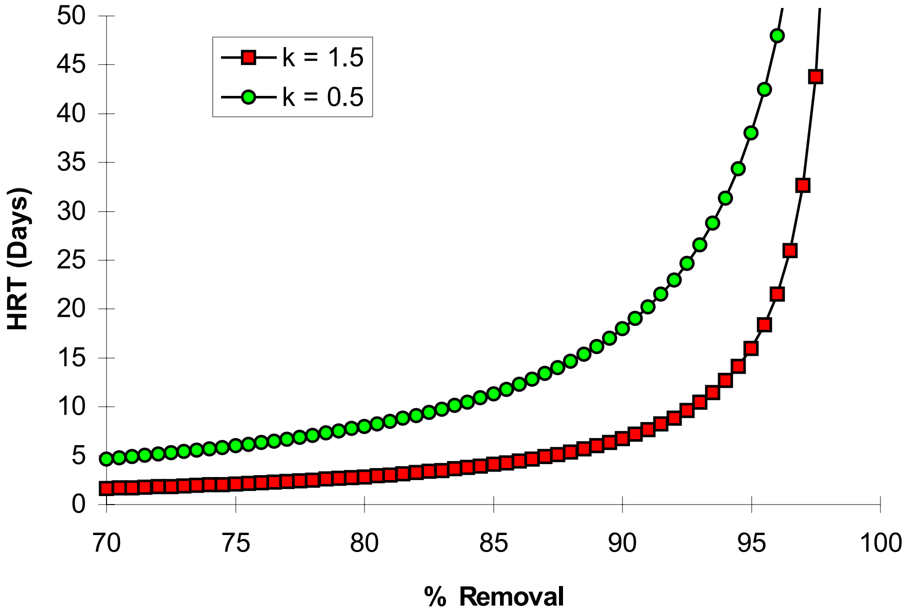
$$k_t = 2.6\,(1.19)^{T-20} \tag{6.3}$$

where T = temperature (^{0}C)

There is good correlation in the literature for the data falling within the temperature range 2^0C-21^0C. The k value is very temperature sensitive. At low temperatures k drops sharply due to the slow down of the metabolic rate of the bacteria. During periods of high temperatures and poor mixing, stratification develops, reducing the kill rate. This is a result of both the anaerobic condition in the bottom layer of the lagoon and the promotion of short circuiting within a particular water layer in the lagoon. The ideal condition for high bacterial reduction would be a well mixed lagoon accompanied by warm temperatures.

In Figure 6.4, using equation 6.1, a theoretical plot of percentage bacterial kill versus the hydraulic retention time for single cells with different k values is derived. It can be seen that where the curves become steeper the required hydraulic retention time becomes significant for very little gain in removal rate. It is often more cost effective to build more lagoons in series rather than larger lagoons to achieve the same theoretical bacterial kill rate, as illustrated by Figure 6.2. Another consideration that the designer should be aware of (especially for systems with low hydraulic load) is that small lagoons have minimal fetch for the influence of wind action which may decrease the k value. For practical purposes, Marais recommended each lagoon is designed for 90% reduction, however, for adequate wind action the minimum HRT (and fetch) may result in greater than a 90 % removal scenario.

Figure 6.4 % Bacterial removal v. theoretical hydraulic retention time



For further information on Marais equation in relation to the design of lagoons for bacterial reduction refer to Marais (1974).

6.4.5 Marais Equation as used in this study

The results from this study show that the Marais equation only predicted well the bacterial (E.coli) kill rate performance for the Campbell Town site. This site had a lagoon system with individual cells separated by levees (i.e. "one way" directional flow) and was well exposed to wind action. All other sites studied were not predicted well by Marais equation. These sites had lagoon cells separated by baffles or walls with two-way exchange of fluid between cells which we believe compromised the bacterial die off rate of the connected cells: this is despite the good integrity of the baffles in many cases. Some sites also stratified regularly due to impaired wind action which further reduced the level of bacterial disinfection in the lagoon system.

It was found, however, that if the E.coli k value was adjusted according to the extent of stratification and the type of cell separation (baffle or levee), a reasonable estimate of bacterial kill rate performance could be made for each cell using the Marais equation. In order to do this, the Marais equation had to be altered to allow for each cell to have its own E.coli k value. The primary cell, however, appeared to not experience reduced bacterial kill rate performance due to stratification.

For a given cell n in series, the modified Marais equation is given by:

$$N_n = \frac{N_o}{\prod_{i=1}^{n} (k_i t_i + 1)} \qquad (6.4)$$

i.e., for a three cell system

$$N_n = \frac{N_o}{(k_1t_1 + 1)\,(k_2t_2 + 1)\,(k_3t_3 + 1)}$$

Where $t_1, t_2, t_3,$ = HRT for cells 1, 2, and 3 respectively,
$k_1, k_2, k_3,$ = E.coli k constant for cells 1, 2, and 3 respectively

To use equation 6.4 the E.coli k values can be estimated for each individual cell using the following information, obtained from the analysis of data collected in this study, as discussed in Chapter 4.

For the primary cell
$k \approx 1.5$, although it was as high as 3 in some lagoon systems.

For secondary cells
$k \approx 1.5$, for cells with high wind action that rarely stratify
$k \approx 1.0$, for cells with moderate wind action that stratify occasionally
$k \approx 0.5$, for cells with low wind action that stratify frequently.

If the cell is preceded by a baffle the expected k value is further reduced by approximately 60%, i.e., by multiplying the k value above by 0.4.

Temperature differences between lagoons did not have to be taken into account because of the similarity of the temperature ranges for lagoons in Tasmania

6.5 Design For Organic Reduction

6.5.1 Organic loading on the primary

The study lagoons showed a relatively small range of BOD_5 loadings on the first cell with most lagoons being loaded between 38 kg/Ha/day and 75 kg/ha/day. The loadings on the first cells are presented in Table 6.8

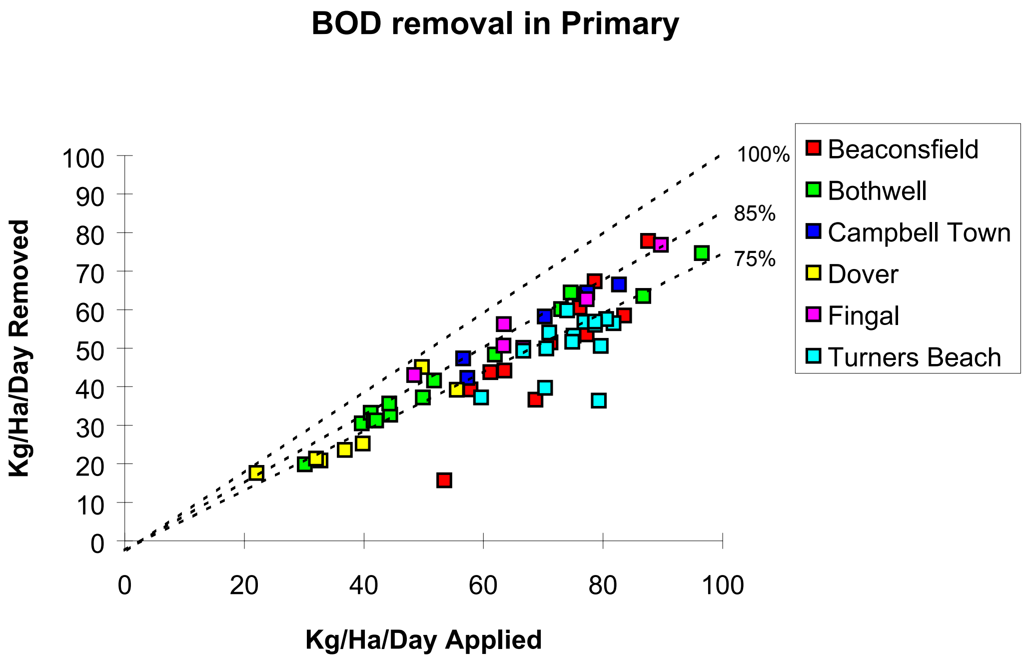
Table 6.8 BOD_5 and NFR loading on the primary lagoons in the study

LOCATION	Primary Area	Av. Flow	Av. BOD		Av. NFR		Av. BOD_5 Load		Av. NFR Load.	
	(ha)	(kL/d)	(mg/L)	SE	(mg/L)	SE	(kg/ha.d)	SE	(kg/ha.d)	SE
Beaconsfield	0.67	241	232	26	204	23	71	3.3	61	2.3
Bothwell	0.46	157	165	13	181	23	56	5.3	62	8.4
Campbell Town	0.94	377	185	7	180	10	70	4.5	70	5.4
Dover	0.38	155	95	16	90	16	38	4.3	37	4.7
Fingal	0.39	89	303	19	397	59	69	5.7	99	18.6
Turners Beach	1.04	367	207	8	221	6	75	1.6	79	2.6

Table 6.8 shows that the Campbell Town, Turners Beach, Fingal and Beaconsfield lagoon systems were loaded with BOD on average at approximately 70 kg BOD/ha.d. The first two sites are exposed windy sites which showed slight stratification while both Fingal and Beaconsfield are sheltered sites which showed significant stratification events throughout the warmer months. Of these four sites, odour problems were reported at Fingal and Turners Beach, which were brought under control quickly by the use of aspirators.

Figure 6.5 shows for the primary cell the relationship between BOD load applied and load removed. For 75-85% removal in the primary lagoon, a loading rate between 40 and 80 kg/ha.d is required. This plot shows that a well designed and operated primary sewage lagoon should achieve a minimum of 75% BOD removal. Performance at higher levels is very site specific and requires more intensive management of the site.

Figure 6.5 kg BOD/ha.d load applied to the primary v. kg BOD/ha.d removed in the primary.



It is therefore suggested that lagoons are loaded conservatively at or less than 60 kg/ha.day but not less than 40 kg/ha.d, as the percent removal would then generally drop to less than 75% across the lagoon.

There is a large amount of literature on the appropriate BOD_5 loading for the first cell of a lagoon system. The ideal BOD_5 loading for different sites is highly temperature dependent. Refer to Appendix A for areal loading rates quoted by various authors.

6.5.2 Appropriateness of published models for lagoons

The use of kinetic design equations as a loading method were also considered as an alternative to the BOD loading method. Using the data obtained from the study, five design equations were appraised for their accuracy in predicting the requirements and/or performance expectations of primary lagoons. The design equations considered are detailed in Appendix B and were those of Gloyna (1971), Marais and Shaw (1961 and 1964), Thirumurthi (1974), and Wehner-Wilhelm (1956). Table 6.9 summarises for this study how the measured values compare with the values derived using these design equations.

Tables 6.9 Comparison of observed and theoretical loading rate determination methods

THEORETICAL AND OBSERVED	Beaconsfield	Bothwell	Campbell Town	Dover	Fingal	Turners Beach
Gloyna's theoretical lagoon volume (m^3) required	16321	10000	24965	5580	10103	22176
Observed primary lagoon volume (kL)	6413	5880	13803	4536	3562	9833
% Difference of Gloyna value to actual primary volume	155	70	81	23	184	126
Observed total lagoon system volume (kL)	17500	7610	22600	7270	5280	21600
% Difference of Gloyna value to actual total lagoon system volume	-7	31	11	-23	91	3
Observed average BOD_5 in primary lagoon effluent (mg/L)	58	34	46	28	47	65
Observed average BOD_5 level in the lagoon system effluent (mg/L)	50	36	37	29	49	30
Average BOD_5 removal across the lagoon system (%)	79	78	80	70	84	87
Marais & Shaw (1961) theoretical primary effluent BOD_5 concentration (Le) in mg/L	48	33	35	22	55	43
Observed average BOD_5 in primary lagoon effluent (mg/L)	58	34	46	28	47	65
% Difference of Marais & Shaw (1961) value to actual BOD_5 in primary lagoon effluent	-17	-5	-24	-20	18	-34
Marais & Shaw (1964) theoretical primary effluent BOD_5 concentration (Le) in mg/L	74	43	47	31	80	72
% Difference of Marais & Shaw(1964) value to actual BOD_5 in primary lagoon effluent	28	27	2	11	69	11
Average of Marais & Shaw 1961 and 1964 theoretical primary effluent BOD_5 (mg/L) levels	61	38	41	27	68	57
% Difference of averaged Marais & Shaw(1961) & (1964) value to actual primary lagoon BOD_5 level	5	11	-11	-4	44	-12
Thirumurthi (1974) theoretical primary effluent BOD_5 concentration (Ce) in mg/L	82	57	50	50	75	70
Observed average BOD_5 in primary lagoon effluent (mg/L)	58	34	46	28	47	65
% Difference of Thirumurthi (1974) value to measured primary lagoon BOD_5 level	41	68	9	80	60	8
Wehner-Wilhelm Equation (1956) theoretical primary BOD_5 concentration (Ce) in mg/L	59	39	41	30	63	52
Observed average BOD_5 in primary lagoon effluent (mg/L)	58	34	46	28	47	65
% Difference of Wehner-Wilhelm (1956) value to measured primary lagoon BOD_5 level	1	16	-11	6	35	-20

Note:

- Gloyna (m^3) is the theoretical volume required in a facultative lagoon to achieve 85 - 95% BOD removal.
- Le and Ce (mg/L) represent the predicted BOD level in the primary lagoon effluent.
- BOD is the total organic load
- Fingal primary cell is aerated
- Dover primary cell is partially aerated intermittently and does short circuit to the outfall through a wooden baffle.

Gloyna uses his equation to determine the required volume of a facultative lagoon to treat the wastewater and achieve an estimated BOD_5 removal efficiency of 85-95% (Gloyna, 1971). From the data it is apparent that the equation would uneconomically over design the first cell (by a factor of about 2 in most cases). However, considering the performance data for Beaconsfield, Campbell Town and Turners Beach, the Gloyna equation could be used as a guide to achieve 80-85% BOD_5 reduction over the whole system.. Data for Fingal and Bothwell, however, do indicate that the Gloyna equation can over estimate the required volume for 80-85% removal for the entire system by 91% and 31% respectively. Admittedly, the Fingal lagoon system is aerated in the primary which would reduce the required volume to less than that predicted by Gloyna.

Using the Marais - Shaw (1961) equation the % difference of the predicted BOD value to the observed value ranged from -34% to 18%. With the exception of one value the predicted BOD_5 levels were lower than the observed value at the outlet of the primary. The predicted values were generally within 30% of the observed values. With the Marais - Shaw (1964) equation the predicted BOD values, however, were higher than the observed values, generally within the range from 2 to 28% (with one exception).

The Thirumurthi equation (1974), used for plug flow lagoon types, generally did not predict the observed primary effluent BOD_5 levels. With the exception of two sites, Campbell Town and Turners Beach, the use of this equation resulted in a 41 to 80% difference from the observed BOD effluent value.

The use of the Wehner-Wilhelm equation (1956), which is quite complicated, generally predicted the observed primary effluent value of BOD to within ± 20%. The BOD effluent value from the Fingal primary cell was not predicted well, however, with a 35% error. This predicted value was higher than the observed level possibly due to the influence of the mechanical aeration in this cell. The equation uses a dimensionless dispersion number which ranges from D=0 for plug flow and D=∞ for complete mixed flow. A value of D=0.1 was found to fit the data best.

The conclusion of this comparison of design models is that their accuracy for estimating the primary effluent BOD concentration is relatively poor and inconsistent. However, if the values determined by the Marais & Shaw 1961 and 1964 models are averaged then an estimate within the range of -4 to 11% to the observed value for BOD in the primary effluent can be achieved. This only applies to non-aerated primary lagoons.

6.5.3 Organic reduction over lagoon systems

The study predominantly used BOD_5 and NFR to monitor the organic removal throughout lagoons. The main factor that affected the final BOD_5 concentrations was found to be the amount of anaerobic and anoxic activity in the final cells. Data collected using the submersible data logger suggest that lagoons that are frequently stratified, due to lack of wind action, such as Beaconsfield and Fingal, have about 40% higher BOD_5 levels than lagoons that are well mixed such as Turners Beach, Campbell Town and Bothwell. The NFR levels in stratified lagoons frequently are only about 8% higher than for well mixed lagoons which is probably not significant.

COD analysis also carried out in the last 6 months of the study show that samples with higher BOD_5 do not seem to have elevated COD. Refer to Chapter 4, Section 4.8 for more detail.

The reason for higher BOD_5 results for these lagoons is most probably nitrification processes consuming oxygen during the BOD_5 test as mentioned in the "Standard Methods For Water And Wastewater Analysis". The lagoons that have high BOD_5 values are the same lagoons that produced elevated nitrite and nitrate, which supports this suggestion. This means that the higher BOD_5 figures may not be indicative of higher organic loads on the receiving environment. Thus when testing lagoon systems it is suggested that the BOD test should be nitrification inhibited to avoid the confounding factor of nitrification-driven use of O_2. In fact, more emphasis should be placed on COD level as a determination of lagoon performance.

For maximum BOD_5 removal lagoons should be exposed to as much wind action as possible to ensure full mixing. If a lagoon site is to be selected it should be placed in an area where the local topography allows for greatest wind action. There is a strong possibility that in areas of low wind action, mechanical mixers may be used to minimise anaerobic and anoxic activity and achieve similar performance to lagoons with good wind action.

6.6 Nutrient Removal

Table 6.10 summaries the nutrient levels being discharged and the removal characteristics of the lagoon systems studied.

Table 6.10 Summary of nutrient removal with lagoon systems studied

PARAMETER			Total N	N-NO2	N-NO3	N-NH3	Total P	P-PO4
BEACONSFIELD	No. of Samples		11	19	19	19	19	19
	Concentration	Average (mg/L)	15.0	0.57	0.65	2.79	5.30	2.87
		Std Error	0.9	0.24	0.26	0.70	0.35	0.38
	% Removal	Average	66.2	na	na	89.0	29.7	41.4
BOTHWELL	No. of Samples		4	11	11	11	11	11
	Concentration	Average (mg/L)	14.4	0.09	0.18	9.79	6.14	3.66
		Std Error	1.1	0.02	0.01	1.31	0.31	0.38
	% Removal	Average	48.3	na	na	31.2	7.1	-6.7
CAMPBELL TOWN	No. of Samples		12	12	12	12	12	12
	Concentration	Average (mg/L)	16.2	0.21	0.57	7.63	5.41	3.97
		Std Error	1.4	0.08	0.06	2.25	0.26	0.29
	% Removal	Average	55.1	na	na	66.0	15.5	4.2
DOVER	No. of Samples		8	11	11	11	11	11
	Concentration	Average (mg/L)	12.8	0.85	1.01	5.72	3.01	1.78
		Std Error	1.1	0.33	0.10	1.54	0.45	0.36
	% Removal	Average	47.8	na	na	57.0	30.2	29.4
FINGAL	No. of Samples		12	18	18	18	18	17
	Concentration	Average (mg/L)	26.4	0.09	0.18	10.22	7.94	4.66
		Std Error	1.5	0.02	0.01	1.65	0.44	0.43
	% Removal	Average	49.7	na	na	67.9	39.9	38.2
TURNERS BEACH	No. of Samples		7	16	16	16	17	16
	Concentration	Average (mg/L)	17.8	0.04	0.3	6.54	7.72	4.00
		Std Error	2.3	0.01	0.02	1.09	0.72	0.64
	% Removal	Average	57.3	na	na	71.1	8.2	3.6

Note: The abbreviation “na” represents not applicable as the % removal of NO_x is excluded on the basis it is unlikely to be significant in a final discharge as these parameters are intermediates in the nitrification and denitrification process.

Lagoons with high wind action which do not stratify regularly are poor at removing total phosphorus in comparison to lagoons with poor wind action and which stratify. Lagoons which lack wind action such as Dover, Beaconsfield and Fingal removed a range of 30 to 40 % of total phosphorus. Lagoons such as Turners Beach, Bothwell and Campbell Town with high wind action averaged 10 % removal of total phosphorus.

The nitrogen removal influences are less clearly defined. Total nitrogen removal in the lagoon systems studied is generally similar, ranging from 47.8 to 57.3 %, whether the lagoons are strongly stratified or fully mixed. There is a marginal improvement of 66.2% removal in the Beaconsfield lagoon system, which is considered to alternate regularly between a fully mixed and stratified state during warmer months. The processes involved in nitrogen removal are likely to operate at varying degrees at each of these sites, however it is difficult to establish which of these processes are more significant. These processes would include the wind stripping of ammonia when the pH is high, nitrification and denitrification in the aerobic and anaerobic layers respectively, uptake of nitrogen by algae, the settling of organic and inorganic solids and adsorption onto soils.

Some researchers believe the main mechanism for nutrient removal is ammonia stripping. In summer, ammonia removal was significant in all the systems in the study, definitely due to high algae growth and high lagoon pH. This shifts the reaction in the equilibrium equation,

$$NH_4^+ + H_2O \leftrightarrow NH_3 + H_3O^+$$

to the right. In windy conditions this mechanism may be major. However, in systems that alternate between stratified and fully mixed states it is considered that nitrogen removal is also through nitrification and denitrification processes.

In winter, algae growth is low, the pH falls, and little ammonia is stripped. In some of this study's lagoon systems, however, nitrification clearly still occurred during this period, when the pH falls to neutral conditions. This occurrence was primarily in the Campbell Town and Beaconsfield lagoon systems. This suggests an alternative nitrogen removal mechanism such as nitrification/denitrification.

An interesting area for further research would be to control wind action and mixing in a lagoon system using a combination of wave baffles and mixers to see if the process involved in this nutrient removal can be optimised.

If regular stratification of the lagoon is a necessary outcome for optimal nutrient reduction, as discussed in this section, then placement of trees or construction of risen banks in certain areas around the lagoon would encourage the alternation of mixed and stratified states through the selective interference of wind action. However, under stratified situations the lagoon's hydraulic retention time and/or the number of cells would have to be increased to achieve the desired bacterial kill rate.

6.7 Sand Filtration

A pilot sand filter with a medium hydraulic load was trialed at the Campbell Town site for four months. It was installed to polish the secondary treated effluent by reducing the residual organics, nutrients, suspended solids (mainly algae) and the bacterial level.

From data supplied by Ti Tree Bend Laboratory, Launceston City Council, covering two years of monthly sampling at a lagoon site it was evident that the removal of solids through filtering resulted in major % reduction in BOD and indicator bacteria, and a reasonable reduction in some nutrients. Table 6.11 summarises the data.

Table 6.11 % Removal of BOD, nutrients and bacteria associated with solids removal from primary and secondary lagoon effluent

PARAMETERS	Primary Effluent % Removal	Secondary Effluent % Removal
BOD	84	70
Total Nitrogen	65	36
Total Phosphorus	41	23
Total Coliforms	96	96
E.coli	90	91

Note: Glass fibre filters normally used for NFR determination were used.

A sand filter bed consisting of one chamber was installed separate from the lagoon system. Lagoon effluent was introduced at the top of the chamber onto a splash plate to assist distribution onto the sand. The effective size of the sand for the filter bed was 0.18 mm with a uniformity coefficient of 1.62. The hydraulic load was $1m^3/m^2$.d which was run for two days and then allowed to dry. Surface crust usually ranging from 6 to 10 mm was removed every second or third run with each run occurring approximately once per five days. The wasted sand could be dried in a shallow bed to remove some pollutants and washed, where the leachate could go into an evaporation basin, to reduce any residual pollutants.

The performance of the sand filter, detailed in Table 6.12, illustrates how very effective intermittent, medium flow sand filters are. The suspended material (mainly algae), organic load, bacterial number, ammonia and chlorophyll “a” were significantly reduced at greater than 90%. The total nitrogen and phosphorus levels were reduced by 51% and 33% respectively.

Table 6.12 Summary of the performance of the pilot sand filter

PARAMETER	INFLUENT		EFFLUENT		REMOVAL
	Average	S.D.	Average	S.D.	%
pH	8.1	1.3	6.1	0.1	na
BOD (mg/l)	60	16	5	2	92
COD (mg/l)	186	26	64	13	66
NFR (mg/L)	78	16	4	2	95
Total Coliforms (No./100ml)	2.60E+06	na	2429	na	99.9
E. coli (No./100ml)	4.04E+03	na	54	na	98.7
F.streptococcus	7.04E+03	na	76	na	98.9
N-NO2	292	341	5	3	na
N-NO3	189	227	3923	1966	na
N-NH3	2315	1202	50	43	98
P-PO4	2935	1281	2941	681	0
Total Phosphorus	5367	685	3594	407	33
Total Nitrogen	11933	2539	5819	2056	51
Chlorophyll a	701	321	53	40	92

The trapping of particulate material in the surface layers probably accounts for a majority of the reductions in the variables as indicated in Table 6.11. The data suggests that the biological activity within the sand filter does further reduce the nutrients through nitrification/denitrification. The regular removal of the surface accumulated material probably optimises the removal of nutrients by minimising their release from decomposing surface material.

A full scale plant would be required to determine labour cost, and operational and maintenance difficulties. A series of sand filter cells, possibly 3 or 4, would be used alternately for two day run periods, to ensure a wet-dry period for each filter and allow for cleaning to take place once per week. At the hydraulic load of 100kL/d flow an area of 10x10 m would be required for each cell. Also, because of their performance, it may be worthwhile considering sand filters after both the primary and secondary lagoons to further reduce the nutrient levels. Better nutrient removal may also be achieved if the surface layer of the sand is removed after every two day run.

6.8 Lime Addition

The application of hydrated lime (i.e. calcium hydroxide or limil) to the secondary lagoon was trialed at the Bothwell site for seven months. The trial was conducted to reduce the residual organics, nutrients, suspended solids (mainly algae) and the bacterial level.

Table 6.13 Hydrated lime dosing in the secondary lagoon cell

Parameter	No treatment with $Ca(OH)_2$			Addition of $Ca(OH)_2$			Reduction
	Average	n	S.D.	Average	n	S.D.	%
pH	8.69	11	0.85	9.46	7	0.27	na
BOD (mg/l)	36	11	16	23	7	6	35
COD (mg/l)	na	na	na	131	6	31	na
NFR (mg/L)	71	11	41	44	7	20	38
Total Coliforms (No./100ml)	1.23E+05	10	na	1.28E+05	7	na	-4
E. coli (No./100ml)	1.02E+04	10	na	3.46E+03	7	na	66
F.strep (No./100ml)	2.72E+03	10	na	2.04E+03	7	na	25
N-NO2 (mg/L)	0.1	11	0.07	0.2	7	0.15	-126
N-NO3 (mg/L)	0.2	11	0.12	0.4	7	0.05	-142
N-NH3 (mg/L)	9.8	11	4.3	7.6	7	5.8	22
P-PO4 (mg/L)	3.7	11	1.3	1.5	7	0.57	58
Total Phosphorus (mg/L)	6.1	11	1.0	3.9	7	0.78	36
Total Nitrogen (mg/L)	14.4	4	2.2	15.4	7	3.3	-7
Chlorophyll a (ug/L)	653	11	485	378	7	318	42

The hydrated lime was broadcast every 2 - 3 days over the secondary lagoon at a dosing rate sufficient to maintain the final lagoon pH at 9.5 or higher. The dosage required increased over the winter months. Table 6.13 illustrates that there was a reasonable improvement in reduction, with hydrated lime addition, of BOD, NFR, chlorophyll "a", Faecal streptococcus, ammonia-nitrogen and total phosphorus ranging from 22 - 36 %. E.coli and ortho-phosphate levels were more significantly reduced at 66 % and 58 % respectively, while total coliforms, nitrate-nitrogen, nitrite-nitrogen, and total nitrogen levels were increased to varying extents.

The addition of hydrated lime to the lagoons does not present any medium term option of upgrade, but may be of use when requiring a short term reduction in the BOD, NFR, indicator organisms, chlorophyll "a" and some nutrients.

6.9 Recommended Design Approach

Step 1

The influent hydraulic load and associated concentrations of pollutants to determine pollutant load should be measured, if possible, before the design of new lagoons or the upgrading of existing lagoons. The flow data in this chapter emphasise the need to quantify actual site specific flow using flow equipment when possible or, if not possible, recognising that when using estimates the margin of error may be significant.

The samples for pollutant load determination must be flow-proportional composite samples. For lagoon systems where these measurements are not possible, the data generated in this document may be used as a guide for load estimation of similar sites proposed or existing. Although the variability and site specific nature of hydraulic and influent pollutant concentrations makes accurate estimation of loadings to lagoons difficult it is sometimes necessary to use estimated flow/EP and influent load/EP.

Step 2

Monitor environmental factors that affect lagoon performance. Wind action and temperature changes are the major factors which affect the development and destruction of stratification in lagoons. It was found that lagoons that stratified frequently had poor bacterial and BOD_5 reduction, but increased levels of nutrient removal. Stratification, which usually occurs during the warmer months on relatively calm days, can develop within 24 hours. It results in anoxic and anaerobic conditions in the bottom layer, short circuiting and reduced hydraulic residence times. The monitoring of this stratification during the warmer months is of use in predicting likely lagoon performance in terms of bacteria and BOD_5 removal.

If the monitoring of stratification can not be carried out the designer should attempt to predict from the exposure of the lagoon site and wind data the likely amount of wind action on the lagoon and the likely frequency of lagoon stratification. More information is detailed in Chapter 4.

If a lagoon does (or is likely to) stratify frequently then the designer should allow for the decreased performance in organic removal and bacterial kill rate by increasing the size of the cells or incorporating more cells in the design. If this is not possible then changes are necessary to enhance mixing and the transfer of oxygen through improving wind action by removing obstructions around the perimeter of the lagoon system or by using aerators (or mixers).

The surrounding trees should be removed, particularly those upwind of the prevailing wind direction in warmer months when stratification is likely. Banks and baffles should be as low as practical to reduce wind interference and maximise wind speed across the lagoon surface.

Step 3

The designer should then use the measured organic load on the lagoon system to size the first cell in the lagoon, so odour problems are minimised and organic removal is optimised. Removal of BOD_5 was modelled using many design equations ranging from areal loading rate through to linear and empirical equations, and various kinetic models. The method recommended for sizing the primary cell is the areal loading rate method.

It is suggested that lagoons be loaded conservatively at or less than 60 kg/ha.day but not less than 40 kg/ha.d, as the percent removal would then generally drop to less than 75% across the lagoon.

It is recommended that the primary lagoon cell be designed with a regular square or slightly elongated rectangular shape to help create a complete mixed reactor. To optimise a lagoon to account for those occasions when stratification exists the inlets should be sub-surface by approximately 0.6 m to avoid the stratification zones as detected in the study, and 5 to 10 m from the edge of the lagoon to prevent localised odour problems and difficulties in dispersing the influent The primary lagoon may be made slightly deeper than the secondary cells to allow for sludge settling and enhanced digestion.

For both primary and secondary cells the path from the inlet to the outlet should be maximised. Wind under certain circumstances can lead to short circuiting within cells so alignment of the inlet to the outlet to the prevailing wind axis is not recommended. The cells should be rectangular, producing a cell with characteristics between a complete mixed and plug-flow situation. With irregular shaped cells hydraulic residence problems such as dead zones and short-circuiting can develop. Optimally the corners of the lagoon cells should be curved. Study observations indicate that the outlets should be designed to prevent the discharge of surface scum and minimise the transfer of algae by using a protective baffle. The depth the baffle needs to be from the surface is uncertain, however it is suggested that it should be approximately 300 mm deep. Free board (including a wave wall) width should be at least 0.5 m. The width will be dependent on the size of the lagoon and its exposure to the wind for wave development.

Step 4

The secondary lagoons should then be designed primarily for bacterial removal. The study confirmed the applicability of the Marais equation in determining likely bacterial kill rate with certain lagoon configurations; refer to 6.4.4 for detail. However, it was necessary for the decay constant or bacterial kill rate (k) to be altered for various cell configurations and stratified states.

Aside from the advantages of well mixed compared to stratified systems for bacterial die-off rates, bacterial reduction is also affected by cell number, type of separation between cells and cell layout.

It is well known that, for a given total lagoon volume, cells in series perform better than a single cell for bacterial removal (Marais 1966). In reviewing the desirable numbers of cells to cost effectively reduce the bacterial number care should be taken to avoid making individual cells too small (ie less than 5-10 days HRT), otherwise short circuiting is likely and wind mixing can be significantly reduced due to the small fetch.

With baffles and walls free exchange of fluid occurs when the water level is unchanged between cells, while cells separated by levees permit only "one way" directional flow from one cell to the next and have changes in water levels. The study showed that those cells in series separated by a baffle or wall had significantly poorer overall bacterial die off rates than cells separated by levees. Despite cell separation by baffles or walls not being as effective as levees, they are still useful to the designer. A large cell divided up with baffles will achieve a better overall performance than if it had not been. Their inclusion in an existing lagoon may be an economic means to improve the performance of the lagoon system

If the E.coli k value was adjusted according to the extent of stratification and the type of cell separation (baffle or levee), a reasonable estimate of bacterial kill rate performance could be made for each cell using the Marais equation. In order to do this, the Marais equation had to be altered to allow for each cell to have its own E.coli k value. The primary cell, however, appeared to not experience reduced bacterial kill rate performance due to stratification.

For a given cell n in series, the modified Marais equation is given by:

$$N_n = \frac{N_o}{\prod_{i=1}^{n} (k_i t_i + 1)} \qquad (6.4)$$

For the primary cell

$k \approx 1.5$, although it was as high as 3 in some lagoon systems.

For secondary cells

$k \approx 1.5$, for cells with high wind action that rarely stratify

$k \approx 1.0$, for cells with moderate wind action that stratify occasionally

$k \approx 0.5$, for cells with low wind action that stratify frequently.

If the cell is preceded by a baffle the expected k value is further reduced by approximately 60%, i.e., by multiplying the k value above by 0.4.

It has been suggested that the length to width ratio should be 3:1 or greater (USA EPA Manual 1983) which may further enhance the E.coli k values.

The depth of the lagoons in the study ranged from 0.9 to 1.8 m. A conservative recommendation is that the depth should be a minimum of 1 metre for effective stabilisation, odour management, control of emergent weed growth and the effects of algal growth

Summary of other design considerations.

Lagoons that are frequently stratified, due to lack of wind action, have about 40% higher BOD_5 effluent levels than lagoons that are well mixed. The NFR levels in stratified lagoons frequently are only about 8% higher than for well mixed lagoons which is probably not significant.

For maximum BOD_5 removal lagoons should be exposed to as much wind action as possible to ensure full mixing. In areas of low wind action, mechanical mixers may be used to minimise anaerobic and anoxic activity and achieve similar performance to lagoons with good wind action

It should also be considered that lagoons with high wind action which do not stratify regularly are poor at removing total phosphorus (approximately 7 - 15% removal) in comparison to lagoons with poor wind action and which stratify (approximately 30 - 40% removal). The nitrogen removal influences are less clearly defined. Total nitrogen removal in the lagoon systems studied is generally similar, ranging from 47.8 to 57.3 %, whether the lagoons are strongly stratified or fully mixed. There is a marginal improvement of 66.2% removal in the Beaconsfield lagoon system, which is considered to alternate regularly between a fully mixed and stratified state during warmer months.

If regular stratification of the lagoon is a necessary outcome for optimal nutrient reduction, as discussed in this chapter, then placement of trees or construction of risen banks in certain areas around the lagoon would encourage the alternation of mixed and stratified states through the selective interference of wind action. However, under stratified situations the lagoon's hydraulic retention time and/or the number of cells would have to be increased to achieve the desired bacterial kill rate.

In addition to the optimisation of a lagoon system with baffles, inlet and outlet arrangement, and aeration, sand filters and lime dosing are possibilities.

The performance of the intermittent medium flow sand filter, detailed in Table 6.12, was very promising. The hydraulic load through the filter was $1m^3/m^2.d$ which was run for two days and then allowed to dry. The suspended material (mainly algae), organic load, bacterial number, ammonia and chlorophyll "a" were significantly reduced at greater than 90%. The total nitrogen and phosphorus levels were reduced by 51% and 33% respectively.

The addition of lime resulted in a reasonable improvement in reduction of BOD, NFR, chlorophyll "a", Faecal streptococcus, ammonia-nitrogen and total phosphorus ranging from 22 - 36 %. E.coli and ortho-phosphate levels were, however, more significantly reduced at 66 % and 58 % respectively, while total coliforms, nitrate-nitrogen, nitrite-nitrogen, and total nitrogen levels were increased to varying extents. The addition of lime to the lagoons does not present any medium term option of upgrade, but may be of use when requiring a short term reduction in the BOD, NFR, indicator organisms, chlorophyll "a" and some nutrients

6.10 Design Examples

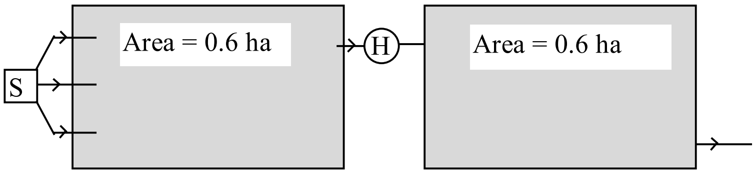
This section includes a number of design examples to demonstrate approaches to lagoon design based on the findings of the SLPIP study.

Design Example 1

A lagoon system is producing an effluent with an E. coli concentration of around 4000 E. coli/ 100ml. The designer wishes to predict whether a lagoon system can be divided up by baffles to conform to the discharge limit of 1000 E. coli/100ml. There is power on site so aerators may be used if odour problems arise. The influent to the lagoon is predominantly domestic and has been determined to have the following characteristics:

Average Flow	= 150 kL/d
BOD_5	= 175 mg/L
E. Coli	= $6 * 10^6$ E. coli/100ml

The lagoon is located in a sheltered calm site. The operators have taken DO profiles which indicate the lagoon stratifies regularly over the summer months. The average depth of both lagoons is 1.2 m. A schematic of the present two cell design is illustrated below.



H = Change in Head Height (i.e. No Back Flow).

(A) The BOD load from the influent is

$$\text{Load (kg BOD}_5\text{/d)} = \frac{\text{Flow (kL/d)} * \text{BOD}_5 \text{ (mg/L)}}{1000}$$

$$\text{Load (kg BOD}_5\text{/d)} = \frac{150 * 175}{1000} = 26.3 \text{ kg BOD}_5\text{/d}$$

(B) As power is available an aerator may be used to control odours when they occur. An areal loading rate of 70 kg/ha/d may be used which may lead to the occasional odour problem. The area required for the primary cell is calculated as follows:

$$\text{Area Required (ha)} = \frac{\text{Load (kg BOD}_5\text{/d)}}{\text{Areal Loading Rate (kg BOD}_5\text{/ha.d)}}$$

$$\text{Area Required (ha)} = \frac{26.3}{70} = 0.38 \text{ ha}$$

Therefore the first lagoon will be divided with a baffle into a 0.38 ha primary cell and a 0.22 ha secondary cell.

(C) The volume of the primary cell would be calculated as follows:

$$\text{Volume (m}^3\text{)} = \text{Area (ha)} * \text{Depth (m)} * 10000$$

$$\text{Volume (m}^3\text{)} = 0.38 * 1.2 * 10000 = 4560 \text{ m}^3$$

(D) The Theoretical Hydraulic Retention Time (t) is then calculated,

$$\text{t (Days)} = \frac{\text{Volume (m}^3\text{)}}{\text{Flow (kL/d)}}$$

$$\text{t (Days)} = \frac{4560}{150} = 30 \text{ d}$$

(E) The approximate bacterial concentration being discharged from this primary cell is then calculated using Marais equation with an E. coli k value of 1.5, from Section 6.4, as it is the first cell,

$$N = \frac{N_0}{(kt + 1)}$$ where N = E. coli concentration in cell effluent (No./100ml)
N_0 = E. coli concentration of the cells influent (No./100ml)

$$N = \frac{6 * 10^6}{(1.5*30 + 1)} = 1.3 \quad 10^5 \text{ (E.coli/100ml)}$$

(F) The final cells should be designed for bacterial reduction taking into account both wind action and cell separation. The designer has also chosen to divide the second lagoon of the original design into two separate cells of equal size using a baffle. The bacterial concentration in the final cell of the lagoon is then calculated as follows:

$$N_4 = \frac{N_1}{(k_2t_2 + 1)\,(k_3t_3 + 1)(k_4t_4 + 1)}$$ where t_2,t_3,t_4 = Hydraulic retention time for each cell
k_2,k_3,k_4 = The k values for each cell

As the secondary cells have been shown to stratify regularly the maximum average E.coli k value for the secondary lagoons is likely to be around 0.5. The k and t values are determined as follows:

$k_2 = 0.2$: The second cell is separated from the primary cell by a baffle so the E.coli k value of this second cell will be further reduced by 60% from 0.5 to 0.2.

$k_3 = 0.5$: The third cell is preceded by a levee, which drops the water level from the second to the third cell, so the E.coli k value will be 0.5; the estimated maximum for a frequently stratified lagoon.

$k_4 = 0.2$: The last (fourth) cell is preceded by a baffle which reduces the E. coli k value by 60% from 0.5 to 0.2

The hydraulic retention times for these three latter cells are calculated in the same way as the primary cell, i.e. for the second cell,

Volume (m^3) = Area (ha) * Depth (m) * 10000
Volume (m^3) = 0.22 * 1.2 * 10000 = 2640 m^3

$$t_2 \text{ (Days)} = \frac{\text{Volume } (m^3)}{\text{Flow (kL/d)}} = \frac{2640}{150} = 17.6 \text{ d}$$

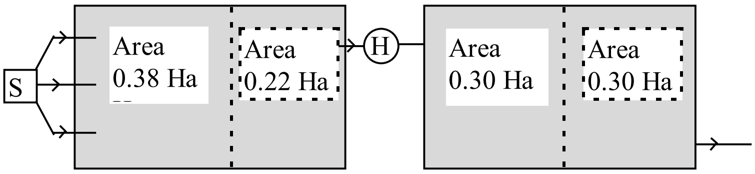
The results for the third and forth cells are:

t_3 = 24 days
t_4 = 24 days

These values are then substituted back into the modified version of Marais equation to solve for the final cell effluent E.coli concentration:

$$N_n = \frac{1.3 * 10^5}{(0.2*17.6 + 1)(0.5*24 + 1)(0.2*24 + 1)} = 380 \text{ E.coli/100ml}$$

Thus the suggested design for the lagoon system to achieve an effluent with an average E.coli concentration within the discharge limit of 1000 E.coli/100ml is shown schematically.



H = Change in Head Height.

Design Example 2

A new lagoon system is required in Tasmania. The system will be situated in either a well exposed windy site or a sheltered calm site. The raw influent is predominantly domestic with the following characteristics:

Flow = 200 kL/d
BOD_5 = 225 mg/L
E. Coli = $6 * 10^6$ E. coli/100ml

The design is to achieve an average E. coli concentration of 200 E.coli/100ml and not be odourous under normal working conditions.

The primary cell is designed as follows using the areal loading rate of 50 kg of BOD/ha.d.

(A) The BOD load from the influent would be:

$$\text{Load (kg BOD}_5\text{/d)} = \frac{\text{Flow (kL/d)} * \text{BOD}_5\text{ (mg/L)}}{1000}$$

$$\text{Load (kg BOD}_5\text{/d)} = \frac{200 * 225}{1000} = 45 \text{ kg BOD}_5\text{/d}$$

(B) To determine the primary cell surface area required for the influent BOD_5 load of 45 kg/d and the areal loading rate of 50 kg/ha.d perform the following calculation:

$$\text{Area Required (ha)} = \frac{\text{Load (kg BOD}_5\text{/d)}}{\text{Areal Loading Rate (kg BOD}_5\text{/ha.d)}}$$

$$\text{Area Required (ha)} = \frac{45}{50} = 0.90 \text{ ha}$$

(C) If the lagoon is built with an average depth of 1.0 m the volume of the primary cell would be calculated as follows:

$$\text{Volume (m}^3\text{)} = \text{Area (ha)} * \text{Depth (m)} * 10000$$

$$\text{Volume (m}^3\text{)} = 0.90 * 1.0 * 10000 = 9000 \text{ m}^3$$

(D) The theoretical hydraulic retention time (t) is then calculated,

$$\text{t (Days)} = \frac{\text{Volume (m}^3\text{)}}{\text{Flow (kL/d)}}$$

$$\text{t (Days)} = \frac{9000}{200} = 45 \text{ d}$$

(E) The approximate effluent bacterial concentration from the first cell is calculated using Marais equation. As it is the first cell the conservative value for an E. coli k value, as determined from the study, is 1.5.

$$N_1 = \frac{N_0}{(kt + 1)}$$ where N_1 = E. coli concentration in the cell effluent (No./100ml)
N_0 = E. coli concentration of the cells influent (No./100ml)

$$N_1 = \frac{6 * 10^6}{(1.5*45 + 1)} = 8.8 * 10^4 \text{ (E.coli/100ml)}$$

The final lagoons, cells 2 and 3, are to be designed for bacterial reduction and the amount of wind action is taken into account. The wind action was not taken into account with the first cell as the study found that the bacterial performance of the primary cell was not significantly effected by wind action.

With new cells it is more desirable for performance reasons to build individual cells that are separated by a levee with a change in water level. The approximate bacterial concentration being discharged from the third cell is calculated using the modified Marais equation, from Section 6.4:

$$N_3 = \frac{N_1}{(k_2t_2 + 1)(k_3t_3 + 1)}$$ where t_2,t_3 = hydraulic retention time for each cell
k_2,k_3 = k values for each cell

Given that the second and third cells will be the same dimensions they will therefore have the same theoretical hydraulic retention time, i.e., t_2 will equal t_3. Also, as both cells are preceded by levees with a drop in water level and are likely to have the same wind action (provided both cells have similar amounts of shelter from the wind), k_2 will equal k_3.

For a *well exposed windy site* the E.coli k value for secondary cells will be approximately 1.5 from Section 6.4.

The above equation could then can be rewritten as:

$$N_3 = \frac{N_1}{(kt + 1)^2} \quad \Rightarrow \quad t = \frac{(N_1/N_3)^{1/2} - 1}{k}$$

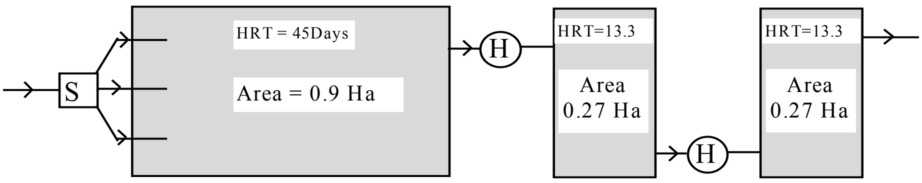
$$t = \frac{(8.8 * 10^4/200)^{1/2} - 1}{1.5} = 13.3 \text{ d}$$

Therefore two more cells would have to be built each having a hydraulic retention time of 13.3 days. If the cells have an average depth of 1.0 m they would have to be 0.27 hectares each, i.e.,

$$\text{Volume (m}^3\text{)} = 13.3 \text{ d} * 200 \text{ kL/d} = 2660 \text{ m}^3$$

$$\text{Area Required (ha)} = \frac{\text{Volume (m}^3\text{)}}{\text{Depth (m)} * 10000} = \frac{2660}{1*10000} = 0.27 \text{ ha}$$

Thus the suggested design for the lagoon system to achieve an effluent with an average E.coli concentration of 200 E.coli/100ml is shown schematically.



H = Change in Head Height.

For a location with ***limited wind action*** the E. coli k value for the secondary cells may drop to as low as 0.5 and the hydraulic retention time required would be calculated as follows.

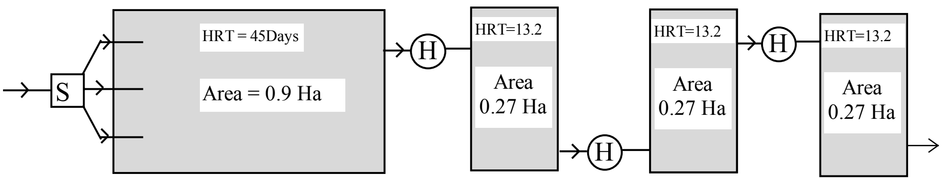
$$t = \frac{(8.8 * 10^4/200)^{1/2} - 1}{0.5} = 40 \text{ Days}$$

As this hydraulic retention time is quite long it is probably more economic to build a fourth cell in series. Under these same conditions the hydraulic retention time would be calculated as follows:

$$N_4 = \frac{N_1}{(kt+1)^3} \quad \Rightarrow \quad t = \frac{(N_1/N_4)^{1/3} - 1}{k}$$

$$t = \frac{(8.8 * 10^4/200)^{1/3} - 1}{0.5} = 13.2 \text{ Days}$$

For a less windy site it would be recommended that a four cell system should be built with the final three cells having a hydraulic retention time of 13.2 days. If the cells have an average depth of 1.0 m they would have to be 0.27 hectares each. The suggested design is shown schematically.



H = Change in Head Height.

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Appendix A

Examples of organic areal loading rates for facultative lagoons from literature.

Average winter air temperature or climate	BOD_5 Loading Rate (kg/ha.d)	Retention Time (d)	Depth (m)	BOD_5 Reduction (%)	Source
	22-56	7-50	1-3	70-95	Eckenfelder (1980)
	50	120	2		GHD (1978)
>15°C	45-90				USEPA (1983)
0-15°C	22-45				
<0°C	11-22	120-180*			
Temperate to semi-tropical	50-150	33-100**			Gloyna (1971)
5-10	<60***	7-100	1.2-2.5	75	SLPIP (1996)***

The loading rate applies to the first pond in a series.

* Total retention time in system

** Based on influent of 100L/c.d

*** BOD_5 concentration is a function of retention time. This areal loading rate is recommended for Tasmania.

APPENDIX B

DESIGN EQUATIONS:

A) Organic Removal

1. **Gloyna (1971) used his equation to determine the design volume of facultative lagoon to achieve 85 -95 % BOD removal efficiency.**

$$V = (3.5 \times 10^{-5})\ Q\ La\ \theta\ (35\text{-}T)\ f\ f'$$

Where V = lagoon volume, m^3
Q = influent flow rate, L/d
La = ultimate influent BOD or COD, mg/L
θ = temperature correction coefficient = 1.085
T = average lagoon temperature of coldest month (^0C)
f = algal toxicity factor, which is assumed to be equal to 1 for domestic wastes
f' = sulphide oxygen demand, which is assumed to equal 1 for sulphate equivalent ion concentration less than 500mg/L

2. **Marais and Shaw (1961) based their equation on a complete mixed model and first order reaction. They used the equation for BOD reduction in the first cell.**

$$Le/Li = 1/(1+kt)$$

Where Le = effluent BOD (mg/L)
Li = influent BOD (mg/L)
k = rection rate constant (d^{-1})
t = hydraulic retention time (d)

3. **Marais (1970) extended the kinetic model to more closely account for the contributions of both the aerobic and anaerobic conditions in the degradation process.**

$$Le/Li = 1/(1+k_e t)$$

Where Le = effluent BOD (mg/L)
Li = influent BOD (mg/L)
ke = equivalent reaction rate constant (d^{-1}), which changes with warmest month average daily maximum teperatures.
t = hydraulic retention time (d)

4. **Thirumurthi (1974) developed his equation for the plug flow model to determine the estimated effluent BOD.**

$$Ce/Ci = \exp(-k_p t)$$

Where Ce = effluent BOD (mg/L)
Ci = influent BOD (mg/L)
k_p= plug flow first order rection rate, (d^{-1}) = $k_{p20}\,(1.09)^{T-20}$
t = hydraulic retention time (d)

5. **Wehner-Wilhelm (1956) developed their equation to consider the scenario of the flow pattern in lagoons to be somewhere between plug flow and completely mixed.**

$$Ce/Co = 4\ ae^{(1/2D)} / \{(1+a)^2 (e^{a/2D}) - (1-a)^2 (e^{-a/2D})\}$$

Where Ce = effluent BOD (mg/L)
Ci = influent BOD (mg/L)
e = base of natural log, 2.7183
$a = (1 + 4ktD)^{1/2}$
k = first order reaction rate constant, d^{-1}
t = hydraulic retention time, d
D = dimensionless dispersion number = H/vL = Ht/L^2
= 0 for plug flow
= ∞ for complete mixed reactor
H = axial dispersion coefficient, area per unit time
v = fluid velocity, length per unit time
L = length of travel path of a typical particle

B) Bacterial Reduction

1. **Marais (1974) equation dealing with faecal bacteria death kinetic for series operation of i cells which assumes k is constant for each cell is given by:**

$$N_n = \frac{N_o}{\prod_{i=1}^{n} (kt_i + 1)}$$

where t_i =influent retention time for each individual cell, days (where t can be calculated from V_i and Q_i as above)
Π=product of all $(kt_i + 1)$ where i =1,2,3.....,
i.e., for two cells, $(kt_1 + 1)x(kt_2 + 1)$
N_n=concentration of faecal bacteria (number per unit volume) in the nth cell (No./100ml)

Experimental data show the effect of temperature on k value for E.coli to be:

$$k_t = 2.6\ (1.19)^{T-20}$$

whereT = temperature (^{0}C)

2. **Marais (1974) SLPIP modified(1996) equation.**

For a given cell n in series, the modified Marais equation is given by:

$$N_n = \frac{N_o}{\prod_{i=1}^{n} (k_i t_i + 1)}$$

For the primary cell

k ≈ 1.5, although it was as high as 3 in some lagoon systems.

For secondary cells

$k \approx 1.5$, for cells with high wind action that rarely stratify

$k \approx 1.0$, for cells with moderate wind action that stratify occasionally

$k \approx 0.5$, for cells with low wind action that stratify frequently.

If the cell is preceded by a baffle the expected k value is further reduced by approximately 60%, i.e., by multiplying the k value above by 0.4.

C) Sludge StorageVolume

The sludge storage volume can be calculated using the following equation (Water Resources 1992):

$$V_s = X_v \, Q \, (0.63 + 2.2 \, t_s)$$

Where V_s = storage volume (m^3)
X_v = influent volatile suspended solids concentration (mg/L)
Q = flow rate (ML/d)
t_s = time between desludging of lagoon (years)

D) Facultative Aerated Lagoon

The purpose is to maintain a positive dissolved oygen level in the surface zone withou disturbing the anaerobic zone. If an aspirator type aerator is used then the simple design procedure (Dr M Johns, Qld Uni, pers.comms) is

$$P = BOD_5 \text{ x a x b}$$

Where P = power required (kW.h)
BOD_5 = BOD load (kg/d)
a = oxygen demand per BOD_5 consumed (kg/kg BOD_5) = 1.2 for domestic sewage
b = aerator efficiency = 0.8 to be conservative

A power to volume ratio (P/V) in the aerated cell of 0.4 is adequate. The propellor shaft needs to be set to avoid scouring the lagoon bed.

APPENDIX C

COMPARISON OF VALUES FROM DESIGN EQUATIONS TO ACTUAL RESULTS

VARIABLES AND EQUATIONS		Turners Beach	Beaconsfield	Campbell Town	Fingal	Dover	Bothwell
Average water temp	Ta	17	15.2	13.6	15.85	13.2	12.5
Min daily water temp	Tm	9	9	6.5	5.9	5.8	5.5
BOD_5 Influent (mg/L)	Li, Ci, Co	207	232	185	303	95	164
COD Estimate		414	464	370	606	190	328
Flow (kL/d)	Q	367	241	377	88.7	155	157
Primary Surface Area (Ha)	A	1.035	0.668	0.939	0.39	0.378	0.49
Total Surface Area (Ha)		2.37	1.72	1.7041	0.581	0.7997	0.6962
Prim Depth (Average)	D	0.95	0.96	1.47	1.2	1.2	1.2
Primary Lagoon Volume (kL)	V	9832.5	6412.8	13803.3	3562	4536	5880
Total Lagoon System Volume (kL)		21600	17500	22600	5280	7270	7610
V assuming 1m depth	V2	10350	6680	9390	3900	3780	4900
Av. HRT (theoretical) in Primary	t	26.8	26.6	36.6	40.2	29.3	37.5
Average BOD_5 load (Kg/Day)		77	47.3	66	27	14.5	25.7
Aereal loading rate Primary (Kg BOD_5/Ha/d)		74.4	70.8	70.3	69.2	38.4	52.4
Gloyna V=0.035*Q*Li*1.085^(35-Tm), (m3)		22176	16321	24965	10103	5580	10000
Measured BOD_5 Primary Effluent (mg/L)		65	58	46	47	28	34
Measured BOD_5 Lagoon System Effluent (mg/L)		30	50	37	49	29	36
Actual BOD_5 Removal Across System (%)		87.2	78.5	80	83.8	69.5	78.1
Temperature Mean Daily Max Feb		21	23	24	22	21	23
Temperature Mean Daily Max July		11	12	10	10	11	10
K est (d-1)	k	0.144	0.144	0.117	0.112	0.111	0.108
Kest 2 (d-1)	k_e	0.07	0.08	0.08	0.07	0.07	0.075
Marais & Shaw(1961) Le=Li(1/(1+kt))		42.6	48.0	34.9	55.2	22.4	32.5
Marais & Shaw(1964) Le=Li(1/(1+k_et))		71.99	74.15	47.08	79.51	31.16	43.06
Average of Marais & Shaw 1961 & 1964		57.3	61.08	41	68	27	38
%Error of Marais & Shaw(1961) to observed		-34.4	-17.2	-24.1	17.5	-20.0	-4.5
% Error of Marais & Shaw(1964) to observed		10.8	27.8	2.4	69.2	11.3	26.6
% Error of Average Marais & Shaw(1961 &64) to observed		-12	5	-11	44	-4	11
Reaction Rate plug flow k_p		0.04	0.04	0.04	0.03	0.02	0.03
Thirumurthi Ce=Ci exp(-k_pt)		70.1	81.7	50.0	75.3	50.4	57.1
% Error of Thirumurthi to measured Ce		7.8	40.9	8.7	60.3	80.0	67.9
k		0.058	0.058	0.047	0.045	0.044	0.043
a		1.27	1.27	1.30	1.31	1.23	1.28
Wehner-Wilhelm Equation		51.8	58.6	40.9	63.3	29.5	39.4
% Error of Wehner-Wilhelm to measured Ce		-20.2	1.0	-11.2	34.7	5.5	15.8

CHAPTER 7

LAGOON CONSTRUCTION

Contents

Table

7.1 Introduction

A literature search showed there is a very little Autralian information on sewage lagoon practice. The most detailed information on this topic is found in the USA Environmental Protection Agency - Design Manual, Municipal Wastewater Stabilisation Ponds (1983). This section has been adapted from the US Manual for Tasmanian situations with due acknowledgment. Readers are referred to the US Manual for additional information not covered here. Some farm dam texts cover techniques used in embankment construction and protection. Nelson (1985) is one such text.

The design of sewage lagoons has two parts: one is the process design and the other is the layout, construction, structure, hydraulics or whatever one wished to call it. Similar to other process designs the latter is equally important as the former. If insufficient attention is given to the layout the actual performance of the system may be far less than calculated. A plant that is well thought out, in accordance with the latest findings, will perform to the design efficiency and be easy to operate and maintain.

For length to width ratio of lagoons refer to Chapter 6 of this manual.

Other matters that need to be considered in the construction of lagoons are the danger of ground water contamination, embankment erosion due to wave action, weather and rodent attack, weed control and other operational considerations. These important physical considerations are discussed in the following sections.

This chapter only covers some sewage lagoon practices and does not cover the engineering aspects. It is expected that the engineering be carried out by a qualified person.

7.2 Embankment Construction

Embankment stability and integrity are often affected by erosion caused by wind driven wave action, rain induced weathering and damaged by burrowing rodents. A good design will incorporate answers to these problems and provide a system which can, through cost-effective operation and maintenance, keep all three under control.

7.2.1 Wave protection

Erosion protection should be provided on all slopes; however, if winds are predominantly from one direction, protection should be emphasised for those areas that receive the full force of the wind driven waves. Erosion protection from wave action should always extend to at least 0.3 m above and below the water level. Wave height is a function of wind velocity and fetch (the distance over which the wind acts on the water). The size of riprap depends on these two factors. Riprap varies from river run rocks that are 15-20 cm to quarry boulders that are 7-14 kg. Uniformly graded river run material, when used for riprap, can be quite unstable. River run rocks, if not properly mixed with smaller material and carefully placed, can be loosened by wave action and slip down the steeper sloped embankments. Broken concrete pavement and large stones can be used for riprap but can make mechanical weed control very difficult.

The construction of wave band provides a clean edge to a lagoon, easing maintenance and preventing erosion. Concrete, fabric and low grasses can also be used to provide protection from wave action. When riprap is used for wave protection, the designer must take into consideration its effect on weed and rodent control and routine embankment maintenance.

Concrete wave slabs must be keyed into the embankment. The use of precast slabs is not recommended owning to the difficulty of providing an adequate key. In addition to unkeyed slabs slipping, joints between precast slabs are prone to weed infestation.

PVC and plastic sheet and rubber liner have been used on some sites but found to come loose after some years. This may be due to the construction technique. If they are to be used, the advice is to adequately anchor the liner. Some methods are documented in USEPA (1983).

Poorly constructed embankments can weather and fall in, and affect the design volume of the lagoons which affects the treatment efficiency of the system.

7.2.2 Weather protection

Embankment slopes must be protected from weather erosion as much as from wave erosion in many areas of the State. The most common method of weather erosion protection when large embankment areas are involved uses grass. Lagoons which have only minimum freeboard and have constant water depth are often protected more cost-effectively when the riprap is carried right to the top of the slope and serves for both wave and weather protection.

In some cases weather and soil conditions are suitable for completely bare embankment slopes without major weather erosion problems. The designer needs to use engineering judgement on this.

Weather erosion, unlike wave erosion, can also affect the top and outside slopes of the lagoon embankment system. The designer should be aware of the potential of vetting, due to operation and maintenance vehicles, that can create runoff erosion problems in areas of high rainfall intensity. Final grading should be specified to minimise these effects.

It is also necessary to protect the exterior surface of embankment. A thin layer of gravel may be used; placement of topsoil and seeding with grass may be less expensive initially but grass requires periodic cutting. In some locations sheep can be used to keep exterior grassed slope maintained. Other native ground cover plantings may also be used. Road works experience on erosion control for cut-and-fill slopes can be used as a guide.

7.2.3 Protection from burrowing animals

If a sewage lagoon is located in an area that supports an exceptionally high population of burrowing animals, good design can control this threat of embankment stability. Broken concrete or other riprap that does not completely cover the embankment soil can become a home for burrowing animals. Riprap design and placement should be aimed toward limiting the creation of voids which allow animals to burrow near the water surface.

Varying lagoon water depth can discourage animal infestation. Some animals prefer a partially submerged tunnel, so design provisions to vary the water level over a several week period will discourage them from burrowing in the embankment. Such provisions will often add to the expense of riprap placement for wave protection but can greatly reduce operation and maintenance expenses.

Local knowledge of the burrowing animal behaviour in the area would assist in the design and maintenance of the system to overcome this problem.

7.2.4 Seepage

Lagoons should be designed and constructed to minimise seepage. Vegetation and porous soils should be removed and the embankment should be well compacted. Use of conventional construction equipment and procedures are usually suitable for this purpose.

Seepage collars should be provided around any pipes penetrating the embankment. The seepage collars should extend a minimum of 0.6 m from the pipe. Seepage collars could be of concrete, clay or other suitable material. Conventional construction techniques should be used.

In some circumstances it may be necessary to control seepage and ensure bank stability at the exterior toe. A filter blanket material can be used. Another method of preventing seepage where embankment material cannot be adequately compacted is placement of an impervious core in the levee with imported material.

7.3 Lagoon Sealing

7.3.1 Introduction

The need for a well sealed lagoon has impacted on modern lagoon design, construction and maintenance. The primary motive for sealing lagoons is to prevent seepage. Seepage affects treatment capabilities by causing fluctuation in the water depth and can cause pollution of ground water. Although many types of lagoon sealers exist, they can be classified into one of three major categories:

(1) synthetic and rubber liners;
(2) earthen and cement liners; and
(3) natural and chemical treatment sealers.

Within each category also exists a wide variety of application characteristics. Choosing the appropriate lining for a specific site is a critical issue in lagoon design and seepage control. Detailed information on (1) is available from manufacturers.

7.3.2 Seepage rates

Seepage is a function of so many variables that it is often impossible to anticipate or predict rates even with extensive soil tests. The importance of controlling seepage to protect ground water dictates that careful evaluations be conducted before construction of lagoons to determine the need for linings and the acceptable types.

The following organisations may be able to give advice on soil and ground water: Department of Primary Industry & Fisheries, Tasmania and Tasmanian Development & Resources - Mineral Resources Division.

It was reported in USEPA (1983) that the American Minnesota Pollution Control Agency initiated an intensive study to evaluate the effects of sewage lagoon seepage from five municipal systems.

Field permeability tests indicated that the additional sealing created by the sludge blanket was insignificant in locations where impermeable soils were used in the construction process. In the case of more permeable soils, it appeared that the sludge blanket reduced the permeability of the bottom soils from an initial level of 10^{-4} or 10^{-5} cm/sec to the order of 10^{-6} cm/sec. At all five systems evaluated in that study, the lagoon was in contact with the local ground water table. Local ground water fluctuations had a significant impact on seepage rates. The buildup of sludge on the bottom of a lagoon appears to improve the quality of the seepage water leaving the lagoon.

Ground water samples obtained from monitoring wells in the Minnesota study did not show any appreciable increases in nitrogen, phosphorus, or faecal coliform over the background levels after 17 years of operation. The seepage from the lagoons did show an increase in soluble salts as great as 20 times over background levels. Concentrations of 25 mg/l to 527 mg/l of chloride were observed.

A comparison of observed seepage rates for various types of liner material is presented in Table 7.1. If an impermeable liner is required, it appears that one of the synthetic materials must be used.

Table 7.1 Seepage Rates for Various Liners
(*Source: USEPA "Design Manual, Municipal Wastewater Stabilisation Ponds*" 1983)

Liner Material	Thickness (cm)	Minimum Expected Seepage Rate at 6 m of Water Depth After 1 Year of Service (cm/d)
Open sand and gravel		244
Loose earth		122
Loose earth plus chemical treatment*		30.5
Loose earth plus bentonite*		25.4
Earth in cut		30.4
Soil cement (continuously wetted)	10.2	10.2
Gunite	3.8	7.6
Asphalt concrete	10.2	3.8
Unreinforced concrete	10.2	3.8
Compacted earth	91	0.76
Exposed prefabricated asphalt panels	1.3	0.08
Exposed synthetic membranes	0.11	0.003

The data are based on actual installation experience. The chemical and bentonite (*) treatments depend on pretreatment seepage rates and in the table loose earth values are assumed.

7.3.3 Natural and chemical treatment sealing

The most interesting and complex techniques of lagoon sealing, either separately or in combination, are natural lagoon sealing and chemical treatment sealing. Only these types of sealing will be discussed. The reader is referred to US EPA (1983) for more information on other types of sealing.

Natural sealing of lagoons has been found to occur from three mechanisms:

(1) physical clogging of soil pores by settled soils;
(2) chemical clogging of soil pores by ionic exchange; and
(3) biological and organic clogging caused by microbial growth at the lagoon lining.

The dominant mechanism of the three depends on the characteristics of the wastewater being treated. Chemical treatment changes the nature of the bottom soil to ensure sealing.

USEPA (1983) cited a study carried out on infiltration characteristics of anaerobic lagoons in New Zealand. Certain soil additives were employed (bentonite, sodium carbonate, sodium triphosphate) in 12 pilot lagoons with varying water depth, soil type and compacted bottom soil thickness. It was found that chemical sealing was effective for soils with a minimum clay content of 8 percent and a silt content of 10 percent. Effectiveness increased with clay and silt content.

Four different soil columns were placed at the bottom of an animal wastewater lagoon to study physical and chemical properties of soil and sealing of lagoons. It was discovered that the initial sealing which occurred at the top 5 cm of the soil columns was caused by the trapping of suspended matter in the soil pores. This was followed by a secondary mechanism of microbial growth that completely sealed off the soil from water movement.

7.4 Lagoon Stability

Embankment slope should generally be between 1 in 2 and 1 in 3 to dissipate energy. Water Resources (1984) quoted figures up to 1 in 3.5 for certain type of soil. It is recommended conventional earthwork practices be applied in Tasmania. An alternative approach is to use steep embankments to give greater depth of liquid more rapidly and to dissipate wave energy by breaking up waves into spray. Spray may cause erosion when wave is reflected back into the lagoon. Steep slopes are therefore more acceptable for small lagoons.

The required degree of compaction of embankments will depend on the nature of the local material. Generally for cohesive soils, compaction at the optimum moisture content range of 95% to 98% of standard maximum dry density is suitable. Fill material should be placed and compacted in uniform layers of about 150 mm.

Consideration should be given to vehicle access when designing the layout of the lagoons. The designer should consult with lagoon operators where possible to ensure ease of vehicle access.

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CHAPTER 8

OPERATION AND MAINTENANCE

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8.1 Introduction

Sewage treatment lagoon systems are obviously simpler to operate than mechanical/biological plants but they do not perform well for very long without attention. In fact, lagoon systems can become a liability if not properly managed. They are investments that the owner of the facility has adopted as a means of providing adequate treatment prior to discharge.

Maintenance issues should be incorporated into all levels of planning, design, construction, management, and operation. The management of these plants must have an effective operation and maintenance program which utilises an organisational chart that clearly defines the responsibilities of those involved. They are systems that require operator competency in lagoon management. The competency is achieved through adequate training and awareness of planned or preventative maintenance to be routinely performed and any trouble-shooting maintenance. To maintain and manage these sites effectively operators must have sufficient tools and materials.

In-field observation and testing, and result interpretation is an integral part of lagoon system management. Adequate monitoring programs are necessary to generate sufficient information to regulate influent characteristics and control lagoon system performance.

8.2 Plant Management

8.2.1 General

Plant management of lagoon type wastewater treatment systems involve two main parties: the owner, who in a majority of cases in Tasmania is the Municipal Council, and the operators. While the owner has the ultimate responsibility for the performance of the treatment facility to comply with the Environment Protection (Water Pollution) Regulations 1974 and, when finalised, the Tasmanian Sustainable Development Water Quality Management Policy (which is to replace the Regulations), the operators are responsible for the proper operation and maintenance of the lagoon systems.

For the plant to be well managed, direct lines of communication must exist between the operators and other employees associated with the facility. Problem solving is made easy with close cooperation between the engineer, experienced local operators and the health officers. A basic principle of the operation and maintenance of a facility is that each of the employees associated with the facility should be taught not only their particular responsibilities but also the rudiments of the entire process at the treatment facility. This approach will assist in better managing problem odours and other potential nuisances associated with the operation of the treatment plant. A simplified instruction manual for the operation and maintenance of the specific plants should be available for easy referencing.

8.2.2 Owner responsibilities

The owner is responsible for the following:

- making policy decisions regarding the lagoon operation;
- engaging operators who are conscientious, competent, and capable of operating and maintaining the treatment plant. The operators must be provided with proper instruction and orientation;
- providing a replacement for an operator who ceases to be available for that position. The replacement should be adequately trained to ensure there is no deficiency in their competency;
- facilitating through protocol a safe working environment;
- providing a working environment that facilitates retention of trained and experienced personnel;
- encouraging operators to attend meetings and training courses to increase their technical knowledge;.
- ensuring compliance with the Environmental Management and Pollution Control Act 1994 and 1974 Regulation discharge limits (or succeeding State Policy);
- obtaining relevant permits and information for the operation of the plant and locating copies of these items at the plant for referencing by operators.

8.2.3 Operator responsibilities

The operator is responsible for the following:

- the proper operation and maintenance of the treatment plant;
- maintaining a safe working environment and being safety conscious;
- conducting the tests and observations required for the proper operation of the lagoon system;
- the optimum performance of the plant;
- providing the results to the owner/EHO/Supervisor in an intelligible form;
- interpretation of laboratory results and application to the better management and operation of the plant;
- determining the flows and loading rates;
- becoming fully acquainted with the plant and its treatment process;
- taking advantage of the training that may be offered;
- notifying the owner/EHO/supervisor in sufficient time for the need for equipment, parts and tools;
- notifying the owner/EHO/supervisor immediately of operational difficulties;
- be aware and implement the conditions in the permit to operate the plant.

8.3 Monitoring

It is important to realise in monitoring the performance of a plant that:

"Test results used for performance assessment are only as good as the sample taken".

8.3.1 Purpose of monitoring

Monitoring is an integral part of the management of treatment plants. Such monitoring assists to:

- quantify and qualify liquid (and solid) waste streams entering and leaving the treatment plant;
- evaluate the performance within the lagoon system and assess lagoon system performance to review management practices and/or upgrade options;
- determine effluent quality and quantity to ascertain suitability for re-use or evaluate the potential impact on the receiving waters.

The determination of the characteristics of the influent is necessary to identify the wastewater entering the plant from domestic and industry sources and to optimise influent through waste minimisation programs and trade waste agreements.

As the quality and quantity of the influent from primarily domestic sources should remain reasonably stable, the frequency of testing could be substantially less than for monitoring programs of industrial sources. It is important to establish whether the influent hydraulic load is influenced by stormwater infiltration and what the actual equivalent person hydraulic load is. Trade waste sources should be identified and permissible levels of performance parameters established and regulated to ensure compliance with any trade waste agreements.

8.3.2 Parameters

It is necessary in the management and operation of a plant to know the quantity, concentration and type of waste entering the plant, the performance within the plant, and the quality and quantity of treated wastewater leaving the plant. The following parameters are used to assess the performance of the plant and to predict the operational changes necessary to maintain good plant performance:

- pH, temperature, dissolved oxygen, flow, biochemical oxygen demand, chemical oxygen demand, non-filtrable residue, oil and grease, faecal coliforms, nutrients (nitrogen as ammonia, nitrate, nitrite, organic: phosphorus as total and ortho-phosphate), chlorophyll "a", and chlorine residual (if disinfecting with chlorine).

An explanation of some of the parameters is detailed below:

8.3.2.1 Flow

Flow

- is determined using pump station data, in-line flow meters, weirs and/or instantaneous flow measurement using a bucket;
- is used for determining hydraulic load (kL/d) on the lagoon system and the organic load (kg/d, kg/ha.d) when linked to the influent wastewater concentration;
- is also indicative of infiltration of stormwater to sewer and /or seepage problems through the lagoon floor.

Methods:

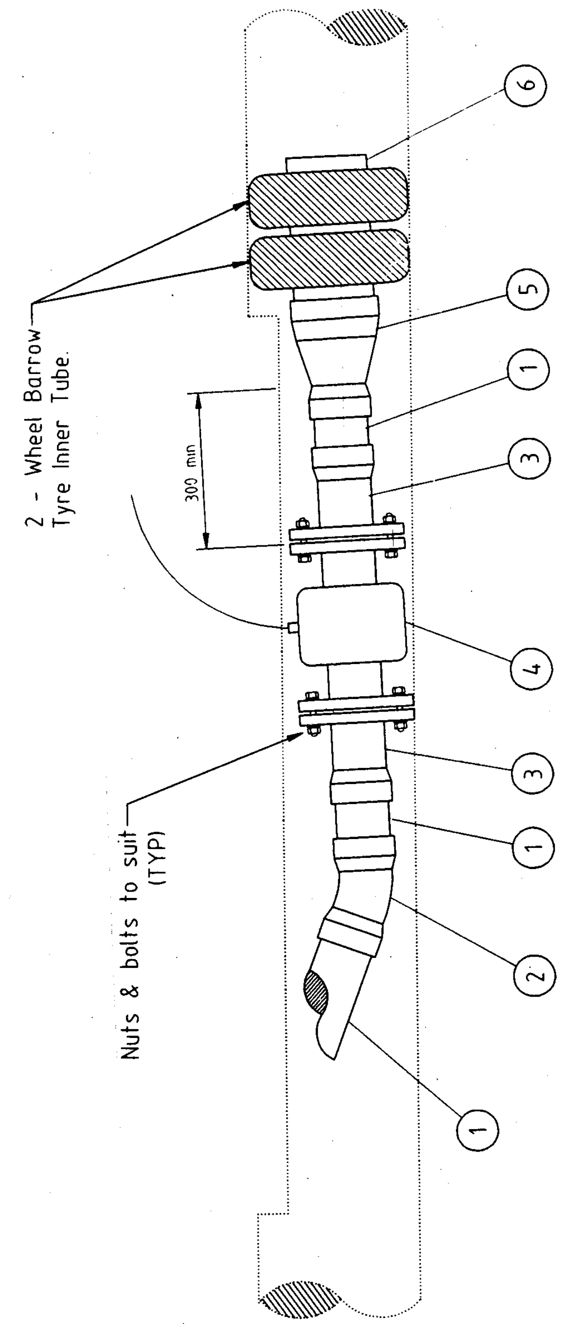
As influent to a sewage lagoon system can be supplied via gravity feed and/or by single or multiple pump stations, different procedures of flow determination are necessary. Those systems with gravity feed require flow meters to be installed upstream of the inlet while those with influent pumped to the system can utilise such information as pump running time and pump capacity and efficiency.

a) Flow Measurement - for gravity feed systems

The method used to measure flow for these systems is the installation of flow meters upstream of the inlet. An easy cost effective method was developed during the Sewage Lagoon Performance Improvement Program (SLPIP). The method involved the insertion of a portable magnetic flow meter (with plumbing attachments) within the existing sewer pipe entering the lagoon system. The meter can be taken to each treatment site and installed within minutes. A signal per kilolitre generated by the flow meter makes it possible for a composite sampler to count the signals and take a corresponding flow proportional sample.

The installation of the flow meter involves assembling the meter with the plumbing parts as per Figure 8.1. Pressure pipe is used in the assembled unit. A 45 degree elbow at the end of the pipe work ensures that the flow meter remains full at all times. For accurate results it is important that there is sufficient length of pipe, of the same diameter, upstream and downstream of the magnetic flow meter to give laminar flow. The only specifications for the "turbo-magnetic" flow meter is that the upstream length of pipe attached to the meter is at least 3 times the diameter of the meter. This is to ensure the flow through the meter was laminar. Manual calibration of this system, using the instantaneous flow measurement (with a bucket) technique, showed the flow measurements were within the specifications of the magnetic flow meter, i.e., there was less than 2% error for a 100kL/day plant with the percentage error decreasing for larger flows.

Figure 8.1 Portable magnetic flow meter design

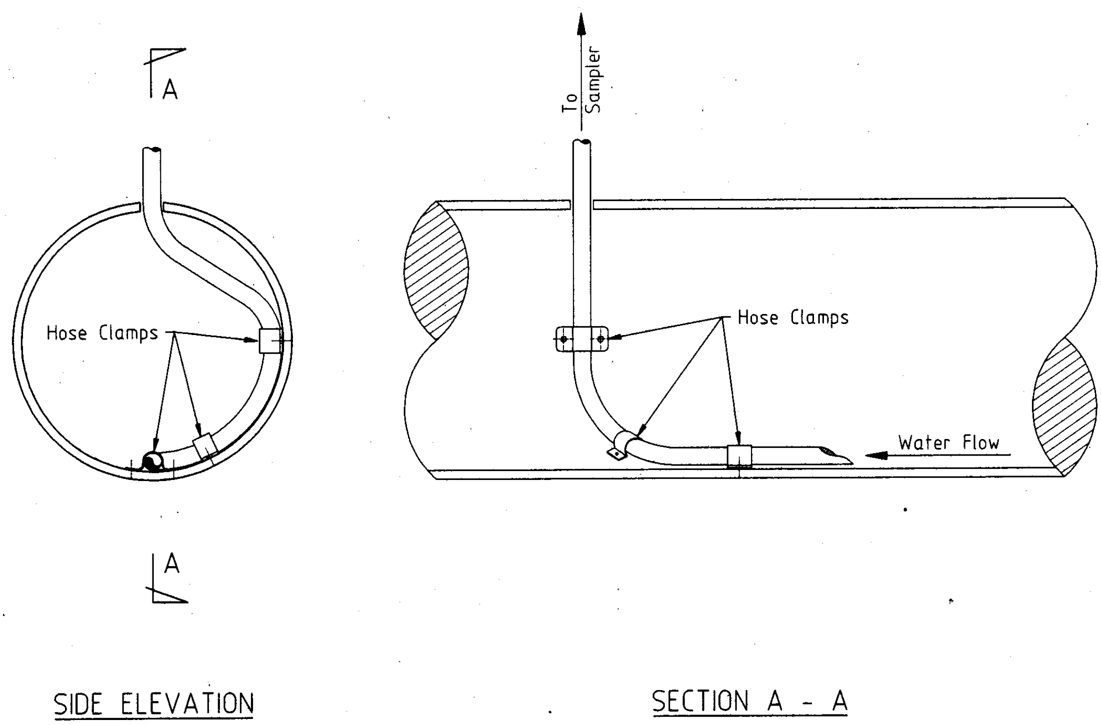


MARK	NOM DIA (mm)	DESCRIPTION	MATERIAL	ENDS	QUANTITY
6	150	CONNECTOR, 500mm LONG (cut to suit)	uP V C	SOCxSOC	1
5	100-150	TAPER	uP V C	SOCxSOC	1
4	100	MAG FLOW METER	C I		1
3	100	UNIFLANGE	uP V C	FLxSOC	2
2	100	22.5° BEND	uP V C	SOCxSOC	1
1	100	CONNECTOR, 500mm LONG (cut to suit)	uP V C	SPxSP	3

FITTING SCHEDULE

The assembled unit is inserted into the pipe and the flow is restricted through the flow meter using appropriately sized tyre inner tubes. In the SLPIP study wheel burrow tyre inner tubes were effectively used for restricting flow when inserted into 225mm and 300mm sewer pipes. If a suitable place to install the flow meter is not available a section can be cut out of the sewer pipe and the flow unit can be inserted as shown in Figure 8.1. Figure 8.2 illustrates how to install a sample hose for an automated sampler. The magnetic flow meter can be set up to give a flow proportional output signal which can be counted by some automated samplers. This is a simple and accurate way of taking flow proportional composite samples.

Figure 8.2 Sample hose installation within a pipe for automated samplers



Because the system restricts the pipe diameter it is important that provisions such as an overflow are provided. The 100 mm magnetic flow meter used in this study was used for lagoon flows ranging from 100 to 400 kL/day plants. Larger flow meters and piping could be used for larger flows, although the system would become more cumbersome to install. The unit used was a 100mm Turbo KS-G Magnetic Flow Meter with a NDFPC2-AT converter which displays the instantaneous flow in L/min and the cumulative volume in kL. (The unit gave trouble free operation over the two year study.)

When selecting a magnetic flow meter it is important to consider the weight, accuracy required, physical dimensions to fit into the sewer pipes being monitored, flow requirements (instantaneous, cumulative, and/or continuous log), the power consumption and whether the meter gives out a flow proportional signal.

The advantages of this unit are:

1. It is very accurate.
2. The system is relatively inexpensive.
3. It is transportable and can be used for other lagoons of similar size.
4. It is easy to install and reliable.
5. The costs of modifying pipe work so the system can be installed is usually minimal.
6. It does not require calibration after it is commissioned (as confirmed by field calibration of the unit over the length of the study). It is recommended that it is calibrated when first commissioned to make sure it has been set up properly.
7. It is not very susceptible to ragging with gross material contained in the raw influent.
8. A contact closure within the magnetic flow meter converter allows for a composite sampler to take a flow proportional composite sample.

The disadvantages of this system are:

1. The power consumption of the system is quite large so a sizeable battery is required if mains power is not available. A 65 amp hour battery supplied sufficient power for 48 hours of operation.
2. There is a head loss through the flow meter unit so this head has to build up upstream of the unit. This back flow has to be allowed for when considering a sample site and the inspection hole levels.
3. The sampling point for the composite sampler needs to be placed upstream of the back flow caused by the flow meter as solids settle out before the flow meter. This settling out also corrupts sample sites down stream of the flow meter.

b) Flow Measurement - using a pump station

A pump station can also be used to give both a flow measurement and a signal for a flow proportional sampling regime.

Flows can be calculated by multiplying the pump hours by the flow rate of the pump. For this method to be accurate the flow rate of the pumps should be consistent and the non-return valves must not leak. The valves are not working effectively if the fluid is observed flowing back down the inlet pipe when the pump is not operating. If the non-return valve is not working effectively it should be fixed as the cost of double pumping effluent is costly and it does lead to over estimates of flow rates into the lagoon system.

The most common method for determining the flow rate of a pump is to measure both the time taken to empty and fill (with influent) a predetermined volume within the pump well. The volume expelled by the pump is calculated by measuring the cross-sectional area of the pump well and the depth change measured for the pump test which may be the depth change for a pump cycle dictated by floats in the well. For inaccessible wells a float on the end of a graduated pole (or rope) can be used to measure the depth change. The pump flow rate is then determined by first dividing the volume of the fluid pumped by the time it takes to empty this volume and adding it to the flow rate entering the pump well for a set time frame. This flow rate entering the pump well is determined by measuring the time taken for the influent to fill a certain volume when the pump is not operating.

These calculations are summarised by:

$$\text{FLOW} = \frac{V_1}{T_1} + \frac{V_2}{T_2}$$

where V_1 = Volume emptied in pump well
T_1 = Time to empty V1
V_2 = Volume filled by influent
T_2 = Time to fill V2

Another method is to time how long it takes for the pump to fill a known volume prior to entering the lagoon system. This is only possible when there is a structure of known volume prior to the lagoon that can be blocked at the outflow. Care must be taken that the filling of the structure does not exert back pressure which would have the effect of decreasing the pump flow rate.

Flow proportional sampling:

A flow proportional sample from a pump station influent can be achieved in a number of ways. Sampling technique is detailed 8.3.3.2 (f). The simplest method is to take a composite sample at set intervals from a sample point that is only under water while the pump is running. The principle of this method is that when the pump is running a sample will be taken and when the pump is idle the automatic sampler will operate but take a dry sample. It would therefore follow that during periods of high flow, with the associated more frequent pump running, there is a greater frequency of non-dry sampling. Therefore as the sample number increases the total sample tends towards being a flow proportional sample. The other technique involves a pulse being generated from a signal generator plugged into a power point at the pump station that only becomes active when the pump is on. The composite sampler takes a sample per set number of recorded cumulative pulses.

8.3.2.2 Dissolved Oxygen (DO)

DO

- is determined in the field using a dissolved oxygen meter and expressed in mg/L or %;
- is required for the respiration of aerobic microorganisms as well as other life forms;
- is used to determine whether the lagoon is operating aerobically, facultatively, or anaerobically;
- profiles of a lagoon on a regular basis will indicate whether a lagoon is well mixed;
- in a lagoon cell can be used to determine when a discharge should occur. (When discharging the DO should be near saturation);
- can be use to determine the extent of oxygen sag occurring in receiving waters as a result of lagoon discharge.

Methods:

The DO meter should be calibrated at least fortnightly or as specified in the manual for the particular meter. To calibrate a DO meter it is necessary to set the DO at 0 % and 100 %. Water which has ready access to air will become 100% saturated. The DO concentration for 100% saturated water does change for different temperatures, but the meters are normally equipped with temperature probes to ensure that any calibration automatically factors temperature in.

Most DO meters have specially designed caps that saturate the probe with a fine film of water on its membrane. If there is no cap, however, 100% saturated water can be made by aerating a beaker of water for a period of time with an airstone on a aquarium air pump or air compressor. Alternatively a jet of tap water for a short time from a garden hose into a bucket of tap water would be a simple way of achieving this. It is also important to check the DO meter reads 0% saturation accurately. To do this it is necessary to put a teaspoon or more of sodium sulphite for every litre of water used. For absolute accuracy when calibrating for 0% also add to the water mixture containing sodium sulphite a few crystals of cobalt chloride as a catalyst (i.e. helper) for the reaction to take place.

It is preferred that DO measurements are taken directly in the lagoon or outlets, i.e. by lowering the probe into the effluent. If the measurement is not taken within the lagoon then a sample needs to be collected in a sample container which has been rinsed with fluid from the target area. In these cases, the sample should be collected as smoothly as possible and measurements made immediately on collection. When taking a DO measurement gently move the probe through the fluid before taking a measurement after the readout on the meter has stabilised.

DO concentrations change during the course of a day, increasing during the day (peaking early afternoon) and decreasing at night to a minimum prior to sunrise. Therefore, the time of day the measurements are taken is important for DO determination. However, it is also important to occasionally take measurements through the day to get an appreciation of the variance that may exist. If the operator detects a downward trend in DO over the day or over several days then it may indicate that the lagoon is going anaerobic and remedial measures such as aeration, sodium nitrate dosing and/or recirculation are necessary.

A very useful measurement for the lagoon operator is a DO profile. This is done by taking a DO reading at a depth of 10cm and then 20cm and so on, until the bottom of the lagoon is reached. If the lagoon is stratified the DO of the lagoon will start dropping as the probe is lowered. This can start happening around 30 cm of depth. If the lagoon is strongly stratified there will be no oxygen in the bottom half of the lagoon (obviously very undesirable in a lagoon which has been designed to operate aerobically). All non-aerated lagoons stratify occasionally, it is how often each lagoon stratifies that is important to the operator and how much of the water column becomes anaerobic. Most lagoons in Tasmania do not stratify much in the winter months.

Refer to the Operating Problem section (8.4.3) and Table 8.3.

8.3.2.3 pH

pH

- is determined in the field with a glass electrode pH meter;
- of the influent is normally between 6.8 and 7.6;
- varies with the natural alkalinity and hardness of the water, and the type and volume of commercial and industrial waste entering the plant;
- is a good indicator of the lagoon's conditions
  - algal reactions can raise the pH above 9.5 as it converts inorganic carbon (carbon dioxide: CO_2) to organic carbon (CH_2O) i.e., new algal cells during photosynthesis
  - high diurnal fluctuations in pH in facultative lagoons may be due to the bicarbonate ion being used for the carbon source for cell carbon
  - pH decreases in lagoons can be due to septic or industrial waste entering the lagoon, algal crashes or sludge inversion
- can decrease during the night due to increased bacterial action and low algal activity. A lower pH near the bottom of the lagoon compared to the top can develop.

Methods:

The pH meter should be calibrated at least fortnightly or as suggested in the manual for the specific meter. To calibrate a pH meter for the expected range of pH found in a lagoon system it is necessary to use a 7 and 9 or 10 buffer. After the pH probe is thoroughly cleaned it is placed in a buffer of known pH and the pH meter is adjusted to read the signal it gets from the probe to be the same as the known pH. A different buffer is then used and the pH meter, after a thorough clean of the probe before immersion in the new buffer, is adjusted to correctly read the known pH of this buffer. This double calibration is often semi-automatically performed by most pH meters. Other pH meters require the slope to be adjusted manually to correctly read the second buffer.

Ideally, measurements should be taken directly from within the lagoon, outlets or waste streams. If the measurement is not taken within the lagoon then a sample needs to be taken in a sample container which has been rinsed with fluid from the target area. The pH level should then be read soon after taking the sample as pH changes considerably with storage. The probe is gently moved through the fluid and the recording of the pH level taken after the readout has stabilised.

As with DO measurements, pH changes during the course of a day, i.e. increasing during the day and decreasing at night. Therefore the time of day the samples are taken is important for pH measurement.

8.3.2.4 Biochemical Oxygen Demand (BOD5)

BOD

- along with chemical oxygen demand and total organic carbon is a measure of the gross concentration of organic matter;
- is a measure of the amount of oxygen required in a 5 day period by the micro-organism in consuming the organic material in the wastewater;
- in domestic sewage ranges from approximately 130 to 250 mg/L;
- indirectly means the organic strength of the wastewater;
- is use to determine, when associated with the flow, the organic load on a lagoon system. (Very informative when compared to the design load);
- is used for evaluating treatment efficiency of the lagoon system;
- is used to determine the impact of the organic strength of the discharge on the receiving waters;
- has several limitations: only biodegradable organics are measured; pretreatment is needed when dealing with toxic wastes; the length of the test before getting results; and the 5 day period may or may not correspond to the point where the soluble organic matter that is present has been used.

Methods:

Samples are taken in 1 L plastic bottles from within the lagoons approximately 2.5 m from the edge of the lagoon embankments or at the inlet to the plant or at the outlets of each lagoon cells. The sample bottles can be new or reused cleaned bottles. The samples are stored at 4°C for up to 24 hours before testing in a laboratory. Refer to section 8.3.3 for more details on sampling.

8.3.2.5 Non-filtrable Residue (NFR)

NFR

- is a measure of the dry weight of solids retained on a glass fibre filter (GFC) and is expressed in mg/L;
- normally in the influent has a value similar to BOD;
- is often difficult to reduce in a lagoon system due to the elevated growth of algae in the systems.

Methods:

Refer to the method for BOD. Normally the sub-sample for NFR determination is taken from the BOD sample.

8.3.2.6 Chemical Oxygen Demand (COD)

COD

- along with biochemical oxygen demand and total organic carbon is a measure of gross concentration of organic matter;
- in domestic sewage ranges from approximately 400 to 700 mg/L;
- is used to measure the organic matter in industrial and municipal wastes that contain compounds that are toxic to biological life;
- is generally higher than BOD because more compounds can be chemically oxidised than can be biologically oxidised;
- can in many types of waste be correlated with BOD in the influent. This can be very useful as the COD can be determined within 3 hours or 24 hours if sent to a laboratory compared to 5 days for BOD measurement. With the correlation established COD can be used more immediately than BOD to assist to control and manage the trade waste entering the treatment plants. However, the lagoon study found poor correlation with total BOD and total COD in the lagoon effluent, and careful examination for any correlation should be made on a case by case basis.

Methods:

Refer to the method for BOD. Normally the sub-sample for COD determination is taken from the BOD sample.

8.3.2.7 Faecal Coliforms

Faecal Coliforms

- indicate the possible presence of pathogens (i.e. disease causing organisms). The source of faecal coliforms can be humans, mammals, and birds.

Methods:

Samples are collected in sterile 250mL bottles from within the lagoons approximately 2.5 m from the edge of the lagoon embankments or at the inlet to the plant or at the outlets of each lagoon cells. The samples are stored at 4°C for up to 24 hours prior to analysis. Refer to section 8.3.3 for more details on sampling.

8.3.2.8 Nitrogen

Nitrogen

- is essential for the growth of plants as is phosphorus and both are referred as nutrients;
- as total nitrogen comprises organic nitrogen (Kjeldahl), ammonia, nitrite, and nitrate;
- as organic nitrogen in the influent is converted to ammonia nitrogen by bacteria as the protein in wastewater is broken down. The ammonia nitrogen is oxidised to nitrite and nitrate which is termed nitrification. The carbon required for the process is derived from the inorganic rather than the organic (BOD) source. Under anoxic conditions (i.e., absence of oxygen) facultative organisms use BOD as the energy and carbon source to reduce nitrate to nitrogen gas which is released into the atmosphere (which contains 79% nitrogen). This is known as denitrification.

Methods:

Samples are collected in new 125mL plastic clean containers which are only used once for nutrient sampling. Sampling occurs at the inlet to the plant and the lagoon cell outlets. A sub-sample from this sample is put in a syringe and filtered through a 0.45 µm filter into a new clean 50mL plastic container. The sample in the 125 mL container is used for determining total Kjeldahl nitrogen while the sub-sample in the 50 mL container is used for determining ammonia-nitrogen, nitrate-nitrogen, and nitrite-nitrogen. The samples can be stored at 4°C while in the field for one to three days and then frozen for up to 180 days prior to analysis. Refer to section 8.3.3 for more details on sampling.

8.3.2.9 Phosphorus

Phosphorus

- as total phosphorus comprises organic phosphorus, orthophosphate, and poly phosphate;
- under aerobic conditions and in the presence of high BOD concentration, is accumulated by certain bacteria as polyphosphate (essentially an energy store) within their cells. This is done by extracting soluble phosphorus from the wastewater. This uptake is possible since the bacteria can consume lower fatty acids under anaerobic conditions at the expense of liberating some phosphate into the water and generate the PHB (poly-hydroxy butyrate), which is used to fuel subsequent phosphate uptake in aerobic conditions;
- in the form of ortho-phosphate is available for biological metabolism without further breakdown, while polyphosphate undergoes very slow breakdown to form ortho-phosphate.

Methods:

The same sample taken for nitrogen is also used for phosphorus analysis. The sample collected in 125mL plastic container is used for determining total phosphorus. While the sub-sample filtered through the 0.45 µm filter into the 50mL plastic container is used to determine ortho-phosphate. Refer to section 8.3.3 for more details on sampling.

8.3.3 Sampling Techniques

Quantitative and qualitative appraisal is achieved through sample collection and flow measurements.

8.3.3.1 Sample Sites

The *sample sites* should be selected as follows:

(a) Influent samples should be collected either at the influent pump station (from a turbulent well mixed point) or from a point prior to the inlet to the plant.

(b) Sampling within lagoons to determine lagoon status, such as pH and DO levels, should be taken at four (or more) locations 2.5m from the waters edge and subsurface (approximately 0.2 - 0.4 m) around the lagoon (avoiding stirring up material from lagoon bottom).

(c) Sampling near or at the outlet to determine lagoon cell performance, i.e. organic and bacterial removal rates should occur (avoiding stirring up material from the bottom).

(d) Effluent should be sampled from the outfall or a well mixed channel in the outfall for plant performance determination.

Samples at the plant inlet (or the last pump station supplying all the influent), the outlets (for determining individual lagoon cell performance, if necessary), and the outfall should have the following tests performed on a regular basis:

- flow (at plant inlet and outfall), temperature, pH, DO, BOD, suspended solids, oil & grease, faecal coliforms, nutrients and chlorophyll “a” (at plant outfall). Also residual chlorine should be tested for in the effluent when disinfecting with chlorine.

These performance parameters not only assist in determining how well a treatment plant is progressing but also predict the necessary operational changes to be implemented to maintain the steady performance of a plant.

8.3.3.2 Sample Collection

The *samples collected* must satisfy the following requirements:

(a) Samples should be representative.

(b) Most stored samples should be refrigerated at 4°C, however nutrient samples should be frozen for storage prior to analysis.

(c) Collection of samples should be into sterile glass bottles for microbiological analyses and clean bottles for the other tests. Samples for BOD, soluble BOD, COD, NFR, and oil & grease (and chlorophyll "a") determination may be taken in washed previously used plastic bottles.

(d) Grab samples, which are single samples taken at no set time or flow, are used for measuring temperature, DO, pH, faecal coliforms, nutrients and residual chlorine. Choosing the sampling time to get a representative sample will require some forward planning.

(e) Samples for temperature, pH and DO should be immediately tested to avoid deterioration, and sampling at the same time each day is recommended as these parameters change through the day with air temperature, sunlight and algal population changes. A high pH and DO is expected in the middle part of the day when the algae are most active. When the algal level/activity is low, the DO and pH is accordingly low.

(f) Composite samples are used to measure BOD and suspended solids. Composite samples are collected by either grab sampling at regular intervals over a selected period of time and mixing proportional to the flow at the time of sampling or more accurately by sampling at a rate which is flow proportional. The flow proportional sampling technique in gravity fed lines and rising mains is as follows:

(i) For gravity fed influent a composite sample involves installing a portable (or permanent) in-line flow meter near the inlet to the plant. The flow meter generates a pulse per set volume of wastewater and the composite sampler counts these pulses and takes a sample per set number of pulses. Refer to Figures 8.1 and 8.2 respectively, for a schematic of the flow equipment and a section of pipe with sample hose configuration which is inserted into existing pipework.

(ii) For influent supplied under pressure from a pump station a composite sampler is either located down stream of the pump-station if the composite sampler takes samples peristaltically or upstream of the pump station if the composite sampler operates under a vacuum (or peristaltically). In either case a pulse is generated from a signal generator plugged into a power point at the pump station that only becomes active when the pump is on. The composite sampler then takes a sample per set number of recorded cumulative pulses.

Refrigeration or cooling with ice of the composite sample is necessary to minimise deterioration of the sample.

(g) Samples for BOD and microbiological determination must be received and prepared for analysis by laboratories within 24 hours. Nutrient samples preferably should be analysed within 24 hours but can be preserved through refrigeration or freezing.

8.3.4 Field Observation

Visual indications through field observation can alert an operator to specific changes in a lagoon system. An odour or change in colour are warnings of a major change and are important field observations for lagoon assessment. As the colour of the lagoon is directly related to pH and DO, the following colours characterise the general condition of the lagoon:

(i) dark sparkling green - good situation; high pH and DO (DO may be above saturation point, 9-10mg/L);

(ii) dull green to yellow - fair situation; pH and DO are dropping;

(iii) grey to black - poor situation; lagoon is septic with anaerobic conditions predominant;

(iv) tan to brown - good situation if due to predominantly a brown type of algae, however not good if due to silt or bank erosion;

(v) pink (patches/whole lagoon) - poor situation; indicates the presence of sulphur reducing bacteria caused by overloading, stratification, or operational deficiencies;

(vi) orange - good situation, usually due to the presence of Daphnia ("water flea") which feed on algae.

When the characteristic green colour of a lagoon begins to change or disappear, the operator should look for things that may be causing it, such as changes in volume, organic load, temperature, light and turbidity. A colour change from green to grey/black, accompanied by floating mats of material from the bottom of the lagoon, usually indicates rapid fermentation of the bottom sediments, frequently as a result of changes in lagoon temperature or in the character of wastewater entering the plant. This situation is normally accompanied by low pH and DO (and potentially odorous conditions).

The surface appearance of a lagoon system is also very informative in regard to operational status. A smooth glassy appearance, which looks heavy or thick, indicates poor DO levels and operational problems. A light wind, which normally creates surface ripples or waves on healthy ponds, will then leave the water surface perfectly calm.

Any changes of pH of the influent can indicate changes in the sewage due to unknown discharges to sewer. Within lagoons, pH levels can indicate significant changes in algal population dynamics and lagoon health status. High pH levels (7 to 10) indicate good levels of algal activity while low pH levels (< 6) could indicate the development of septic conditions and the need for aeration or chemical addition to elevate the DO level.

Other daily data such as weather and water depth of lagoons are important for assessing lagoon activity. Rapid changes in lagoon depth may indicate stormwater infiltration problems or lagoon floor leakage. Records showing such factors as periods of sunshine, cloudiness, weather temperature, period and extent of rainfall, often explain what has happened in the lagoon, the quality of effluent and/or expected storage capacity of the lagoon system.

8.4 Operation And Maintenance Practices

8.4.1 Introduction

Operation and maintenance programs are an essential part of "good housekeeping" practices and optimising lagoon performance such that the lagoon effluent complies with the discharge requirements. In establishing programs consideration should be given to:

- frequency of operator visits (minimum weekly)
- control of emergent vegetation
- control of floating sludge mats
- removal of non-degradable wastes
- addressing odour problems
- removing water weeds
- determining sludge build up
- observation of lagoon appearance and weather conditions
- in-field testing
- sampling regime.

8.4.2 Aerobic, facultative, aerated and anaerobic lagoon systems

For the operation and maintenance of aerobic, facultative and aerated lagoon systems the following information applies:

8.4.2.1 Aerobic and facultative lagoons

(a) The primary lagoon should have a deep green sparkling colour which indicates high pH and DO;

(b) the secondary or final lagoons should have high DO and the effluent from the plant should comply with the Environment Protection (Water Pollution) Regulations 1974 (or Tasmanian Sustainable Development Water Quality Management Policy requirements when finalised) and/or the requirements for the land application option;

(c) wave action on the surface of lagoon should be present when windy, if this is not the case then anaerobic conditions or an oily surface may exist;

(d) no weed growth in the water or tall weeds on the bank should exist as this inhibits wave action and re-aeration;

(e) wave walls should be well seeded above the water line with grass and kept mowed to prevent soil erosion and insect problems;

(f) erosion of wave walls should be prevented at waters edge by the use of rubble or poured concrete;

(g) inlet and outlet structures are to be kept clean. No floating debris, caked scum, or other trash that might produce odours or be unsightly should exist;

(h) lagoons should be inspected and maintained on a weekly basis (minimal);

(i) sheep or goats may be used to manage grass around lagoons. These animals are to be tagged for easy identification if mixed with other stock at some time. Such animals are not recommended for human consumption.

8.4.2.2 Aerated lagoons

(a) The above information for aerobic and facultative lagoon systems applies for aerated systems;

(b) these lagoons require daily inspections and maintenance as special attention is required for the aeration equipment;

(c) a minimum of 1mg/L of DO should be maintained throughout the lagoon at heaviest loading periods (monitor DO at least at the outlet daily);

(d) the mechanical aerators should produce good turbulence and a light amount of froth;

(e) for diffused air systems, that use a blower and pipelines to diffuse air over the entire bottom of lagoon, check blower daily and also check aeration patterns for "dead spots" or line ruptures to ensure even distribution of air. Also test the DO level at several points weekly;

(f) periodic maintenance must be performed, such as lubrication, adjustment and replacement to a written schedule (refer to Table 8.1) and records are to be kept on these activities.

(d) Grab samples, which are single samples taken at no set time or flow, are used for measuring temperature, DO, pH, faecal coliforms, nutrients and residual chlorine. Choosing the sampling time to get a representative sample will require some forward planning.

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(e) wave walls should be well seeded above the water line with grass and kept mowed to prevent soil erosion and insect problems;

(f) erosion of wave walls should be prevented at waters edge by the use of rubble or poured concrete;

(g) inlet and outlet structures are to be kept clean. No floating debris, caked scum, or other trash that might produce odours or be unsightly should exist;

(h) lagoons should be inspected and maintained on a weekly basis (minimal);

(i) sheep or goats may be used to manage grass around lagoons. These animals are to be tagged for easy identification if mixed with other stock at some time. Such animals are not recommended for human consumption.

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(d) the mechanical aerators should produce good turbulence and a light amount of froth;

(e) for diffused air systems, that use a blower and pipelines to diffuse air over the entire bottom of lagoon, check blower daily and also check aeration patterns for "dead spots" or line ruptures to ensure even distribution of air. Also test the DO level at several points weekly;

(f) periodic maintenance must be performed, such as lubrication, adjustment and replacement to a written schedule (refer to Table 8.1) and records are to be kept on these activities.

8.4.2.3 Anaerobic Lagoons

(a) No DO should be present in these lagoons;

(b) the lagoon ideally has a capping of scum over the entire surface to maintain the anaerobic conditions and minimise the release of offensive odours;

(c) maintain the lagoon at approximately pH 7;

(d) for proper operation of the lagoon the following parameters must be determined: BOD_5, Alkalinity, Volatile Fatty Acids (VFA), and pH. BOD_5 reduction should be between 60 and 80%, the alkalinity 1500 to 3000 mg/L, VFA 50 to 250 mg/L, VFA/Alkalinity should be less than 0.25 and the pH between 6 and 8.

An appropriate check list should be compiled which details the operation and maintenance programs. The lagoon operation should be scheduled and plant records should detail weather data and basic test results such as pH, DO, BOD, NFR, oil & grease, faecal coliforms, and nutrients (and levels of free chlorine residual, if applicable). Also equipment maintenance regimes are to be scheduled. Refer to Tables 8.1 and 8.2 for examples of the checklist document and the "schedule organiser".

Table 8.1 Operation and maintenance check list (example)

Summary of issues in an Operation and Maintenance Check List (example)

1. Lagoon Inspection

2. Maintenance Issues
   - Lagoon (eg., mowing, fencing, clearing channels, removing non-degradables and scum rafts,)
   - Equipment (eg., pump stations, comminutor, aerator, flow measuring devices, values)

3. Monitoring Program
   - field: weather, rainfall, flows, DO, pH, temperature,
   - sampling: BOD, NFR, faecal coliforms, oil & grease, nutrients, chlorophyll "a".

8.4.3 Operating Problems

Table 8.3 details options for the solution of particular problems typically experienced with lagoon systems. The table has been adapted from the USA EPA Operations Manual. The control of odour will be briefly discussed here.

A number of situations in lagoon systems can give rise to odour. Decaying mats of algae blown to corners or banks and sludge rafts from the bottom of lagoons during inversion can give rise to offensive odours. Shock overloads can cause anaerobic conditions to develop in most of the lagoon which eventuates in methane and hydrogen sulphide gases being given off, causing nuisance odours.

The solution to the mat problem is immediate dispersion through agitation with a water jet or aerator or removal through a "vacuum-truck". With anaerobic conditions the addition of ammonium nitrate (or sodium nitrate) will assist in restoring aerobic conditions. The use of an aspirator is also very effective in creating aerobic conditions.

Nitrate is normally added at 112 kg/ha or 20g/kL for the first day and 56 kg/ha every day there after until the odour is effectively managed. A side reaction to this addition is it may cause sludge to rise to the surface. It should be realised that this addition of nitrate is only a temporary measure and a long term solution needs to be implemented such as desludging and/ or the installation of an aspirator.

Table 8.2 Schedule Organiser (example)
(Adapted from the USA EPA Operations Manual)

	Frequency					
	Daily	**Wkly**	**Mon.**	**3Mon.**	**Yrly**	**As Needed**
(A) Lagoon Inspection:						
• lagoon surface characteristics	X					
colour						
wind action						
scum build						
algal rafts						
• water weed	X					
• lagoon depth	X					
• odour	X					
• lagoon bank integrity	X					
• grass status on banks and surrounding area	X					
(B) Lagoon Maintenance						
• clean inlets, lagoon wave walls,	X					
• break up or remove scum rafts						X
• mow embankment and surrounding area						X
(C) Monitoring						
• weather conditions (including rainfall, ambient temperature, wind direction, cloud cover)	X					
• influent and effluent quantity	X					
• influent and effluent quality (and load determination)						
-field measurements (DO, pH, lagoon temperature, colour)	X					
-sampling for laboratory analysis (BOD, NFR, oil & grease , faecal coliforms, nutrients). Determining Chlorophyll "a" in effluent.	X					
• within lagoon field measurements (DO, pH, lagoon temperature, colour)	X					
(D) Mechanical Equipment						
• Pump Stations						
- remove debris	X					
- check operation	X					
- log times	X					
- clean floats and other control devices		X				
- lubricate						X
• Comminutor						
- check		X				
-lubricate					X	
• Aerators						
- log running time		X				
- lubricate						X
• Flow measuring devices						
- clean		X				
- verify accuracy			X			
• Valves						
- check			X			

Table 8.3 Possible solutions to problems in lagoon systems

PROBLEMS	OBSERVATIONS	CAUSES	SOLUTIONS
AERATED LAGOONS - fluctuating do - floc in final cell - frothing & foaming	Fluctuating DO, fine pin floc in final cell effluent, frothing and foaming.	Shock loading, over-aeration, industrial wastes.	1. Control aeration system by using timer to allow operation during high load periods, monitor DO to setup schedule for even operation, holding approximately 1mg/L or more. 2. Vary operation of aeration system to obtain solids that flocculate or 'clump' together in the secondary cell. 3. Locate industrial wastes that may cause foaming or frothing and eliminate or pretreat wastes.
ALGAE IN EFFLUENT	Elevated suspended solids in the lagoon effluent can be attributed to algae.	Favourable weather, temperature and lagoon conditions	1. Draw off effluent from below the surface by use of a good baffling arrangement. 2. Use multiple ponds in series, e.g., 5-6 ponds to effectively reduce loading in these last lagoons. 3. The use of intermittent sand filters and submerged rock filters. 4. The additional calcium oxide (<100mg/L), iron sulphate and alum (at approx. 20mg/L: a level not considered toxic).
ANAEROBIC CONDITIONS	Facultative pond that turned anaerobic resulting in high BOD, suspended solids and scum in the effluent. Unpleasant odors, the presence of filamentous bacteria and grey colour and heavy surface indicate anaerobic conditions.	Overloading, short circulating, poor operation or toxic discharges.	1. Change from a series to parallel operation to divide load. 2. Add supplementary aeration if lagoon is continuously overloaded. 3. Change inlets and outlets to eliminate short-circuiting. Refer to section on "short-circuiting". 4. Recirculation to provide oxygen and mixing. 5. Temporary relief can be potentially achieved with an additional ammonia or sodium nitrate at rates described under the section on "odor control". 6. Eliminates sources of toxic discharges.
ANAEROBIC LAGOONS - ODOUR - PH	Odors Hydrogen sulfide (rotten egg) odors or other disagreeble conditions due to sludge in a septic condition.	Lack of cover over surface possibly due to insufficient load to form scum blanket.	Spread straw over surface or some other cover until good surface has developed.
	Low pH pH below 6.5 accompanied by odour as a result of acid bacteria operating under anaerobic conditions.	Acid formers working faster than the methane formers in an acid condition	Raise the pH using hydrated lime (lime slurry) at 120g/10000 L. The slurry should be mixed while adding to the inlet of the lagoon.
BLUE-GREEN ALGAE	Low PH (less than 6.5) and dissolved oxygen (less than 1 mg/L). Foul odors develop when algae die off.	Blue-green algae is an indication of incomplete treatment. Overloading and/or poor nutrient balance.	1. Apply 3 applications of a solution of copper sulfate. a. If the total alkalinity is above 50 mg/L apply 1200 mg/m^3 of copper sulfate. b If alkalinity is below 50mg/L reduce the amount of copper sulfate to 600 mg/m^3 **NOTE:** Copper sulfate concentrations greater than 1mg/L is toxic to certain organisms and fish. 2. Break up algal blooms by portable pump, hose and aerator

Table 8.3 Continued

PROBLEMS	OBSERVATIONS	CAUSES	SOLUTIONS
BURROWING ANIMALS	Burrowing animals can cause damage to embankments/walls through weakening the structure.	Bank conditions attract such animals and there are population in area adjacent to lagoons.	1. Remove food supply from lagoons and adjacent areas. 2. If problem persists check with Parks and Wildlife Division, if dealing with native animals, for methods of removal, such as live trapping.
HIGH BOD IN EFFLUENT	High BOD concentrations	Short resident time, poor inlet and outlet placement induced short circuiting, high organic or hydraulic loads, toxic compounds, high algae levels	1. Infiltration investigation of sewer 2. Recirculate the lagoon effluent with portable pump 3. Alter inlet and outlet arrangement 4. Reduce influent loads through waste stream investigation and implementation of waste minimisation strategy. 5. High algae reduction program. Refer to “algae in effluent section”
INSECTS	Insects present in area and larvae or insects present in pond water.	Poor circulation and maintenance.	Mosquito Control 1. Keep lagoon clear of weeds and allow wave action on banks to prevent mosquitoes from hatching out. 2. Keep lagoon free from scum. 3. Spraying with larvacide as a last resort. Discussion issue with DELM prior to spraying. Midges Control 1. Spay with approved insecticide. Discuss issue with DELM prior to spraying.
LIGHTLY LOADED LAGOONS	Lightly loaded lagoons may produce filamentous algae and moss which limits sunlight penetration	Over design, low seasonal flow.	1. Correct by increasing the loading or reducing the number of cells in use. 2. Use series operation if presently run in parrallel
LOW DISSOLVED OXYGEN (DO)	A low, continued downward trend is Do is indicative of possible impending anerobic conditions and the cause of unpleasant odors.. Treatment effectiveness reduces.	Poor light penetration, low wind action, low residence time, high BOD loading or toxic industrial wastes.	1. Remove waterweed on lagoon 2. Reduce organic loading to primary cell(s) by going to parallel operation. 3. Add supplemental aeration (surface aerators(aspirator)/diffusers) 4. Recirculate final effluent to the plant inlet. 5. Apply sodium or ammonium nitrate. Refer to the section on “odour”. 6. Determine if overload is due to industrial source and if so manage it.
ODOURS	Odours are a general nuisance and can be offensive to the public.	The odors are a result of over loading, long periods of cloudy weather, poor lagoon circulation, industrial wastes or sludge inversion.	1. Use parallel feeding to primary cells to reduce loading. 2. Apply chemicals such as sodium nitrate/ammonia nitrate at a rate of 112kg/hectare for first day, then 56kg/hectare per day thereafter if odours persist. Mix in if possible. 3. Install supplementary aeration such as floating aspirator, caged aerators, or diffused aeration to provide mixing and oxygen. Note: odours may be initial worse for short periods, but the total length of odorous periods will be significantly reduced. 4. Recirculate pond effluent to the pond influent to provide additional oxygen and to distribute the solids concentration. Recirculate on a 1 to 6 ratio. 5. Eliminate/manage septic or high-strength industrial wastes.

Table 8.3 Continued

PROBLEM	OBSERVATIONS	CAUSE	SOLUTIONS
OVERLOADING	Overloading results in incomplete treatment of the wastewater. Overloading problems can be detected by offensive odors, grey colour, low pH and DO, and excessive BOD load per ha.	Short-circulating industrial wastes, under design, infiltration and weather conditions.	1. Bypass the cell and let it rest. 2. Use parallel operation 3. Apply recirculation of pond effluent 4. Look at possible short-circulating. 5. Install supplementary aeration equipment.
pH - DECREASING	pH should be on the alkaline side, preferably about 8.0 to 8.4 Both pH and DO will vary throughout the day with lowest reading at sunrise and highest reading in late afternoon. Measure pH at the same time each day and plot on a graph.	A decreasing pH is followed by a drop in DO as the green algae die off. This is most often caused by overloading, long periods of adverse weather or higher animals, such as Daphia, feeding on the algae.	1. Bypass the cell and let it rest 2. Use parallel operation 3. Apply recirculation of pond effluent. 4. Check for possible short-circulating. 5. Install supplementary aeration equipment if problem is persistent and due to overloading. 6. Look for possible toxic external causes of algae die-off and correct at source.
SHORT-CIRUITING	Odor problems, low DO in parts of the lagoon which are not receiving good circulation, anaerobic conditions and low pH found.	Poor wind action due to trees, poor arrangement of inlet and outlet locations, poor lagoon shape, weed growth, or irregular lagoon floor.	1. Cut trees and growth at least (150m) away from lagoon if in direction of prevailing wind. 2. Install baffing around inlet location to improve distribution. 3. Add recirculation to improve mixing. 4. Provide new inlet-outlet location. 5. Clean out weeds. 6. Level irregular lagoon floor
SURFACE SCUM	To manage potential odour problems and to eliminate breeding spots for mosquitoes it is necessary to control scum formations. Also, sizeable floating rafts will reduce sunlight and disinfection.	Lagoon bottom is turning over with sludge floating to the surface (inversion). Poor circulation and wind action. High amounts of grease and oil in influent will also cause scum.	1. Use rakes, a portable pump to get a water jet to break up scum formations. Broken up scum usually sinks but odour will be initially released. 2. Any remaining scum should be skimmed and disposed of by burial or removed to an approved RDS approved by DELM.
VEGETATION ON EMBANKMENT	High weed growth, brush, trees and other vegetation will provides nesting places for animals, which can potentially cause weakening of the embankment. Will affect wind action on the lagoon and performance.	Poor maintenance	1. Periodic mowing is the best method 2. Sow embankments with a mixture of fescue and blue grasses on the bank and short native grasses elsewhere. It is desirable to select a grass that will form a good sod and drive out tall weeds by binding the soil and “out compete” undesirable growth. 3. Spray with approved weed control chemicals. All chemicals are to be bio-degradable. 4. Use small animals, such as sheep to graze grass. Care is needed to not increase the faecal coliform in the final lagoon cell. Rotate the grazing to prevent destroying individual grass species.
WATER WEED	Weeds provide food for burrowing animals, cause short-circulating problems, stop wave action so that scum collects and results in mosquito infestation and odours. Water weed inhibits sunlight penetration and interferes with wind action thus reducing the oxygen transfer in the lagoon. Root penetration can cause leaks in pond seal.	Poor circulation, maintenance, insufficient water depth.	1. Pull weeds by hand if new growth. 2. Mow weeds. 3. Lower water level to expose weeds, then burn. 4. Increase water depth to above tops of weeds. 5. To control waterweed, use rakes or offer means to physically remove from pond.

8.5 Effluent and Sludge Disposal

8.5.1 General

The management of effluent and sludge is dependent on information generated from analysis through the monitoring programs mentioned above. The quality requirements for liquid and solid waste streams as related to the final disposal options need to be established.

8.5.2 Sludge

Desludging of lagoons is not required on a regular basis, perhaps every 10 to 15 years. Possibly one of the major problem to be faced when dealing with sludge removal is economic dewatering of the sludge on site so that only a minimum amount of material has to be disposed of or removed from the site.

In accordance with Environment Tasmania's hierarchy of waste management, expressed in Chapter 5, re-use and recycling of sludge should be investigated and implemented where possible. Sludge re-use can be achieved by firstly composting the sludge by mixing with a carbon source. Sludge may compost itself if left for sufficient time in drying beds. When implementing sludge re-use, requirements of the Tasmanian Effluent Reuse Coordinating Group (CG) must be adhered to. The CG has representatives from Department of Environment and Land Management, Department of Primary Industry and Fisheries, Department of Health and Community Services and Department of Mines.

The New South Wales EPA's "Environmental Management Guidelines For The Use and Disposal Of Biosolids Products" (August 1995) should also be used as a reference when considering sludge management.

Sludge which is not re-used or recycled should be disposed of via landfill with the approval of the Director of Environmental Management and the local authority.

8.5.3 Effluent

Environment Tasmania encourages the reuse of effluent discharged from sewage lagoon systems onto land provided that such application is sustainable and feasible. To optimise the utilisation of the wastewater resource and minimise the environmental impact the CG released "Guidelines For Reuse Of Wastewater In Tasmania"(June 1994). The CG is involved with any approval of effluent reuse programs for Level 2 (Scheduled) sewage treatment plants.

If reuse proves to be unfeasible then discharge to receiving waters would be accepted provided that such discharge has no detrimental effect on the protected environmental values (PEV) of the receiving waters. The PEV classifications are detailed in Chapter 5 and were defined by the "Australian Water Quality Guidelines For Fresh and Marine Waters, November 1992". PEVs need to be identified on a case by case basis. The (draft) Tasmanian Sustainable Development Policy on Water Quality Management provides a mechanism for establishing ambient water quality objectives based on environmental values.

8.6 Wastewater Minimisation

As an ongoing refinement and optimisation of lagoon system performance it is beneficial to ensure that the waste streams identified when designing the system (and any further significant connections to sewer) undergo waste management programs. Such programs should extend the life of the lagoon system as well as improve the public and industrial management of water.

Waste minimisation programs more commonly tend to be implemented as a result of identifying a problem in the lagoon system and determining the cause/s. The monitoring of wastewater quality and flow is useful for suggesting possible causes for poor performance. The flow monitoring permits routine checking of the theoretical hydraulic retention times through the system and the identification of abnormal peak flows.

When a problem has been identified the cause/s should be determined rapidly. Wherever hydraulic loading is a problem, input through infiltration (detection may be possible with smoke injection into the sewer lines) and/or industry discharge should be assessed.

When investigating industries connected to sewer through monitoring programs, it is necessary to check that their waste management strategies are in accordance with their trade waste agreements. Waste minimisation in industry can be achieved through recycling and reuse, segregation of waste streams, reduction of quantity and concentration of wastewater, upgrading with modern accepted technology and product recovery.

8.7 References and Bibliography

1. USA ENVIRORONMENTAL PROTECTION AGENCY, *”Operations Manual Stabilisation Ponds”,* Chuck Zickefoose & RB Haynes August 1977.

2. TASMANIAN DEPARTMENT OF ENVIRONMENT AND LAND MANAGEMENT, *“Draft State Policy, Setting New Standards For Water Quality”,* Government Printer 1996.

3. APHA - AWWA - WPCF, *“Standard Methods for the Examination of Water and Wastewater”,* APHA - AWAA - WPCF 1995 (Ed 19)

4. TASMANIAN DEPARTMENT OF ENVIRONMENT AND LAND MANAGEMENT, *“Guidelines for Reuse of Wastewater in Tasmania”,* Government Printer 1994.

5. NSW EPA, *“Environmental Management Guidelines for the Use and Disposal of Biosolid Products”* August 1995

6. ANZECC NATIONAL WATER QUALITY MANAGEMENT STRATEGY, *“Australian Water Quality Guidelines for Fresh and Marine Waters”,* ANZECC, 1992.

CHAPTER 9

CLICK FOR INSTRUCTIONS AND PONDCAL SOFTWARE

Contents

Figures

9.1 Introduction

The Pondcal program has been devised as part of the study to assist in lagoon design, operation and management for Tasmanian conditions. The program uses the modified Marais model described in Chapter 6 to assess and calculate bacterial performance. It runs in Microsoft Excel under Microsoft Windows. These instructions detail how to install and operate the program as well as interpret the data it generates.

The Pondcal program allows for the input of design parameters and measurements taken from the lagoon to make performance predictions. The data required to be entered into the program are daily influent flows, rainfall events, influent BOD, surface areas and average depths of cells, bacterial counts and stratification status of the lagoon system. The program will then calculate the organic loads on the primary cell, wet weather and dry weather flows, hydraulic retention times, predict bacterial performance. and calculate actual bacterial performance.

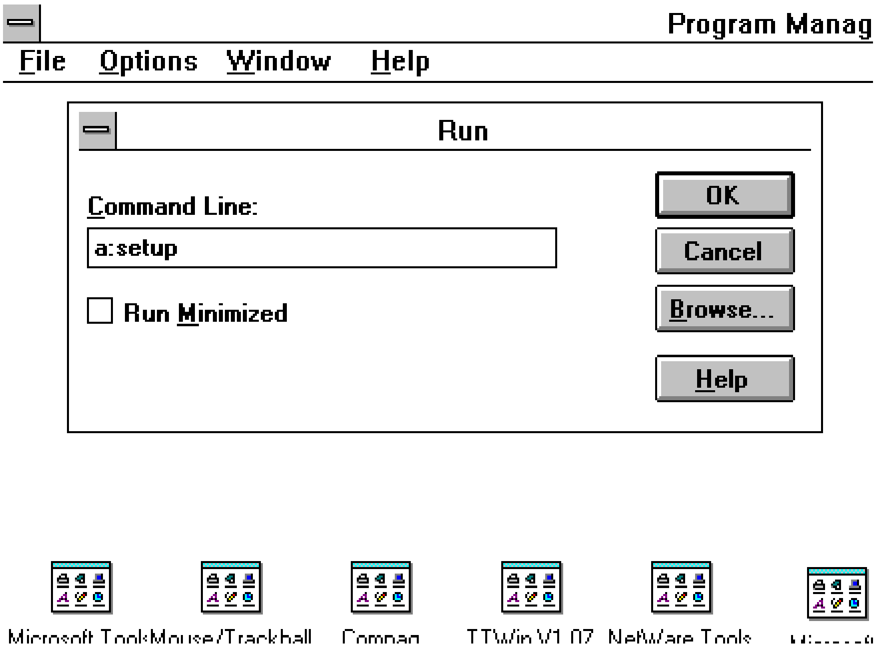
Once the relevant data for the lagoon have been entered, different design and load scenarios can be substituted into the program to investigate different options available to improve lagoon performance.

9.2 Installation

Users must purchase and install their own copies of Microsoft Windows (3.1 or higher) and Microsoft Excel (version 4.0 or higher).

The Excel template and corresponding macro can be installed by simply running a set up program from the program manager within windows. This is done by inserting the 3.5 inch disc labelled Pondcal in drive A or B and choosing the file menu from the top left hand corner of the Windows Program Manager and selecting run. The dialogue box as shown in Figure 9.1 should now be visible. If your drive is Drive A, type **a:setup** in the command line box and if it is Drive B, type **b:setupb** and then press OK. The set up program automatically places the Pondcal template and corresponding macro into a directory named lagoon on the c drive of the computer.

Figure 9.1. The run dialog box in windows version 3.1.



9.3 Running Pondcal

It is necessary to run the Pondcal template through Excel 4.0 or a later version. On running the program open the directory "lagoon" on the c drive and select Pondcal.XLT. Immediately save the template with another name before entering data. The template should be left in the lagoon directory to allow the macro's triggered from within the template to still operate.

9.4 Entering and Interpreting Data

Once the template has been opened it is then a simple task of entering data obtained from the pond system. The template will calculate design information and also allow for prediction of performance under different design and load scenarios. By varying changes in flows, mixing states and design of the lagoon system the expected results in performance can be observed. This information is of use to managers, operators and designers of lagoon systems.

Data entry is restricted to white cells and dialog boxes within the template. On a colour monitor the light blue cells show data calculated from the data entered. It should be remembered that sufficient numbers of samples to generate a reasonable estimate of loads should be entered. Low numbers of measurements (e.g. 5 or less) may lead to significant errors.

Pond size data is entered in the table below in Figure 9.2. The pond size of existing lagoons should be calculated from a survey of the lagoons as the original design specifications often differ from the actual size. The base area is the area of the bottom of the lagoon excluding the batter (slope of the banks). This entry allows the program to take into account the loss of lagoon volume due to batter at the edge of the lagoon. If this is left blank, the program assumes the wall of the lagoon is vertical. The depth of the lagoon should also be measured manually to allow for deviations from the design and sludge build up. The depth entered should be the average depth of the lagoon which takes into account sludge build up.

The equivalent people estimate (EP) is an approximation of the number of people connected to the sewer system. Additional loads from commercial and other sources are also estimated as equivalent to a certain number of people.

Figure 9.2 Data entry for pond size

Lagoon data

Cell	Surface Area (Hectares)	Base Area (Hectares)	Depth (m)
Primary	1.035	0.91	0.9
Secondary 1	0.533	0.42	0.9
Secondary 2	0.802	0.72	0.9
Secondary 3			
Equivilent people estimate		1500	

The data for measured flows and inlet concentrations is entered into the table as detailed in Figure 9.3. These data allow loads on the lagoons to be quantified. Additionally, the effect of infiltration on the system is also quantified from data entered in this table.

The column for number of days is simply the number of days over which the flow measurements were taken. The flow measurements should be taken strictly over 24hr periods. If BOD and NFR samples are taken, the flow measurement over the 24 hour period the sample was taken should be entered.

The total flow is the flow in kilolitres over the total number of days the flow measurement was taken. It is **not** the average flow per day.

The “Recent Rain” column allows for dry and wet weather flows to be calculated. If the rain status is unknown simply leave this column blank and the flow measurement will only be used to calculate average flow and not the dry and wet weather flow calculations. It is necessary that the “Recent Rain” column is only filled in for data obtained over equal time periods.

If BOD and NFR samples are to be taken they should be composited over a 24 hr period while a flow measurement is being taken. This allows for fluctuations in concentrations that occur throughout the day. If possible the sampling should be flow proportional using an automatic sampler. However, if sampling is being carried out manually more samples over the 24 hour period should be taken in the mornings and evenings to take into account the increases in flow over these time periods. Alternatively, increase sample volume proportional to the flow taken at set times through the 24 hours period.

Figure 9.3 Data entry table for flow and load measurements

15	Influent Flow Measurements						
16							
17	Date	Measurement	Total Flow	Recent	BOD	NFR	E.Coli
18	dd-mm-yy	Period (Days)	KL	Rain (Y or N)	mg/L	mg/L	No.
19	10/05/95	1	345	N	214	240	7300000
20	12/06/95	1	389	Y			
21	14/07/95	1	401	Y	200	200	9000000
22	7/08/95	7	2386				
23							
24							
25							
26							
27							

In Figure 9.4, the average flow per day and the average concentrations of BOD, NFR and E.coli are presented in the bottom row of the table. If the data entry table for flow and load measurements is filled simply click on the unhide button and the table is extended. The hide button returns the table back to its original size.

Figure 9.4 Buttons to extend the data entry table for flow and load measurements

34							
35							
36							
85							
86	AVERAGE/DAY		352		207	220	8150000
87	UNHIDE		HIDE				
88							

A flow summary calculated from the previously entered data is presented in Figure 9.5. This data is critical to the design and management of the lagoon system.

Figure 9.5 Flow and load summaries

90	Flow Summary	
91		Value
92	Average flow (kL/Day)	352
93	Average flow per E.P. (L/EP.d.)	235
94	Average dry weather flow (kL/Day)	345
95	Average dry weather flow / E.P. (L/EP.d.)	230
96	Wet weather Flow (kL/Day)	395
97		
98	Load Summary	
99		Value
100	BOD LOAD (kg/Day)	77
101	BOD LOAD Primary (kg/Ha.d)	74
102	BOD LOAD Per E.P. (g/E.P.)	51
103	NFR LOAD (kg/Day)	82
104	NFR LOAD Primary (kg/Ha.d)	79
105	NFR LOAD Per E.P. (g/EP.d)	54
106	Average E.coli (No./100ml)	8150000
107	HRT (Theoretical) Primary (Days)	25
108	HRT (Theoretical) Secondary 1 (Days)	12
109	HRT (Theoretical) Secondary 2 (Days)	19
110	HRT (Theoretical) Secondary 3 (Days)	
111	HRT (Theoretical) Total (Days)	56
112	HRT = Hydraulic retention time	

The average flow is the most critical figure for the lagoon design. The above calculations in the table may give indications of how this figure may be improved.

The average flow per EP gives an indication of whether the amount of flow per person is high. If this figure is around or below 240 L/EP.d it suggests that the sewer flow is reasonable for the population although with good waste minimisation and good control of infiltration 180 L/EP.d is achievable. Refer to Chapter 6 for more details on flow/EP.

The average dry weather flow should be compared to the average flow. If it is low, the difference between the flows will dictate the scale of the infiltration problem that exists. These figures can be used also to calculate how much money infiltration is costing the council in extra pumping, maintenance and treatment costs.

The average dry weather flow per EP gives an indication of water usage. If it is much higher than 180 - 200 L/EP then high water usage in the community may be a problem or there may be permanent infiltration. If it is not due to permanent infiltration or high commercial use a community water minimisation strategy should be implemented.

Wet weather flow is another indicator of infiltration and should be compared to the average and dry weather flows.

The Load Summary is used for lagoon design. Of particular interest is the BOD load on the primary. If the load is around 70 kg/ha.d or above, as in this example, then the potential for odour problems is significant for Tasmanian lagoons.

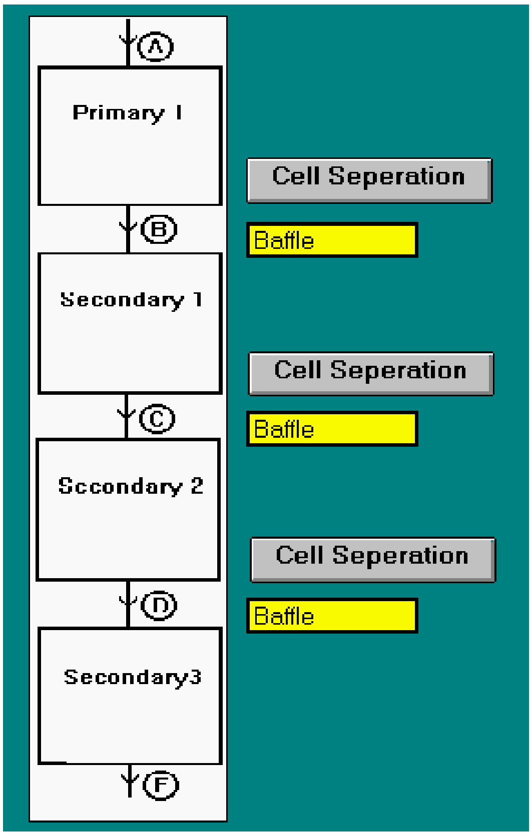
The bacterial data entry table in Figure 9.6 is used to enter bacterial data collected from the lagoon system. It is important that the data is placed in the correct column according to the position of the sample site within the lagoon system. When no data are available for a sample site, simply leave the cell blank so the data are not taken into account when the average is calculated. If required the table may be extended using the hide and unhide commands. A schematic showing the appropriate sample site identification (A,B,C,D or E) is presented in Figure 9.7.

Figure 9.6 Bacterial data entry table

114	E. coli DATA					
115						
116–118	Date dd-mm-yy	Primary Influent (A) E.coli/100ml	Primary Effluent (B) E.coli/100ml	Secondary 1 Effluent (C) E.coli/100ml	Secondary 2 Effluent (D) E.coli/100ml	Secondary 3 Effluent (E) E.coli/100ml
119	10/05/95	7300000	163000	18000	1190	
120	12/06/95		150000	15000	1000	
121	14/07/95	9000000	172000	21050	1400	
122						
123						
124						
125						
126						
127						
128						
185						
186	AVERAGE	8200000	160000	18000	1200	
187		Note: < or > not to be used in entries (unless you don't want the value				
188		to be used in the average).				
189	UNHIDE		HIDE			
190						

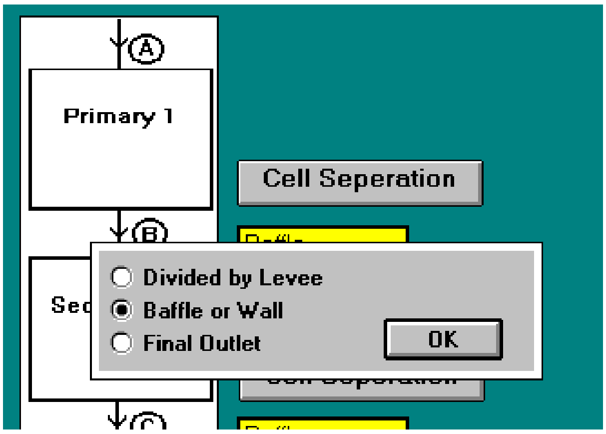
Figure 9.7 shows a schematic of a lagoon system with sample site symbols as presented on the Excel template. The button at the end of each cell allows for the type of separation between each cell to be entered. This information is critical as it is taken into account in calculations of expected bacterial kill rate performance.

Figure 9.7 Schematic of pond layout



When the cell separation buttons are selected the dialog box as shown in Figure 9.8 appears. It is critical that the right option is selected. If there is a levee between the cells with a change in water level such that the flow is in one direction then the "Divided by Levee" option should be chosen. For any other baffle, wall or levee that **does not** change the water level and allows for bi-directional flow the "Baffle or Wall" option should be chosen. If it is the end of the lagoon system choose the "Final Outlet" option. Once the appropriate selection is made click on the OK button to return to the template.

Figure 9.8 Schematic of pond layout

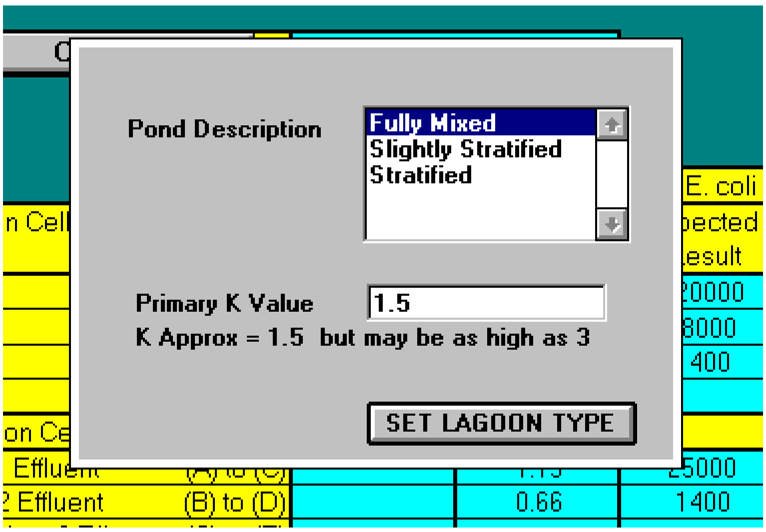


The final table in the template is presented in Figure 9.9. For the calculations in this table to be correct both the “Lagoon Type” and “Cell Separation” selections are critical. To change the “Lagoon Type” select the change button as shown in Figure 9.9. The dialog box as presented in Figure 9.10 will appear.

Figure 9.9 Bacterial performance table

196	E.coli Performance					
197						
198	Lagoon Type	CHANGE	FULLY MIXED			
199						
200						
201			Kill Rates		E. coli No./100ml	
202	Single Lagoon Cells		Expected k Value	Actual k Value	Expected Result	Actual Result
204	Primary	(A) to (B)	1.5	2.02	210000	160000
205	Secondary 1	(B) to (C)	0.6	0.65	19000	18000
206	Secondary 2	(C) to (D)	0.6	0.72	1400	1200
207	Secondary 3	(D) to (E)	0.6			
208	Two Lagoon Cells in Series					
209	Primary Influent to Secondary 1 Effluent	(A) to (C)		1.17	26000	18000
210	Primary Effluent to Secondary 2 Effluent	(B) to (D)		0.68	1500	1200
211	Secondary 1 Effluent to Secondary 3 Eff	(C) to (E)				
212	Three Lagoon Cells in Series					
213	Primary Influent to Secondary 2 Effluent	(A) to (D)		0.96	2000	1200
214	Primary Effluent to Secondary 3 Effluent	(B) to (E)				
215	Four Lagoon Cells in Series					
216	Primary Influent to Secondary 3 Effluent	(A) to (E)				
217						

Figure 9.10 Pond description dialog box.



It was found in the Tasmanian study lagoons that stratify cause oxygen depletion in the bottom layer of water and regularly have significantly lower bacterial die off rates. Stratification usually occurs over the warmer months from November to April and is caused by low wind action on the lagoon surface. To determine how frequently the oxygen is depleted in the bottom layer the DO (and temperature) profile in each cell of the lagoon system should be carried out weekly. If a Tasmanian lagoon is located in a well exposed and windy site it should be stratified less than 25% of the time during the warmer months. In this situation the “Fully Mixed” Option should be chosen from the dialog box shown in Figure 9.10. If the lagoon system stratifies half of the time the “Slightly Stratified” option should be chosen. If the lagoon is mostly stratified throughout the warmer months select the “Stratified” option. For the primary cell performance a “Primary k value” of around 1.5 is quite common although it may be as high as 3 for well loaded primary cells with good cell design. Stratification in these lagoons did not appear to impact significantly on their performance.

Once the cell separation types and the lagoon type has been entered, the expected bacterial die off performance can be compared to actual bacterial die off performance. If the program is giving reasonable performance predictions, hypothetical alterations to flows, loads and designs can be made using the template. This gives designers, lagoon operators and managers an easy way to look at the different options. These options include adding more cells to the system, splitting large cells with baffles, increasing flow due to extra connections, water use minimisation strategies and reducing infiltration scenarios.