

LAGOON EFFLUENT TREATMENT USING GRASS FILTRATION BAYS

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PUBLICATIONS AND AWARDS DURING CANDIDATURE

PUBLICATIONS

Ibrahimi F., Takyi A. K., Perera B. J. C and Lechte P., (1999), *Nutrient Removal Efficiency of Grass Filtration Trial Bays at Western Treatment Plant in Werribee*, Water99 Joint Congress, Proceedings; 25th Hydrology and Water Resources Symposium, 2nd International Conference on Water Resources and Environmental Research, 6-8 July 1999, pp 563-568 Vol 1, The Institution of Engineers Australia.

Ibrahimi F., Takyi A. K., Perera B. J. C and Lechte P., (1999), *Wastewater Treatment using Grass Filtration*, Eds; Anagnostopoulos P and Brebbia C. A., Water Pollution V, Modelling, Measuring and Prediction, pp 27-36, WIT Press, U.K.

Ibrahimi F, (1998), *Nutrient Removal Efficiency of Grass Filtration Bays in Tertiary Treatment of Secondary Effluents*, Proceedings; Environmental Futures Forum held at The University of Melbourne Sep '98, <http://www.envirolinx.org.au/ResourceBase.htm>, Environment Protection Authority (EPA) Victoria, Australia.

Takyi A. K., Ibrahimi F., Perera B. J. C and Lechte P., (1997), *Lagoon Effluent Treatment using Grass Filtration Bays*, Department of Civil and Building Engineering, VUT, ISBN No. 30001004921955, VUT Library.

AWARDS

First prize winner of The 1998 Environmental Futures Forum Award, (The Rio Tinto Award) Australia-wide. The award was presented by the EPA and Rio Tinto for producing the paper titled "*Nutrient Removal Efficiency of Grass Filtration Bays in Tertiary Treatment of Secondary Effluents*". The paper was based on the issues of ecological sustainability.

EXECUTIVE SUMMARY

The Western Treatment Plant (WTP), operated by Melbourne Water, is identified as a major contributor of nutrient to Port Phillip Bay, especially in winter. To minimise eutrophication and maximise beneficial uses of the receiving waters of the Bay, the Environmental Protection Authority (EPA) of Victoria has issued a more stringent waste discharge licence for WTP. Therefore, Melbourne Water has proposed a number of strategies to improve effluent quality into the bay and meet its new waste discharge licence requirements. These strategies include provision of lagoon treatment followed by grass filtration of all dry weather flows in winter.

This thesis presents the results of a project established to determine the waste treatment efficiency, especially for nitrogen removal, and an optimal hydraulic loading rate for the proposed winter grass filtration system when fed with effluent from lagoons at WTP. The project involved hydraulic data acquisition and twice-weekly monitoring of influent, effluent, and wastewater quality within the bays for seven trial Italian Ryegrass filtration bays during a 22-week winter period from May to early October 1997. Four hydraulic loading rates of 20, 30, 40, and 50 mm/day, were used in the trials.

There was a significant amount of cross flow between some of the grass filtration bays used in the trials. Also, infiltration losses were higher than the values expected on the basis of hydraulic conductivity values from studies conducted on soils in the area of the trials. On average, about 5.8 mm/day of flow was lost through infiltration while the net effects of rainfall and evapotranspiration account for about 1.2 mm/day of losses. The high infiltration losses may be due to the root channelling and cracks in the soil.

The trials showed a temporal trend in most of the monitored parameters. Ammonia, total nitrogen, total phosphorus, and BOD removals were generally higher during the first half of the trial period, while effluent colour was poorer during the first three and last five weeks of the monitoring period. The relatively higher ammonia and

phosphorus removals during the first half of the trials were attributed to higher plant uptake and lower influent mass loading. Although nitrification/denitrification is identified as generally the most important nitrogen transformation and removal mechanism on the bays, its relative importance does not vary significantly with time during the entire grass filtration period. The poor colour levels are due to decaying organic matter left on the bays during present or previous grass filtration treatment.

Dissolved oxygen concentration of the effluent also showed a temporal trend similar to that observed for ammonia, total nitrogen and phosphorus. This is due to ageing and reduced photosynthetic activity by the Italian Ryegrass, resulting in a lower rate of oxygen production on the bays during the second half of the trials. Apart from the first three weeks of the trials, most of the other effluent parameters generally remained fairly constant throughout the trials.

Three representative bays (i.e. bays 3, 4 and 7) with hydraulic loading rates of 30, 40, and 50 mm/day respectively, were selected for analysis and discussion of the grass filtration bays. These bays were selected as they represented one of each of the higher hydraulic loading rates and also they were the bays with the most reliable results. The results show that nitrogen removal efficiency decreases with increases in hydraulic loading rate. For the entire trial period, bays 3, 4 and 7 achieved reductions of 63, 46, and 35% of total nitrogen respectively. The same bays achieved 70, 54, and 39% removals of ammonia respectively. These nitrogen removal efficiencies were significantly higher during the first three months of the trials, especially for the lower loaded bays. Generally, it was found that nitrogen removal was inversely related to hydraulic loading.

Phosphorus also showed different removal efficiencies during the first and second halves of the trial period. In fact, in the second half of the trials a small amount of net production of phosphorus occurred, probably from decaying plant tissue. For the entire trial period, bays 3, 4 and 7 achieved reductions of 21, 8, and -2% of total phosphorus respectively. The same bays produced 53, 83, and 95% ortho phosphorus respectively. Similar to nitrogen removal, there was a significantly higher phosphorus removal during the first half of the trials.

Evaluation of bay performances in TN removal suggest that hydraulic loading rate is inversely related to the removal capacity of the bays. Within the bays nitrification/denitrification was identified as the major nitrogen removal mechanism. This is because detailed analyses of all 3 representative bays showed that more than 65% of the total nitrogen was removed by nitrification/denitrification as opposed to plant uptake, volatilisation and infiltration/cross flows. It was also noted that most of the nitrification occurred within the bays by the release of DO through the root zone interface.

Similar to nitrogen removal, it was found that phosphorus removal was inversely related to hydraulic loading and was significantly affected by the detention time of the bay. The longer detention of the bay provides longer contact time between the wastewater and the soil matrix, as well as the wastewater and the vegetation. It was also found that removal efficiency via Italian Ryegrass decreased as the season progressed due to ageing of the vegetation. As season progresses, the phosphorus uptake capacity is lowered, leaving the soil as the next available medium to uptake the phosphorus from the wastewater. However, if the site has been used for wastewater irrigation for many years, then phosphorus deposits from the previous seasons may interfere with this causing a deposit of phosphorus in the wastewater.

Comparison of the trial results against the EPA discharge limits showed that the only parameter within the water quality parameters measured and analysed in this study, which constantly violated the discharge limits, was the effluent colour. All other parameters showed compliance with the EPA discharge limits.

On the basis of the results of these trials, an optimal hydraulic loading rate for the winter grass filtration trial bays was found to be 40 mm/d. At this hydraulic loading rate, high nutrient removal efficiencies as well as high flow discharges can be achieved. However, this hydraulic loading rate should be investigated further through another set of experimental trials and a detailed evaluation of compliance with the license requirements using results from both the winter trials and summer effluent quality data from WTP. Also, since cross flows between adjacent bays can significantly affect the performance of the grass filtration treatment, land preparation

of the grass bays for future trials and a full scale treatment system should be given high priority to minimise channeling and the potential for cross flows between the bays.

Preliminary nutrient modelling was also carried out to obtain nitrogen and phosphorus removal models for the grass filtration trial bays. Only TN and TP were considered in this modelling process. The models give an estimate of the effluent TN and TP concentration when fed with the influent TN and TP data. The modelling involved the derivation of temperature-dependent reaction rate constants to be used in first-order reaction kinetics models, and derivation of simple regression models using linear regression. Due to low correlation coefficients of the data, models of both types were only developed for Period 1 of the study. An evaluation of the two developed models for Period 1 (i.e. first-order reaction kinetics and simple regression models) suggested that the first-order reaction kinetics model produced a better estimate of the effluent TN and TP concentrations.

To evaluate whether the results of this trial study matched those of other studies, a comparison of the results of this study with those of the currently operational grass filtration system at the WTP and the Pakenham (Victoria) grass filtration system was also carried out. The comparison showed some similarities and differences between these systems. The comparisons showed that the trial bays were more efficient in $\text{NH}_3\text{-N}$ and TN removal than the currently operational system at the WTP. Meanwhile the Pakenham system was more efficient in removal of $\text{NH}_3\text{-N}$ than the WTP trial bays and slightly less efficient in removal of TN in 1996. However, it should be noted that the influent to these systems are quite different due to different processes of the prior treatment.

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Last but not least, I would like to thank my family and friends for their patience and encouragement.

DECLARATION

Except otherwise stated , the work described in this thesis is original and has not been submitted for the award of any other qualification in any other University.



Farshad Ibrahimi

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Victoria's Port Phillip Bay is an almost landlocked marine embayment, with the City of Melbourne and its Metropolitan area scattered around its northern and north-eastern shores. The waters of this bay exchange very slowly with the oceanic waters with a detention time of about 1 year (Hunter, 1992). This restricted flushing along with the effects of added pollutants have important implications for the management of water quality and ecology of Port Phillip Bay. Pollutants entering the Bay may cause eutrophication and in turn minimise its beneficial uses. One of the largest sources of pollution to the Bay is the Western Treatment Plant (WTP) in Werribee, formerly known as the Melbourne Metropolitan Board of Works (MMBW) farm (Murray, 1994).

For effective collection, treatment and disposal of Melbourne's wastewater, the Victorian State Parliament established the MMBW farm by Act of Parliament in 1891 and acquired 3,582 hectares of land in 1893 (Paspaliaris, 1996). The MMBW, now known as Melbourne Water, commenced its operation in 1891 to protect public health in the city of Melbourne, and also to protect the environment. The responsibility for collection, treatment and disposal of Melbourne's wastewater led to the establishment of a network of underground sewers for collection and transfer of the wastewaters to the WTP.

The WTP is situated 35 km to the south west of Melbourne and now covers an area of about 10,851 hectares, of which about 6,500 hectares is used for treatment purposes (Murray, 1994). The WTP is a recognised world leader in lagoon technology including commissioning, operation and sludge management. It is also known for its wetlands being of international significance with over 250 bird species. The declared sanctuary which covers the entire site also provides habitat for a range of mammals and reptiles.

Being one of the world's largest lagoon and land sewage treatment works, it services a population of about 1.6 million people from the western and northern suburbs of Melbourne. The plant currently treats almost 500 Ml of sewage per day. This is more than 55% of the total sewage generated in the metropolitan region. The WTP utilises three different biological treatment processes. Treatment consists of lagooning (all year) supplemented by land filtration during warmer months and by grass filtration during cooler months. The treatment system consists of land filtration over 3,633 hectares of relatively permeable soils, grass filtration over 1,143 hectares of lower permeability soils, and an extensive system of lagoons covering an area of 1,667 hectares. Both land and grass filtration processes promote healthy pasture, which is grazed by cattle and sheep. Currently, 70% of the wastewater entering the treatment plant is directed to the lagoons and the remainder through to the land treatment and grass filtration areas. Following a treatment period of 60 to 90 days, the final treated wastewater is in turn discharged to Port Phillip Bay via five Environment Protection Authority of Victoria (EPA) licenced outlets (one of which is only operational during winter months). Figures 1.1 and 1.2 show the discharge outlets and the allocated sections for the different treatment processes at the WTP respectively.

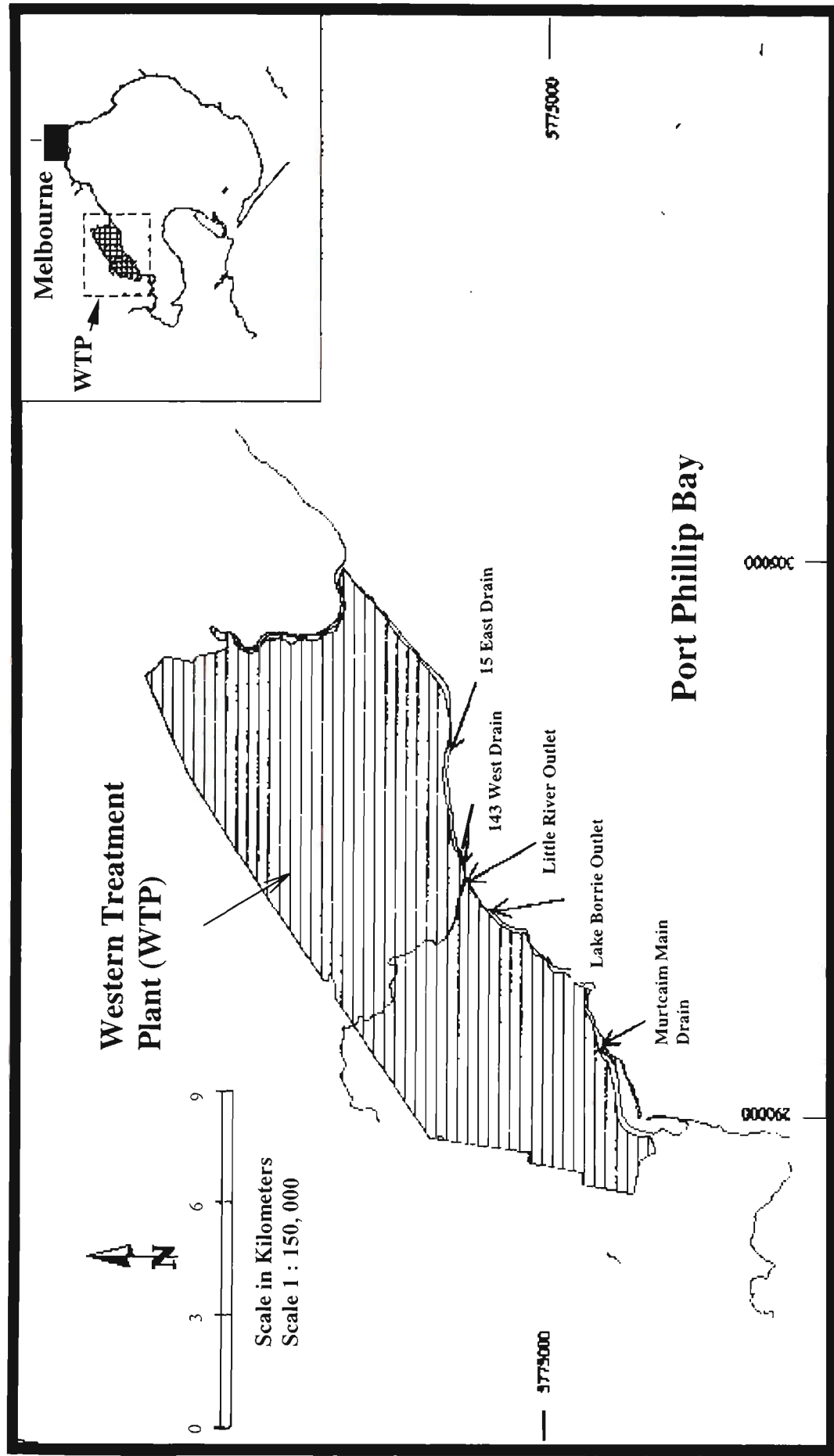


Figure 1.1 Plan of WTP and its EPA Licenced Discharge Outlets to Port Phillip Bay

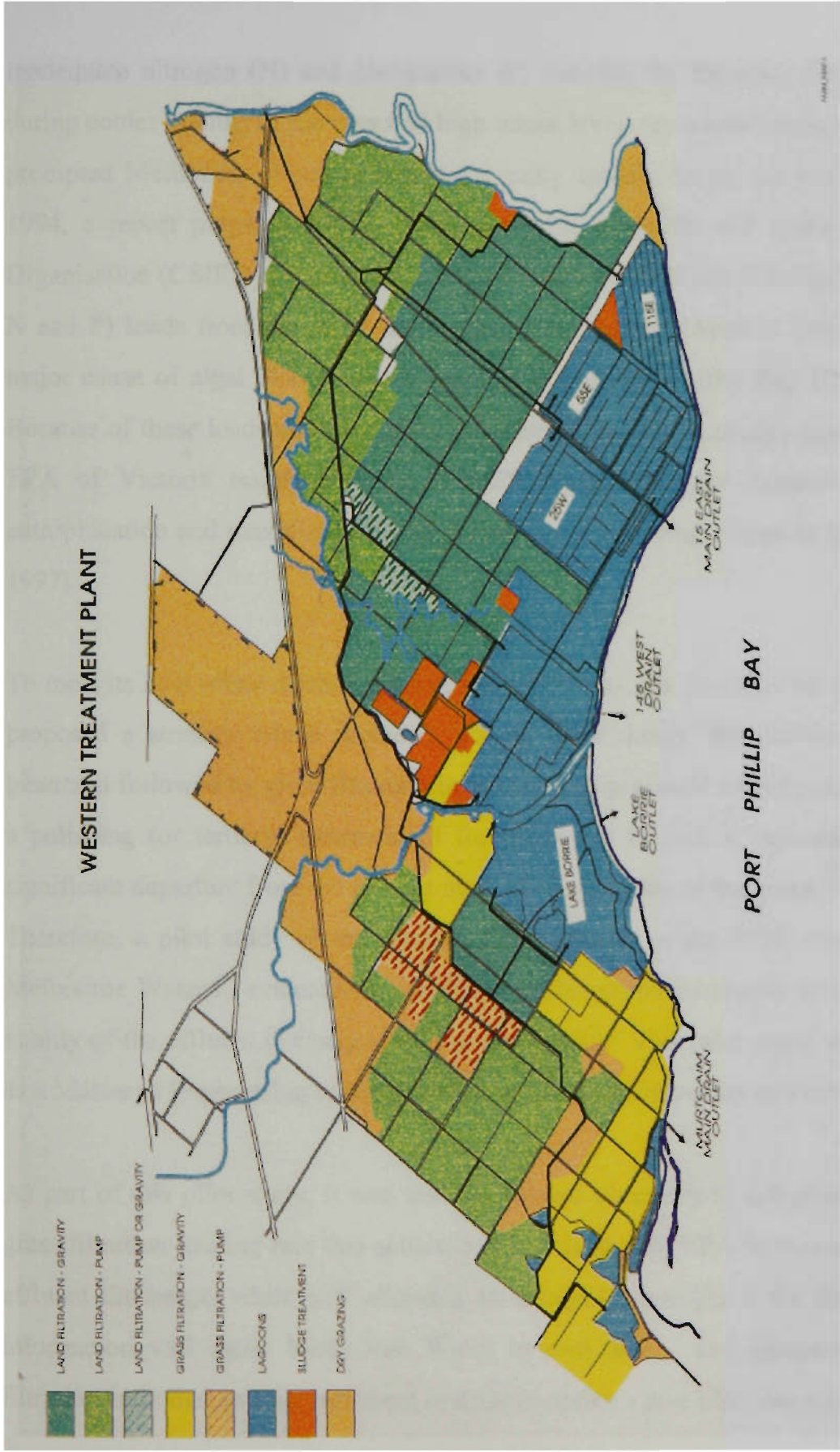


Figure 1.2 Plan of WTP Showing its Lagoons, Land Filtration and Grass Filtration Areas

1.2 PROBLEM UNDER INVESTIGATION

Inadequate nitrogen (N) and phosphorus (P) removal by the grass filtration system during cooler months of the year and high odour levels are major concerns which have prompted Melbourne Water to undertake many studies during the last ten years. In 1994, a report prepared by the Commonwealth Scientific and Industrial Research Organisation (CSIRO) pointed out that the discharge of effluent with high nutrient (i.e. N and P) loads from the WTP during the winter period (April to October) was the major cause of algal blooms and eutrophication of Port Phillip Bay (CSIRO, 1994). Because of these loads and the public requirement for a high quality environment, the EPA of Victoria issued a more stringent waste discharge licence to minimise eutrophication and maximise beneficial uses of the receiving waters of the Bay (EPA, 1997).

To meet its new waste discharge licence requirements, the Board of Melbourne Water proposed a strategy which included, among other things, the provision of lagoon treatment followed by grass filtration in winter. The proposed use of grass filtration as a polishing (or tertiary) treatment of flows already treated in lagoons represents a significant departure from the present method of operation of the grass filtration areas. Therefore, a pilot study of the grass filtration system at the WTP was initiated by Melbourne Water to evaluate the efficiency of this proposed system in improving the quality of the effluent discharged to Port Phillip Bay. This pilot study was conducted as a Master of Engineering research project at Victoria University of Technology.

As part of this pilot study, it was also considered necessary to determine an optimal grass filtration loading rate that achieves compliance with EPA licence conditions for effluent discharge, while still allowing an adequate throughput for the plant. This information will assist Melbourne Water in evaluation and comparison of grass filtration and other sewage treatment options to derive a cost effective system.

1.3 OBJECTIVES OF THE STUDY

The major objective of this project is to conduct a pilot study to evaluate the efficiency of grass filtration during winter as a polishing system for treating wastewaters which have received secondary treatment (i.e. lagoon treatment). In particular, the N and P removal capacity of this treatment process is examined. This major objective is achieved through:

- Establishment and instrumentation of seven trial grass filtration bays,
- Collection of hydraulic and water quality data from the grass filtration bays,
- Evaluation and analysis of N and P mass balances for the bays,
- Identification of possible nitrogen transformation and removal mechanisms,
- Determination of N and P removal efficiency of the bays under various hydraulic loading rates, and
- Derivation of an optimal hydraulic loading rate for the bays, which achieves effluent quality that complies with EPA licence conditions for effluent discharge to Port Phillip Bay and maintains an adequate throughput for the plant.

During the pilot study, Melbourne Water initiated collection of an extensive amount of water quality data for flows through the grass filtration bays. The collected data is relevant in determining removal rates for many contaminants in the wastewater. However, the scope of this thesis is limited to N and P removal.

An attempt is also made in the thesis to develop simple models to predict the behaviour of grass filtration bays in nitrogen and phosphorus removal.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The water and wastewater industry is one of the principal industries in every country, and becomes more important with increase in population (Abeyasinghe et al., 1996). Wastewater disposal is a key component of the water and wastewater cycle. However, the disposal of wastewater to other water bodies, without proper treatment, would drastically affect the quality of the receiving waters. Therefore, scientists and engineers world wide have examined possible options for treatment of domestic wastewaters since the early 1960's.

Due to the demand for better wastewater treatment processes and hence a cleaner environment, various treatment strategies must be examined and implemented. This requires careful planning and research before any action can be undertaken. Tchobanoglous and Burton (1991) stated that in the planning and implementation of wastewater treatment strategies, the re-use application will usually govern the wastewater treatment needed, and the degree of reliability required for the treatment processes and operation. The planning stage must evaluate the reliability of existing or proposed treatment processes and operations (Tchobanoglous and Burton, 1991). There are many ways in which effluent from wastewater treatment can be returned to the environment. Some involve beneficial uses by the community prior to its return to natural water cycle, while others involve direct discharge to waters after some form of treatment (Ang and Marczan, 1996). In this project, the latter method is considered.

However, wastewater is treated by grass filtration as a tertiary treatment process. Grass filtration is a natural treatment process.

The literature review in this chapter incorporates a general overview of natural wastewater treatment processes with emphasis on constructed grass filtration bays and constructed free water surface (FWS) wetlands, along with some examples of existing systems in Australia and other parts of the world. Since the processes governing wastewater treatment are similar in both grass filtration systems and constructed FWS wetlands in relation to municipal wastewater, both are considered in the review. The significance of the water quality parameters that are affected by grass filtration treatment is also discussed in this chapter. The removal efficiency of nutrients (i.e. nitrogen and phosphorus) of the grass filtration treatment process is of great significance in this study. Therefore, a review on nitrogen and phosphorus removal mechanisms in relation to tertiary treatment of municipal wastewaters is also provided within the chapter. Finally, a review of previous modelling work on nutrient removal through natural treatment processes which is directly relevant to this study is also presented.

2.2 NATURAL TREATMENT PROCESSES

Wastewater treatment processes usually consist of primary, secondary and tertiary treatment processes. Depending on the level of treatment desired, wastewater can be treated using various combinations of all three processes to maximise its quality. Natural treatment processes of wastewaters are generally used at secondary and tertiary levels. Natural treatment processes include land treatment, wetland treatment and lagoon treatment.

2.2.1 Land Treatment

Water Environment Federation (1990), Water Environment Federation (1992) and Reed et al. (1995) referred to land treatment as the controlled application of wastewater to the soil to achieve treatment of constituents in the wastewater. This is

generally done via one or several conventional methods involving irrigation or groundwater recharge. The basic purpose of land treatment is to recycle the nutrients and water contained in wastewaters for productive use in agriculture or in other areas, usually following secondary treatment (Russell et al., 1978). Part of the wastewater applied is lost by evapotranspiration, while the remainder returns to the hydrologic cycle through overland flow or the groundwater system. Land treatment is often considered as an alternative to advanced wastewater treatment (or tertiary treatment) for nitrogen and phosphorus removal (Russell et al., 1978). Generally, the land treatment system is used during the warmer months (i.e. high evaporation periods) of the year to enhance the treatment process by making use of the solar energy of the sunlight.

The land treatment process can be classified into three main groups, namely slow rate, rapid infiltration and grass filtration. All three types use natural, physical, chemical and biological processes within the soil-plant-water matrix in treating wastewater. The slow rate and rapid infiltration processes utilise the soil matrix for treatment after infiltration of the wastewater, with the major difference between the processes being the rate at which the wastewater is loaded onto the land. In these systems, the effluent is collected through underground drains and eventually discharged to receiving waters (i.e. lakes, rivers etc). The grass filtration process uses the soil matrix and vegetation for treatment, with the treated effluent being collected as runoff and also eventually discharged to receiving waters. These treatment processes are described in detail below.

2.2.1.1 Slow Rate

Slow rate systems are the predominant form of land treatment of municipal and industrial wastewater. It is by far the most common method of land treatment (Dinges, 1982). Slow rate wastewater treatment entails application rates of up to 0.6 to 1 m/year. An application rate feasible for a specific site is dependent upon soil permeability, evaporation, plant transpiration and border areas.

Land treatment by slow rate irrigation is area intensive and site specific. This treatment process is known to be quite efficient in nutrient reduction of secondary effluents (Dinges, 1982). Soils must be reasonably permeable and areas having dense clay soils or shallow underground rock layers are not suitable for slow rate irrigation. In slow rate irrigation, the wastewaters are applied to the land through spraying or surface flooding. Spraying is carried out using fixed spray nozzles or manual sprays. The surface flooding requires that the land be on relatively flat grades.

2.2.1.2 Rapid Infiltration

Rapid infiltration treatment processes have a long history in treatment of domestic wastewaters (Dinges, 1982). Using this method of treatment, the major portion of irrigated wastewater enters the groundwater, although there is some loss by evaporation (Russell et al., 1978). The basins used for this treatment process, are usually dosed on an intermittent basis to maintain high infiltration rates. Soils are usually coarse textured sands, loamy sands or sandy loams. This process has been developed for groundwater recharge of municipal effluents, municipal wastewater disposal and industrial wastewater treatment and disposal. The distinction between treatment and disposal for this system is quite fine. Wastewater applied to the land for the purpose of disposal is also undergoing treatment by infiltration and percolation.

2.2.1.3 Grass Filtration

Dinges (1982) defined grass filtration as a controlled discharge system of effluent onto land which is planted with vegetations with a large proportion of the wastewater appearing as runoff. Grass filtration is primarily a biological treatment process in which wastewater is applied at the upstream part of the slope and is allowed to flow under gravity across the vegetated surface to runoff collection ditches (Kirby, 1976; Imhoff, 1989; Jayawardane et al., 1996). Renovation (i.e. recycling of the wastewater and reduction of nutrients) is accomplished by physical, chemical and biological means as the wastewater flows in a thin sheet, down the relatively impervious slope. Wastewaters are evenly distributed along the upper ends of gentle slopes (2-6%)

planted with grass. Grass filtration has also been referred to as 'sheet flow' and 'overland flow' (Russell et al., 1978; Water Environment Federation, 1990).

Soils suited to grass filtration are clays and clay loams with limited drainability. The land surface should be evenly graded with essentially no mounds or depressions. The smooth grading and ground slope make possible sheet flow of water over the ground without ponding or stagnation. Grass is usually planted to provide a habitat for biota and to prevent erosion. As the effluent flows down the slope, a portion infiltrates into the soil, a small amount evaporates and the remainder flows to collection channels. As the effluent flows through the grass, the suspended solids are filtered out and the organic matter is oxidised by the bacteria living in the vegetative litter (Russell et al., 1978).

In Australia, grass filtration has mainly been used for treatment of municipal wastewaters. It has been employed at the Werribee land treatment system during the winter season for many years (Seabrook, 1974). Grass filtration could be used as a secondary treatment or as an advanced wastewater treatment process. The latter process will allow different rates of application depending on the degree of advanced wastewater treatment required. Where a surface discharge is prohibited, runoff from the grass filtration system can be recycled or applied to the land in irrigation or infiltration-percolation systems.

One of the disadvantages of this system is that it requires relatively large areas of land. Typical application rates are in the order of 5m/year or no more than 100 mm/week (Dinges 1982). The vegetation used for this treatment process usually consists of grasses, cattails, bulrushes and reeds. However, the most common vegetation used in the United States and Australia Italian Ryegrass.

2.2.2 Wetland Treatment

Everyone has a vague idea of what constitutes a wetland, but not everyone has the same idea. Obviously, wetlands are not continuously dry lands. On the other hand, they need not be continuously wet. Generally, wetlands are an edge habitat, a

transitional zone between dry land and deep water, an environment that is neither clearly terrestrial nor clearly aquatic (Briggs, 1981; Haigh, 1982; Campbell, 1983; Hammer and Bastian, 1989). Therefore, there is no single correct definition of wetlands for all purposes. Water Environment Federation (1990) defined wetlands as 'areas that are periodically inundated with a frequency and depth sufficient to promote the growth of specific vegetation adapted to life in saturated soil conditions'. Several classifications and definitions of wetlands have been made for different purposes. In the context of wastewater treatment, they are generally aquatic environments in which the plants and the soil serve the treatment purpose over a period of time. This naturally depends on the type of wastewater and the level of treatment required.

Wetlands have been sub-divided into two main categories, which are natural and constructed wetlands. Natural wetlands occur through the natural landscape as transitional areas between aquatic ecosystems and uplands. The constructed wetlands are often referred to as artificial and or man-made wetlands. Although natural and constructed wetlands may differ widely in their absolute combination of physical, chemical and biological characteristics, they are similar in macroscopic functions such as organic matter and nitrogen assimilation potential. Sections 2.2.2.1 and 2.2.2.2 give a general overview of natural and constructed types of wetlands respectively.

2.2.2.1 Natural Wetlands

Reed et al. (1995) define natural wetlands as land where the water surface is near the ground surface for long enough each year to maintain saturated soil conditions, along with the related vegetation. Nutrients deposited from wastewaters support an abundance of macro and microscopic vegetation, which convert inorganic chemicals into organic materials.

Natural wetlands along coasts, lakeshores and riverbanks have been receiving great attention because of their valuable role in stabilising shore-lands and protecting them from erosion. One of the greatest benefits of inland wetlands is natural flood control.

Some natural wetlands may function as groundwater recharge areas, allowing water to seep slowly into underlying aquifers. However, the most important function of natural wetlands is water quality improvement. Natural wetlands provide effective and free treatment for many types of water pollution. Wetlands can effectively remove or convert large quantities of pollutants from point sources and non-point sources, including organic matter, suspended solids, metals and excess nutrients. Natural filtration, sedimentation and other processes help clear the water of many pollutants. Some are physically or chemically immobilised and remain there permanently unless disturbed. Many nutrients are held in the wetland system and recycled through successive seasons of plant growth, death and decay. If water leaves the system through seepage, peat or other substrate removes excess nutrients and other pollutants. If water leaves over the surface, nutrients trapped in substrate and plant tissues during the growing season do not contribute to the algae blooms and excessive aquatic weed growths in downstream rivers and lakes. Excess nutrients from decaying plant tissues released during the non-growing season have less effect on downstream waters (Reed et al., 1995).

2.2.2.2 Constructed Wetlands

Constructed wetlands are defined as a designed and man-made complex of saturated substrate, emergent and submergent vegetation, animal life and water that stimulates natural wetlands for human use and benefits (Gearheart and Higley, 1993; Reed et al., 1995). Constructed wetlands have five principal components (Hammer and Bastian, 1989):

- substrate with various rates of hydraulic conductivity
- plants adapted to water-saturated anaerobic substrate
- a water column (water flowing in or above the surface of the substrate)
- invertebrates and vertebrates
- an aerobic and anaerobic microbial population

The plants in constructed wetlands appear to have two important functions (Hammer and Bastian, 1989). These are:

- within the water column, stems and leaves significantly increase surface area for attachment of microbial populations.
- wetland plants have the ability to transport atmospheric gases including oxygen down into the roots to enable their roots to survive in an anaerobic environment.

Some leakage occurring within the roots results in formation of a thin-film in the aerobic region referred to as rhizosphere surrounding each root hair. The rhizosphere mainly supports large microbial populations that conduct desirable modifications of the nutrients and other compounds (Hammer and Bastian, 1989).

Substrates, various soils or gravel provide physical support for plants and other compounds and attachment surface for microbial populations. Surface and subsurface water transports substances and gases to microbial populations, carries by-products and provides the environment and water for biochemical processes of plants and microbes.

Constructed wetlands appear to have very broad applicability as wastewater treatment systems, for an array of water pollution problems (Watson et al., 1989; Tettleton et al., 1993; Hiley, 1995). They have emerged as a viable option to solve a wide range of environmental and water quality problems, both overseas and within Australia (Greenway and Simpson, 1996). Gearheart et al. (1989) indicated that constructed wetlands for treating wastewaters were highly productive. Since nitrogen and phosphorus concentrations are high in raw and treated wastewater effluents, nutrient cycling is rapid in these systems. Rapid nutrient turnover and high standing crop create considerable biomass, which represents a harvestable source of digestible products. Maehlum et al. (1995) indicated that there were several advantages of using constructed wetlands, not the least of which was manipulating control of flow in an environment where temperature dependent processes, which may be physical, biological or chemical, usually take place.

Reed et al. (1995) indicated that typically a constructed wetland should perform better than a natural wetland of equal area, since the bottom is usually carefully graded and the hydraulic regime in the system is controlled. Process reliability can also be improved since the vegetation and the other system components can be managed as required. WEF Manual of Practice (1992) and Reed et al. (1995) reported that there are two types of constructed wetlands in use all around the world, namely free water surface (FWS) and sub-surface flow (SSF) wetlands. The FWS systems is more commonly used in comparison with the SSF system. In FWS wetlands, the water surface is exposed to the atmosphere and the bed contains emergent aquatic vegetation. In addition, it generally consists of a layer of soil to serve as a rooting media, a liner if necessary to protect the groundwater, and appropriate inlet and outlet structures. The water depth in this type of wetlands can range from a few centimetres to 0.8 m or more, depending on the purpose of the wetland. The FWS system would appear to have more in common with grass filtration system, where most of the flow is over land. The SSF wetlands are excavated basins filled with a porous media, usually gravel with the water level maintained below the top of the gravel. In SSF systems, a liner is also used if necessary to protect the groundwater quality. The depth of the media is typically 0.3 to 0.6 m. Existing systems of this type range from those serving single-family dwellings to large-scale municipal systems. The same species of vegetations are used in both types of wetlands.

2.2.3 Lagoon Treatment

Lagoons, also referred to as oxidation ponds, are mainly used for treatment of raw sewage. Lagoons used to treat raw sewage are known as 'stabilisation ponds'. However, lagoons used for tertiary treatment of wastewaters are referred to as 'maturation ponds' (Bartlett, 1972). Lagoons which have been used for polishing sewage effluents, and which have had detention times ranging up to 10 days or more, have been reported to be most efficient in suspended solids removal. The operation of lagoons is relatively cheap and, as quoted by Bartlett (1972), the cost is as little as 10% of the cost of grass filtration operations. However, lagoons require large areas of land.

Lagoons are usually operated at depths of about 1 m and detention time of 10 days, which represents a typical loading rate of about 1000 m³/ha/day. However, lagoons with depths of 2 to 4 m and detention times of up to 17 days have been reported to have greater efficiencies in treatment of wastewaters (Bartlett, 1972). In lagoons, there are always the possibility of floating sludge and algal growth, which may cause seasonal increase in the suspended solids content of the final effluent.

Purification of the final effluent in ponds or lagoons occurs through a combination of sedimentation and biological purification due to various flora. The quality of the effluent will, however, vary with the quality of the influent, more with lagoons than with other forms of tertiary treatment (Bastian et al., 1989). Although reductions of up to 99% in total coliform bacteria and faecal coliforms have been reported during summer with fully oxygenated effluent, the increase in nitrification has been small (Bartlett, 1972).

2.3 NATURAL WASTEWATER TREATMENT IN AUSTRALIA

All Australian large cities, apart from Canberra, are located along the coastline and take advantage of the oceans capacity for treated sewage disposal. The degree of treatment provided depends on the population served, the proximity of the outfalls and the environment. The strategy for systems serving large populations is either secondary treatment with near-shore discharge or primary treatment with long outfalls. In total, Australians produce 5000 Ml of liquid waste per day, which after treatment are discharged to groundwater, streams and eventually to the ocean (Whyte, 1988).

In 1988 it was predicted that over the next 25 years the major risk of pollution offshore in Australia would be from a number of onshore sources which contribute high discharge volumes to a confined or limited body of water (Whyte, 1988). To eliminate the coastal pollution problems, most capital cities started to review their ocean discharge strategies and upgrading of their treatment facilities in the late

1980's. In this review, emphasis was placed on the natural tertiary treatment practices in relation to wastewater treatment.

Two of the Australian States which have made significant use of natural treatment processes for wastewater treatment are Victoria and Queensland. Therefore, brief reviews of these applications are provided in the following sections.

2.3.1 Western Treatment Plant (WTP) in Victoria

The Western Treatment Plant (WTP) in Werribee, Victoria is one of the largest treatment plants in Australia, and treats 55% of the total sewage generated in the Melbourne metropolitan area. Covering an area of almost 10,851 hectares, the WTP employs a number of treatment processes all year around. These include land treatment during summer, grass filtration during winter and lagoon treatment all year around (Russell et al., 1978). Details of these processes are given in Sections 2.3.1.1 to 2.3.1.3.

A recent CSIRO study (CSIRO, 1994) identified the discharge of nutrients from the WTP to Port Phillip Bay as the major cause of algal blooms in the bay. The report indicated that nutrient levels, especially nitrogen, discharged to the Bay from the WTP were the highest during the winter. Therefore, Melbourne Water Corporation initiated a research project (i.e. the project described in this thesis) to investigate the grass filtration process in treating the effluent that had received secondary treatment, with the aim of reducing nutrient loads discharged to the Bay during winter.

2.3.1.1 Land Filtration

As stated in Section 2.2.1, the land filtration system involves periodic application of wastewater to the soil and relies on treatment by passage through the soil matrix. Microbial action, filtration and plant uptake purify the wastewater during the application cycle. Specially prepared bays are used for irrigation and the wastewater is distributed to the irrigation areas via concrete lined channels at WTP. Each bay is irrigated for one to two days followed by a five day drying period and a 14 day

livestock grazing period. The depth of application depends on the circumstances, but an average depth of about 100 mm is common. About 40% of the wastewater applied is collected by sub-surface drains and discharged to Port Phillip Bay, while the remainder is lost through evapotranspiration.

2.3.1.2 Grass Filtration

Prior to 1930, the excess wastewater which exceeded the capacity of the original land treatment was directly discharged to Port Phillip Bay. However, criticisms were raised due to this action and an additional treatment process was considered. Experimental work carried out prior to 1930 indicated that satisfactory results could be obtained from any winter growing vegetation which could withstand cold winter conditions (Borrie, 1931). Initially, the grass filtration bays were laid at natural surface gradients of the land (i.e. 1:1000) and were planted with *Phalaris Tuberosa*. This was then (soon after 1932) replaced by Italian Rye Grass (*Lolium Multiflorum*), and the latter has been used ever since.

The current grass filtration system of treatment at WTP involves continual irrigation of pre-treated wastewater over graded bays, which contain a dense sward of fine stemmed Italian Ryegrass. The grass filtration system receives sewage, which has been pre-treated to remove large suspended solids that may block the wastewater distribution system. The treatment efficiency of the grass filtration system mainly depends on the level of this initial treatment and on the biologically active film which builds up around the vegetation. Suspended solids are mainly removed in the top half of the bays by the mass of roots and stems, and the organic matter is filtered out by the biologically active film created around the stems of the vegetation. A period of three to five days is required for the wastewater to pass through the Ryegrass, and the treated effluent is collected via a main drain of the downstream end of the bay and then discharged to Port Phillip Bay. The system mainly operates during winter periods between April and October, when land filtration is not in operation because of low evapotranspiration. The bays are dried out at the end of the season and grass is grazed by the livestock. Typical effluent quality from the grass filtration bays (loaded at 14-18

mm/day) is: 25 mg/l TN; 6 mg/l total phosphate as P; 25 mg/l five day biochemical oxygen demand (BOD₅); 30 mg/l suspended solids (SS) and 500-50,000 Org/100 ml.

Currently, the graded bays are laid out on a conventional gravity border check pattern, with bays measuring about 400m long x 40m wide and laid on a gentle slope of 1 in 300 to 1 in 250. Wastewater is distributed to the bays via concrete lined trapezoidal channels and rectangular inlet structures which are controlled by orifice plates attached to drop boards operating under constant head conditions. The orifice diameters vary from 20 to 50 mm. These orifice plates require regular checking to prevent any blockages. All outlets are controlled by 90° V-Notch weirs affixed to drop boards.

2.3.1.3 Lagoon Treatment

This is the most extensive lagooning system in Australia. Wastewater is treated by passage through a series of interconnected ponds. This wastewater contains a high concentration of naturally occurring bacteria, which consume the biodegradable matter. Mechanical surface aeration is used in some of the ponds to aerate the wastewater. The current lagoon system uses a deep high rate anaerobic pond, followed by surface aeration on the second pond and a series of subsequent ponds optimised for nitrogen removal through the nitrification/denitrification process.

2.3.2 Natural Wastewater Treatment in Other Parts of Australia

The climatic conditions of Queensland consisting of dry winters and wet monsoonal summers make it conducive to high plant growth rates and hence offer great potential for constructed wetlands for water pollution control (Greenway and Simpson, 1996). Since most wetlands both in Australia and overseas were used during cold temperate climates at the time of review, it was interesting to discover how they behaved in Queensland conditions. Therefore, pilot wetland projects were set up in 1992 before any full scale treatment plants could be operated with full confidence. As at 1996, nine of these projects were commissioned in the towns of Mossman, Cairns, Ingham,

Townsville, Mackay, Yeppoon, Emu Park, Wamuran and Logan city to treat municipal secondary treated sewage effluent (Greenway and Simpson, 1996). However, the authors reported only on the pilot wetland projects which have been in operation for the longest period (i.e. wetlands of Ingham, Townsville and Blackall). They are reviewed in the following sections.

2.3.2.1 Ingham Wetland Project

The Ingham wetland, situated in tropical north Queensland, was commissioned in 1993 by the Hinchinbrook Shire Council. The main treatment objectives were polishing of secondary effluents (BOD and SS removal), nutrient reduction and disinfection. This was initiated in a response to comply with up-to-date environmental protection legislation with respect to the disposal of secondary effluents into coastal and inland waters (Greenway and Simpson, 1996).

The Ingham wetland was of the Free Water Surface (FWS) type. The bays were planted with several species of vegetation, which included emergent reed, rushes and sedges, aquatic grasses including fodder species and vines. This wetland consisted of 3 U shaped channels 110m x 12m x 500mm. Although the U shape may increase the detention time of the bays, allowing more time for the plants to uptake the nutrients, it may not be economically viable. This is because the authorities would often require the highest volume of wastewater treated in the shortest time possible. Furthermore, since artificial wetlands are normally declared to have a detention time of between 4-6 days, a detention time of 12 days for the Ingham wetland as quoted by Greenway and Simpson (1996) seems longer than average for bays of such geometry.

It was claimed by Greenway and Simpson (1996) that the wetland performed to expectations as it had achieved 48, 52 and 8% reductions in BOD, total nitrogen (TN) and total phosphorus (TP) respectively between influent and effluent. These performance measures are in agreement with most wetlands reported performances, especially the low reduction of TP. However, it must be noted that several species of vegetation were used at the Ingham wetland. Therefore, it is difficult to draw conclusions on exactly which type of vegetation may have been responsible for BOD

and nutrient reductions that took place at the wetland. It is also not known, whether the wastewater temperature was measured and how significant its effect was on the biological activities that occurred within the wetland. The hydraulic loading rate of the bays were also unknown.

2.3.2.2 Townsville Wetland Project

In March 1993, the Townsville City Council commissioned a pilot project at Mt St John, with the objective of polishing the effluents from the wastewater treatment plant (Greenway and Simpson, 1996). For this pilot project, 4 straight bays and 1 U shaped bay (each with dimensions of 60m x 4m x 400mm) were used. Although the water depth of the wetland is 400 mm, this system can be considered as a FWS system according to Reed et al. (1995). The bays were initially vegetated with six species of macrophytes, which were then colonised with an additional 8 species resulting in fully vegetated bays. The design detention time was reported as 5 days and 7 days for the straight and the U shaped bays respectively. The water quality parameters monitored for this pilot study consisted of BOD, suspended solids (SS), TN, ammonium nitrogen ($\text{NH}_4\text{-N}$), oxidised nitrogen ($\text{NO}_x\text{-N}$), organic nitrogen (Org-N) and TP. However, there was no mention of the hydraulic loading rates used to irrigate the wastewater on the bays or whether any mass balance analysis was performed.

Greenway and Simpson (1996) indicated that the pilot wetland project produced effluent of high quality. This was also evident in the results obtained from the pilot project, which showed reductions of BOD, SS, TN and TP between influent and effluent concentrations of 67%, 44%, 74% and 6% respectively for a 5 to 7 days detention time. These results show significant reductions, especially in TN and BOD. However, the TP reductions are considerably low. These results are consistent with the results of Ingham wetland.

The discharge of the effluent after its passage through the Townsville wetland into the Town Common was highly effective, as the Town Common had attracted a variety of bird species during the drought and dry seasons. Therefore, after obtaining the results of the wetland project and considering re-use ability of treated wastewater, it was

decided by the authorities that this method of treatment was worthy of further investigation for other re-use purposes.

2.3.2.3 Blackall Wetland Project

The Blackall wetland situated in the Western Region of Queensland was commissioned in February 1993 and was initially planted with only 3 species of vegetation. The major objective of this wetland project was re-use of effluent. Therefore, nutrient retention was considered more important than nutrient removal and removal of BOD.

The wetland consisted of 4 straight bays of 120m x 7m x 600mm. This system may be considered as a FWS system, with a water depth of 600mm. Three of the bays were planted with vegetation from the start of the trials and the fourth bay was left unplanted as open water. The vegetation used in the wetlands were different types of grasses and aquatic species (Greenway and Simpson, 1996). The bays were reported to have detention time of 3 to 4 days. Information on hydraulic loading rates, temperature effects and mass balances for this study was not available.

In spite of having low detention times of 3 to 4 days, reductions of 46 and 68% were reported for BOD and TSS respectively between influent and effluent. However, on concentration basis, a low reduction in TP of only 3% was achieved.

2.3.2.4 Final Remarks

All these trial wetlands show significant reductions in BOD, nutrients, suspended solids and very low reductions in TP. The reductions are higher for wetlands with higher detention times. However, it is difficult to find correlations between removal efficiencies and detention time, since there are other variables (such as temperature which were not available).

Greenway and Simpson (1996) indicated that there was much conjecture in the literature (Mitchell et al., 1994; Thomas et al., 1994) with regards to phosphorus

removal in artificial wetlands. Although the wetlands of Ingham, Townsville and Blackall showed low reductions of TP, further investigations of the vegetations had shown phosphorus accumulation in the plant tissue of the macrophytes in the wetlands. It was shown by the authors that the vegetations of the artificial wetlands accumulated much higher levels of phosphorus in comparison to natural wetlands.

After the completion of the Blackall wetland pilot project, the local Council decided to investigate the effluent re-use schemes for different purposes including, construction of a riverbank eco-tourism wetland complex, irrigating commercial treelots and irrigating community parks and gardens in order to conserve the resources of the Great Artesian Basin (Greenway and Simpson, 1996).

2.4 NATURAL WASTEWATER TREATMENT IN OTHER PARTS OF THE WORLD

Juwarker et al. (1995) stated that more than 500 wetlands were successfully installed all around the world for treatment of municipal and industrial wastewaters. These countries include Denmark, Austria, Saudi Arabia, Italy, Hungary, Brazil, Mexico, Finland, Canada, U.S.A, most European countries, Asia and many other countries. A national survey of Canadian Provinces in 1992 identified 67 wetlands for treatment of wastewater or stormwater, of which 67% were full scale operating systems (Pries, 1994). These systems were mainly located in the cold temperate regions and have successfully met the effluent standards criteria across Canada.

Kadlec and Knight (1996) reported that the North American treatment systems contained 176 wetland treatment sites. Of these, 60 were considered to be located in the colder temperate regions with over 40 located in South Dakota. Almost 90% of these wetlands treated municipal wastewater and almost all of them were of the FWS type. Kadlec and Knight (1996) also estimated that all-together there were over 650 natural and constructed wetlands in Canada and U.S.A in 1994. However, the design criteria and performance capabilities of these wetlands are either unknown or not reported.

Scandinavia is another pioneer country in constructed wetland technology. Between 1983 and 1990, more than 130 wetlands were constructed in Denmark for treatment of municipal wastewater (Schierup et al., 1990). However, Sweden has had little interest in constructed wetlands. In comparison to Denmark, only 6 FWS and 8 SSF wetlands have been constructed in Sweden. This may appear as a surprise, since the constructed wetlands are known to be used more in colder climates. However, they are not utilised in Sweden as much due to the problems arising from the freezing of wastewaters.

Wetlands have also been extensively used in the Eastern Europe. One of the leading countries in use of wetland systems in Eastern Europe is the Czech Republic, where 28 systems have been built since 1989 and an additional 54 systems were in the design stage in 1996 (Vymazal, 1996). Most of these systems have been successful in achieving their primary objectives (i.e. reduction of nutrients from the industrial and municipal wastewaters). However, land space has been a major issue.

Brief summaries and performance of some wetlands for which information were found, are described in the following sections.

2.4.1 Arcata Pilot Project, California

California has two operational constructed wetlands at the two small towns of Gustine and Arcata. Both constructed wetlands were established after successful pilot studies. Due to unavailability of data on the Gustine pilot project, only the pilot project of Arcata is discussed in this section.

The City of Arcata completed four years of pilot project studies with a series of constructed wetlands to treat the secondary effluent (Gearheart et al., 1989). The outcome of the pilot project demonstrated that a constructed wetland can provide reliable tertiary treatment for municipal wastewater, and the wetland effluent can enhance water quality in Humboldt Bay. Following the experimental studies on the pilot project, the Arcata Wastewater Treatment Plant was upgraded to a full scale treatment system in June 1986. The upgrade included provision of a 13.6 hectare effluent polishing wetland. Gearheart et al. (1989) also reported that the pilot project

consisted of two experimental studies, using FWS wetlands, with the first being from September 1980 to September 1982 and the second study from 1984 to 1986. These are reported in Sections 2.4.1.1 and 2.4.1.2 below.

2.4.1.1 First Pilot Study

In the first study, 12 experimental wetlands received the effluent from an oxidation pond from September 1980 to September 1982, after a one-year construction and start-up period. Hydraulic loading rates were varied from 20 - 240 mm/d to cover a wide range of loadings to the experimental bays. The experimental bays were 6.1m x 61m x 1.2m and planted with bulrush (*Scripus Validus*). However, the slopes of the experimental bays were not reported.

The detention times of the bays ranged from 38 to 183 hours (equivalent to 1.6 to 7.6 days). The variation in the range of detention times seems quite high, considering the fact that all bays were of the same dimensions. However, it is due to the large variation in the hydraulic loading rate, which is a significant factor in determining the detention time. Although many water quality parameters were monitored, this study mainly focused on SS, BOD₅ and coliforms. The wastewater temperature and its seasonal variations, which can have an effect on biological activities taking place within the bays, were not reported.

The results from the first study showed highly variable influent BOD₅ and SS values. The average effluent BOD₅ values from the experimental wetlands ranged from 9.0-15.3 mg/l with an average across all bays of 13.3 mg/l. The BOD₅ removal rates in bays varied from 41% to 65% with an average across all bays of 56%. However, SS were effectively removed to less than 10 mg/l at all hydraulic loading rates with no seasonal variations. Average SS effluent concentration in bays ranged from 4.0 to 9.4 mg/l with an average across all bays of 5.3 mg/l, representing an average removal of 85%. Faecal coliforms concentrations ranged from 272 to 3183 CFU/100 ml with an average percentage removal across all bays over the period of study of 86%.

Gearheart et al. (1989) concluded that within the tested loading range of 20 - 240 mm/d, the effectiveness of SS removal was not a function of hydraulic loading rate but appeared related to some minimal detention time. This appears to be consistent with the general behaviour of the FWS wetlands in removal of SS, however, the other factors such as the type and density of the vegetation may also be significant in removal of SS from secondary treated effluents.

2.4.1.2 Second Pilot Study

The second study was conducted from 1984 to 1986. The objectives of this study were to determine the effect of vegetation on effluent water quality parameters and to determine the kinetics of BOD₅ and faecal coliform removal in a wetland treatment system. It must be noted that the study specifically aimed at maximising DO levels in the effluent and maximising BOD₅, SS and nutrient removal in treating primary wastewaters to secondary standards. For this purpose, the inlet and outlets were monitored by placing a 60° V-notch weir, and a 90° V-notch weir respectively.

For this study, 10 bays with dimensions of 61m x 6.1m, planted with cattails, were used. It is not known whether these bays were the same as those used for the first study. Heavy clay soil was used in the construction of the bays and therefore, seepage was insignificant. Based on the inflow rate of 0.34 l/s, a theoretical detention time of 7.5 days was reported for the bays (Gearheart et al., 1989). However, the actual detention time was not provided and therefore, it is assumed that any analysis and conclusions drawn relating to detention time are based on the theoretical detention time.

Results of the study showed that the influent BOD₅ loadings were in the range of 15.7 to 67.5 kg/ha/day. The average BOD₅ and SS removal over the period of study from all bays were 72.9% and 77.5% respectively. Gearheart et al. (1989) stated that autofloculation and settling accounted for the high removal of SS. Since one of the objectives of the second study was to determine the removal kinetics, Gearheart et al. (1989) concluded that plots of log of BOD₅ against detention time in the wetland produced an excellent straight line fit of the experimental data linking the removal of

BOD₅ directly to the detention time. Gearheart et al. (1989) also stated that the log of remaining coliforms against the theoretical detention time gave a correlation coefficient (R^2) value of 0.99 yielding a 0.29/day removal rate constant for reduction of faecal coliform density through wetland system with emergent vegetation.

2.4.1.3 Full Scale Operations

As mentioned in Section 2.4.1, the Arcata Wastewater Treatment Plant upgrade, finished in June 1986, included 13.6 hectares of effluent polishing wetlands. The wetlands served as unit processes for BOD₅ and SS removal, a wildlife refuge and a passive recreation facility (Gearheart et al., 1989). The design criteria for the wetlands were developed from the first pilot project.

Gearheart et al. (1989) reported the full scale operation results from August 1986 to May 1988. During this period, the BOD₅ and SS average effluent concentrations were quoted as 26 and 30 mg/l respectively. The overall removals of BOD₅ and SS were 54 and 53% respectively. Gearheart et al. (1989) stated that the first two years of the full scale operations (i.e. 1986 to 1988) were considered as start-up conditions because vegetation patterns and density were changing rapidly during initial wastewater loadings. It was concluded that open water areas (i.e. FWS system) in constructed wetlands permit phytoplankton production, thereby contributing to the suspended solids level. DO levels are also a function of the amount of open water. Natural re-aeration plus photosynthetic contribution from phytoplankton populations produce higher DO levels. Effluent BOD₅, on the other hand, is a function of organic loading rates.

2.4.2 Alabama Pilot Projects

Slayden and Schwartz (1989) briefly reported the pilot projects and activities related to the use of constructed wetlands in various States of the U.S. It was indicated by the authors that the Alabama Department of Environmental Management (ADEM) monitored the performance of four constructed wetlands, three full scale and one pilot project. The three full scale projects are (1) Degussa, Inc.; (2) Tennessee Valley

Authority (TVA) Fabius Mine/Washer; (3) Vredenburgh, and the pilot-scale project is Hurtsboro. The authorities of Alabama used the FWS system for their wastewater treatment. The vegetation used on the wetlands was marsh meadow and rock-reed filters were used at some applications. The recommended loading rate for secondary treated influent was 2-25 mm/d for a marsh system (Slayden and Schwartz, 1989).

Slayden and Schwartz (1989) stated that the Alabama Department of Public Health regulated the Vredenburgh constructed wetland. Vredenburgh was a classic example of a poor rural community with either failing septic tanks or no facilities, and seriously in need of a proper sewerage system. The wastewater from about 100 households passed through two FWS bays and then through a rock-reed filter. This system operated for one year. However, due to limited availability of data for the Alabama projects, it is rather difficult to judge their performances or make comparisons with other systems used in the U.S. or elsewhere.

2.4.3 Natural Wetland Treatment in Other American States

Constructed wetlands are used for treatment of domestic and industrial wastewaters in most States of U.S. Some of these states are Florida, Kentucky, Mississippi, Missouri, North Carolina, Pennsylvania, South Dakota, Tennessee, Texas and Virginia, all making use of constructed wetlands in some form for their wastewater treatment (Slayden and Schwartz, 1989). Although not all of these States use FWS systems for their constructed wetlands, most of them aimed at removing nutrients from treated wastewaters and use constructed wetlands as a polishing scheme before the wastewaters are discharged to other waters. This closely correlates with the purposes for which constructed wetlands and overland flow systems are used in Australian practice. However, no operational and performance data were reported for these constructed wetlands.

2.4.4 Oxelosund Pilot Project in Sweden

In 1993, the Swedish Environment Protection Authority (EPA) re-evaluated their wastewater discharge limits to the surface waters due to increased nitrogen loads

which caused algal blooms in the Baltic Sea (Wittgren and Tobiason, 1995). Due to this decision, wastewater treatment plants which served more than 10,000 person equivalents, and which were situated along or near the coast of southern Sweden, were expected to remove at least 50% of the incoming nitrogen.

The town of Oxelosund (with 12,500 inhabitants) on the Baltic coast of Sweden was faced with the requirement to remove 50% of the nitrogen entering the municipal wastewater treatment plant. The town was previously equipped with only mechanical and chemical treatment facilities to remove BOD and phosphorus. With extra funding from the Swedish EPA, a wetland system was built in Oxelosund for nitrogen removal with the additional aim of performing a detailed monitoring program on the nitrogen removal capacity of wetlands. Although the pilot project was to be conducted from 1993 to 1998, the results of the first year of the study (i.e. 1993 to 1994) were reported by Wittgren and Tobiason (1995).

The system used at Oxelosund was a FWS system consisting of 5 bays, four running in parallel and one final common bay, covering a total area of 21 hectares (Wittgren and Tobiason, 1995). The bay outlets were equipped with rectangular weirs, through which the final effluent was discharged and transferred to the Baltic Sea. However, no details were given on the inlet flow control devices. Flow, temperature and pH were continuously monitored at the bays outlets. Various types of vegetation were used throughout the site, including cattails, rushes, reeds and sedges. Since nitrogen removal was the main objective of the system, vegetation establishment was focused on rapid achievement of a large standing crop of pioneering wetland species. As a result, cattails (*Typha latifolia*) and *Elodea canadensis* were used as the dominant species in the bays. However, no details on the detention times of the bays were given. The experiments were conducted during winter and spring of 1993. The system loading rate was 28 mm/d of pre-treated wastewater from the Oxelosund wastewater treatment plant.

A different loading strategy (i.e. an intermittent emptying and filling strategy) to the normal practice (i.e. loading the bays for a particular season followed by an off season) was used for the Oxelosund wetland system. The intermittent emptying and

filling was used in an attempt to promote nitrification, denitrification, and to minimise hydraulic short-circuiting within the bays. It was argued that the design loads in many cases were too high to leave enough oxygen for substantial nitrification to take place (Wittgren and Tobiason, 1995). While the intermittent emptying and filling might affect the nitrification and denitrification, the potential for a wetland system to remove greater amounts of nitrogen than those indicated in other experiments, may only be greater provided that the BOD loads are low and that the system is primarily designed and operated to optimise nitrification. On the other hand, the intermittent emptying and filling may not be a feasible solution for most places where land space is an issue, since this particular strategy may require extra bays and may cause delays in the normal operating time.

The operations of the Oxelosund wetland system also included detailed water quality monitoring. The water quality monitoring and method of analysis included twice weekly grab sampling from the outlets. The samples were analysed on a weekly basis. They were mainly analysed for TN, ammonium nitrogen ($\text{NH}_4^+\text{-N}$), $\text{NO}_3\text{-N}$, BOD_7 and TP.

The results from the study conducted by Wittgren and Tobiason (1995), showed that the nitrogen input to the wetland was totally dominated by $\text{NH}_4^+\text{-N}$, since there was no biological treatment in the treatment plant prior to the wetland process. The $\text{NH}_4^+\text{-N}$ made up for an average of 79% of inlet TN. However, significant (although low) concentrations of $\text{NO}_3\text{-N}$ indicated that nitrification took place in the wetland. BOD_7 and TP concentrations were also significantly reduced at the outlet compared to the inlet. A 41% reduction in Nitrogen was achieved, which was less than the 50% reduction required by the Swedish EPA. Average TN load removal was 78 kg/ha/month during autumn but only 24 kg/ha/month during winter (December 1993 to March 1994). Over the whole year, a total of 15 tonnes of nitrogen was removed from the total load of 38 tonnes in the influent. Although significant reductions in nitrogen loads were achieved, they did not meet the removal standards of the Swedish EPA.

Wittgren and Tobiason (1995) concluded that there were clear indications that nitrification did take place and that nitrification-denitrification together with plant uptake were responsible for all the ammonia removal. Volatilisation of ammonia may be eliminated as a removal mechanism, since the maximum pH level measured was 7.6, which was insufficient for ammonia production. Bohn et al. (1985) stated that “when a FWS wetland is loaded with wastewater rich in NH_4^+ -N, these ions may be sorbed to cation exchange sites at the surfaces of organogenic sediment and plant litter, since organic matter has a high cation exchange capacity”. When these surfaces are exposed to oxygen rich water or air, there should be a large potential for nitrification of sorbed NH_4^+ -N. Therefore, at the next flooding event, the produced NO_3^- could be denitrified at anaerobic microsites with plant produced organic matter as the carbon and energy source. Wittgren and Tobiason (1995) stated that these were the assumptions on which the intermittent emptying and filling system was based for the project. However, the authors have not commented on whether an alternative system (i.e. one where intermittent emptying and filling was not used) would perform better or worse for the same conditions as those of the Oxelosund wetland system in reduction of nitrogen loads. Due to lack of further information on this pilot project, it is not known whether a full scale system was installed after its completion in 1998 or whether the monitoring was continued until 1998.

2.4.5 Sainik School Pilot Project in Orissa, India

The volume of wastewater generated in India as reported in 1995 was almost 8,642,000 Ml per annum from 212 cities and 241 towns (Juwarker et al., 1995). Due to the high cost of conventional systems of treatment, only 23% of the generated wastewaters are treated at primary level before disposal and the remaining 77% are disposed to surface waters and on land without any treatment. Severe pollution problems have occurred in India due to this practice. Therefore, steps towards installation and operation of constructed wetlands were taken from the early 1990s.

Since little was known about the design, operation and performance of constructed wetlands in India for wastewater treatment, the Sainik School constructed wetland at Bhubaneswar in Orissa State was installed as part of a pilot study. The major

objective of the study was to determine the BOD, nitrogen and phosphorus removal capacity of constructed wetlands planted with emergent macrophytes. The wetland was 90m x 30m planted with *Typha Latifolia* and *Phragmites Carca*. The water depth used on the wetland was not reported. The wetland was sub-divided into three bays with one bay planted with *Typha Latifolia*, the other with *Phragmites Carca* and the last bay left unplanted to see the effect of vegetation on the nutrient removal capacity of the wetlands. However, no information on the detention time of the bays were given.

Primary treated wastewater was irrigated at 50 mm/day over the surface of the vegetated bay with the effluent discharged to a main drain through the bay outlet. Influent and effluent samples were collected once every 5 weeks for 40 weeks, and analysed for BOD, nitrogen and phosphate. Although the sample collection was carried out regularly, it must be noted that the frequency of sample collection (i.e. once every 5 weeks) seems rather low. Depending on the magnitude of the project and the accuracy of the results desired, more frequent sample collection would have been more advantageous.

The results of the study showed that the constructed wetland had variable removal capacity depending on the type of vegetation used on the bay. The constructed wetlands achieved nitrogen removals of 66 and 73% for *Typhya Latifolia* and *Phragmites Carca* respectively. The nitrogen effluent concentrations ranged from 8.5 to 9.8 mg/l for *Typhya Latifolia* and 8.2 to 9.2 mg/l for *Phragmites Carca*. It was further observed by Juwarkar et al. (1995) that the removal efficiency of nitrogen was greater with respect to *Phragmites Carca* due to profuse growth compared to *Typha latifolia*. The results also show that removal of phosphate was far less than for nitrogen which was 23 and 48% respectively for *Typhya Latifolia* and *Phragmites Carca*, however, the BOD removal was observed to be 78 to 91% for *Typhya Latifolia* and *Phragmites Carca* respectively. The effluent BOD ranged from 10.2 to 40.6 mg/l compared to the inflow, which ranged from 165 to 227 mg/l. It was also found that the vegetated bays were more efficient in nutrient removal than the unvegetated bay. This was based on nitrogen removal in the vegetated bays of 66-73% compared to the unvegetated bay which removed 20-28% of the influent nitrogen. Similarly, the

phosphorus removal in the vegetated bay was higher than the unvegetated bay. These figures seem to agree with results of other experiments, which once again confirms that constructed wetlands are effective in removal of significant loads of nitrogen from domestic wastewaters. However, while there is consensus that constructed wetlands are not very effective in phosphorus removal and that in most cases they have shown phosphorus production, this is not true in the case of the Sainik School wetland project, since a reasonable amount of phosphorus was removed during the trial.

Juwarkar et al. (1995) concluded that the macrophyte plant root and rhizomes in the rhizosphere leak oxygen into the microzones in an otherwise anaerobic zone, which in turn stimulate the breakdown of carbonaceous compounds. The authors also concluded that the constructed wetland system of treatment achieved significant removal of BOD, nitrogen and phosphate, was easy to operate and economically viable, and was the answer to wastewater management.

2.5 SIGNIFICANCE OF WATER QUALITY PARAMETERS MEASURED IN THIS STUDY

A brief overview of the significance of the water quality parameters which were considered and tested in this research project is given below. These parameters include physical, chemical and microbiological parameters, and nutrients.

2.5.1 Physical Parameters

These parameters include wastewater temperature, pH, dissolved oxygen (DO) (often referred to as a physical parameter), redox potential, total suspended solids (TSS) and colour. It should be noted that temperature, pH and redox potential are usually measured in-situ. However, TSS and colour measurements are obtained from laboratory analyses of the field samples.

2.5.1.1 Temperature

The temperature level can directly affect the growth of algae in the wastewaters. Measurement of temperature is thereafter critical, if biological activity in the wastewater is a matter of concern. Wittgren and Maehlum (1997) argued that nutrient uptake by plants and microbial transformations of wastewater components and plant litter in wetlands are both directly and indirectly affected by climatic conditions. The direct influence is that plant physiology is governed by solar radiation and temperature, while the microbial processes are governed by temperature alone. Oxidation of organic matter and nitrogen transformations are the most important microbiologically mediated treatment processes, which are directly affected by temperature. Meanwhile, the indirect effects are the dependence of biological and biochemical processes on physical conditions, which are in turn affected by climatic conditions. Oxygen availability and freezing/thawing of soil are important examples of such physical conditions.

The effluent wastewater temperature typically ranges between 8 to 14 °C during the winter period using grass filtration bays.

2.5.1.2 pH

The pH value (or the hydrogen-ion concentration) of a liquid is an indication of its degree of acidity or alkalinity (but not of total quantity). Pure neutral water contains 0.000 0001 g/litre of hydrogen ions, which could be expressed more simply by using the logarithmic value of the reciprocal (i.e. in this case 7). The pH value is then measured up to 14, and with neutral solutions having a value of 7, all solutions below pH 7 are said to be acidic while those with pH above 7 are alkaline (Bartlett, 1972). Domestic sewage is expected to be alkaline with an average pH value of about 7.2 since it's major constituent is tap water. Acidity is generally due to presence of mineral acids from industrial wastes, organic acids from breweries, dairies and similar establishments.

In tertiary treatment processes (i.e. grass filtration), one of the most important water quality parameters is nitrogen which is present in various forms. When effluents are discharged to receiving waterways, the form of nitrogen which is most toxic to fish is ammonia, while ammonium (i.e. the ionised form of ammonia) is harmless. The relation between ammonia and ammonium, is dependent on pH (Russell et al., 1978; and Sorensen and Jorgensen, 1993). As Thomas and Lauden (1989) indicated, throughout a wetland, bacterial mediation of organic decay generates $\text{NH}_3\text{-N}$ and HCO_3^- , which raise the pH and cause hydroxide precipitation. These factors imply that the pH value is critical in situations where the nutrients discharged to receiving waters may cause eutrophic conditions. For this reason, pH measurements of the wastewaters were carried out in this research project.

2.5.1.3 Dissolved Oxygen (DO)

DO levels in natural and wastewaters depend on the physical, chemical and biochemical activities in the water body. The analysis for DO is a key test in water pollution and waste treatment process control (Hauser, 1996). DO measurements are important, since vital information like pollutant transformations occurring on the grass filtration bays can be obtained. Also, the amount of DO in water can significantly affect the level of biological activity in the water. Therefore, it is considered as one of the most significant water quality measurements.

Most micro-organisms use dissolved oxygen in water for respiration. Oxygen depletion in natural water bodies caused by addition of organic wastes may limit life in that water. Hence, the amount of DO in the wastewater limits the ability of the microbes in the wastewaters to degrade any added nutrients. Meanwhile, photosynthesis adds DO to the water as a by-product.

2.5.1.4 Redox Potential

Redox potential is a term used for oxygen-reduction potential, which is a measure of the effectiveness of the dissolved oxygen content of a waste, and also an indication of the corrosiveness of a liquid. Many chemical and biological reactions in treatment

processes are fundamentally oxygen-reduction systems (Bartlett, 1972). Redox is measured either positive or negative in terms of millivolts. A liquid with values above 400 mV are not corrosive, while those with values less than 100 mV are severely corrosive.

Measurement of redox potential is significant for obtaining information on corrosiveness of the wastewaters and availability of oxygen. More importantly it is a measure of the likelihood for nutrient and metal desorption for the substratum.

2.5.1.5 Total Suspended Solids (TSS)

After biochemical oxygen demand (BOD), the TSS is probably the next most important water quality constituent for both crude sewage and treated effluents. Generally, the amounts of suspended and soluble solids in a sewage determine the design details of sedimentation tanks and the sludge drying process for the initial treatment. In tertiary treatment (i.e. grass filtration), suspended solids gives a good indication of the pollutional strength of the sewage. TSS could be in the form of organic (fixed) or inorganic (volatile). The amount of TSS (fixed or volatile) indicates the insoluble content of the sewage. Therefore, TSS is considered as an important water quality parameter in tertiary treatment processes.

2.5.1.6 Colour

The analysis of colour in wastewater is usually based on either true colour or apparent colour. True colour is due only to matter which is in true solution, while apparent colour includes the effects of matter in suspended and colloidal states as well (Mancy and Weber, 1971). Classically, the colour of effluents have been determined by visual comparison with coloured solutions of known concentrations or with special coloured glass disks. However, in laboratories a comparison is made to standard platinum-cobalt colour solutions and the standard unit of colour is that produced by 1 mg of platinum per litre, in the form of chloroplatinate ion.

The EPA Victoria have set certain colour standards for effluents discharged to the Port Phillip Bay from the treatment plants around Victoria. This is done to improve the aesthetics of the Bay and to ensure that the Bay can be used as a pleasant environment for recreational purposes such as swimming, sailing, fishing, boating, etc. Therefore, analysis of effluent colour from WTP grass filtration bays was considered significant, since the five discharge outlets to Port Phillip Bay are licensed by the EPA to discharge effluent of certain colour standards.

2.5.2 Chemical Parameters

The chemical parameters that were monitored in this study were biochemical oxygen demand (BOD), carbonaceous BOD (CBOD) and chemical oxygen demand (COD). Concentrations of all these parameters are obtained from laboratory analyses of the field samples.

2.5.2.1 Biochemical Oxygen Demand (BOD)

BOD is the amount of oxygen consumed by bacteria in metabolising a waste (Hauser, 1996). BOD is also a surrogate measurement for biodegradable material. Biodegradable material consists of organics that can be utilised for food by naturally occurring organisms within a reasonable time period. Wastewater treatment aims to reduce the BOD of wastewater prior to discharge in both secondary treatment and tertiary treatment processes. Oxygen is a key element in biological processes. For this reason, BOD measurement is considered critical in both secondary and tertiary wastewater treatment processes.

2.5.2.2 Carbonaceous BOD (CBOD)

A representative sample of sewage may contain aerobic and facultative heterotrophic bacteria which utilise organic compounds for food, breaking them down with the use of oxygen and providing an accurate measurement of BOD in the conversion of organic carbon to carbon dioxide (Delzer and McKenzie, 1999).

The sample may also contain some aerobic nitrifying bacteria, which use carbon dioxide for food, but use oxygen to oxidise ammonia to nitrate (*Nitrosomonas* and *Nitrobacter*). This oxygen use has nothing to do with the organic strength of the wastewater, but will become incorporated into the BOD test result. Therefore, CBOD tests on the samples are conducted to measure the amount of oxygen use in the conversion of organic carbon to carbon dioxide.

2.5.2.3 Chemical Oxygen Demand (COD)

The COD is a measure of the oxygen equivalent of the organic matter content of a sample that is susceptible to oxidation (Sorensen and Jorgensen, 1993). Its measurement is also useful in secondary treatment processes as it determines the amount of air needed in aeration processes. COD oxidised compounds include non biodegradables (which BOD cannot measure), and toxics (which will inhibit the BOD test). Therefore, COD is considered as an independent measure of wastewater strength. For many waste types its possible to correlate COD with BOD. In comparison to the BOD test which takes 5 days to complete, the COD test only takes two hours to complete.

2.5.3 Microbiological Parameters

Faecal coliforms was the only microbiological parameter that was measured. Concentrations of this parameter is obtained from laboratory analyses of the field samples.

2.5.3.1 Faecal Coliforms

Faecal Coliform bacteria are primarily found in human and animal intestines and wastes. These bacteria are widely used as indicator organisms to show the presence of such wastes in wastewater and the possible presence of pathogenic (disease-producing) bacteria. *Escherichia coli* (*E. coli*) is one of the faecal coliform bacteria widely used for this purpose. Since *E. coli* is found in human and animal intestines, its presence in wastewater indicates faecal contamination.

Coliform bacteria can attach to solids and subsequent sedimentation may remove a significant percentage of virus and bacteria (Shimohara et al., 1985). Omura et al. (1985) demonstrated that coliforms were removed by absorption to sludge particles or slime. Due to the density of the grass growth, the level of slime that build up on the grass root and stems is very high. Therefore, potential for virus and bacteria attached should be quite high. Paspaliaris (1996), who conducted studies on the 'die off' of indicator organisms at the WTP, concluded that E.coli is a reliable indicator of water quality in both lagoon and grass filtration systems.

2.5.4 Nutrients

The nutrients measured in this study included, nitrogen and phosphorus. The forms of nitrogen measured in this study included organic nitrogen, ammonia nitrogen ($\text{NH}_3\text{-N}$), nitrite nitrogen ($\text{NO}_2\text{-N}$) and nitrate nitrogen ($\text{NO}_3\text{-N}$). The forms of phosphorus measured included, orthophosphate phosphorus (OP-P) and total phosphorus (TP). All forms of these nutrients, mentioned above, have been reviewed in the following sections.

2.5.4.1 Nitrogen

The removal or control of nitrogenous matter in wastewater has assumed greater significance in recent years as a means of protecting and preserving the environment (Russell et al., 1978). Nitrogen itself is a measure of the strength and quality of nitrogenous organic matter. Nitrogen compound is a key nutrient for growth in living systems, and in biological treatment processes where there is a net production of biomass, a certain amount of this material is removed by assimilation into the biomass. The forms of nitrogen, which are of greatest interest in water and wastewaters are organic nitrogen, nitrite nitrogen ($\text{NO}_2\text{-N}$), nitrate nitrogen ($\text{NO}_3\text{-N}$), ammonia nitrogen ($\text{NH}_3\text{-N}$) and total kjeldahl nitrogen (TKN).

Nitrogen compounds are becoming increasingly important in wastewater management, due to the many effects that nitrogenous material can have on the

environment. Russell et al. (1978) stated some of the environmental problems associated with the various forms of nitrogen, which are as follows:

- Nitrogen compounds are nutrients and may cause undesirable algal growths.
- $\text{NH}_3\text{-N}$ can be toxic to fish and other aquatic life.
- $\text{NH}_3\text{-N}$ and organic nitrogen in effluents can cause a dissolved oxygen demand in receiving waters.
- $\text{NH}_3\text{-N}$ is corrosive to certain metals.
- $\text{NH}_3\text{-N}$ can have detrimental effects on disinfection of water supplies.

The most significant sources of nitrogenous wastes are those sections of the chemical industry manufacturing and using nitrogen compounds, and biological sources such as human, animal and food processing wastes.

Nitrogen can be removed by chemical, physical and biological treatment processes. However, biological treatment is by far the most economical and efficient method. In biological treatment, nitrogen removal and reduction occur by nitrification and denitrification. Nitrification involves the bacterial oxidation of $\text{NH}_3\text{-N}$ into $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$, and denitrification is the microbiological reduction of the $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$ produced by nitrification, into nitrogen gas. The product of denitrification is a non pollutant gas and therefore this process is said to be the true nitrogen removal process.

(a) Organic Nitrogen

Organic nitrogen is defined fundamentally as organically bound nitrogen in the tri-negative oxidation state. It does not include all organic nitrogen compounds (Eckenfelder and Argaman, 1991). Organic nitrogen includes such natural materials as proteins and peptides, nuclear acids and urea (usually found in domestic wastewaters), and numerous synthetic organic materials. Therefore, its measurement in wastewater treatment is often undertaken together with other forms of nitrogen. Typical organic nitrogen concentrations may vary from a few hundred micrograms per litre in some lakes to more than 20 mg/l in raw sewage.

(b) Ammonia Nitrogen ($\text{NH}_3\text{-N}$)

$\text{NH}_3\text{-N}$ is formed by decomposition of organic matter (Sorensen and Jorgensen, 1993). Proteins and other nitrogenous organic matter are decomposed to simpler organic molecules such as amino acids, which again are decomposed to $\text{NH}_3\text{-N}$. Urea and uric acid, and waste products from animals are also broken down to $\text{NH}_3\text{-N}$. Nitrifying micro-organisms can use $\text{NH}_3\text{-N}$ as an energy source, as the oxidation of ammonia is an energy-producing process. $\text{NH}_3\text{-N}$ is present naturally in surface and wastewaters. It is produced largely by denitrification of organic nitrogen-containing compounds and by hydrolysis of urea.

(c) Nitrite Nitrogen ($\text{NO}_2\text{-N}$)

$\text{NO}_2\text{-N}$ is an intermediate state of nitrogen in the oxidation of ammonia to $\text{NO}_3\text{-N}$. Such oxidation and reduction may occur in wastewater treatment plants, water distribution systems, and natural waters (Sorensen and Jorgensen, 1993). Therefore, since nitrite is formed via nitrification of $\text{NH}_3\text{-N}$, its measurement in all treatment processes, especially tertiary treatment process, is desirable.

(d) Nitrate Nitrogen ($\text{NO}_3\text{-N}$)

$\text{NO}_3\text{-N}$ generally occurs in trace quantities in surface water but may attain high levels in some groundwaters. $\text{NO}_3\text{-N}$ is found only in small amounts in fresh domestic wastewaters but may be found in concentrations of up to 30 mg/l (as N) in the effluent of nitrifying biological treatment plants. It is an essential nutrient for many photosynthetic autotrophs and in some cases has been identified as the growth-limiting nutrient.

2.5.4.2 Phosphorus

Phosphorus is one of the key elements necessary for growth of plants and animals. Phosphorus in elemental form is very toxic and is subject to bioaccumulation. Phosphates (PO_4) are formed from this element. Phosphates exist in three forms;

orthophosphate, metaphosphate (or polyphosphate) and organically bound phosphate. Each compound contains phosphorous in a different chemical formula. Ortho forms are produced by natural processes and are found in sewage. Poly forms are used for treating boiler waters and in detergents. In water, they change into the ortho form. Organic phosphates are important in nature as they can be nutritious to some living organisms. In this research project, the laboratory analysis of field samples only reported phosphorus measurements in the form of orthophosphate phosphorus (OP-P) and total phosphorus (TP). The EPA Victoria effluent discharge licence requires measurements of effluent levels of phosphorus in the form of OP-P and TP.

Phosphorus is essential for the growth of organisms and can be a nutrient that limits the primary productivity of a body of water. Cellular activities (such as reproduction, locomotion and growth) require phosphate compounds as an energy source from the food consumed (Hauser, 1996). Therefore, cell life cannot exist without phosphorus energy compounds. However, excessive phosphorus in natural water bodies stimulates bacteria and algal growth. Hence, its excessive discharge into water bodies may eventually cause eutrophication of the receiving waters, making phosphorus one of the most significant water quality parameters. Therefore, its measurement and monitoring become one of the highest priorities in this project, together with the measurements of nitrogen, since both are nutrients causing eutrophication of Port Phillip Bay.

Phosphorus removal on most FWS grass bays (i.e. the bays used for grass filtration trials at WTP) is not very effective, because of the limited contact between the wastewater and the soil (Water Environment Federation, 1990). However, for SSF systems, the potential for phosphorus removal is greater than for FWS systems depending on the media and the detention time (Water Environment Federation, 1992). Bartlett (1972) stated that sewage treatment works in the U.K. were not usually designed to remove phosphorus, since the processes normally used removed other nutrients but generally have little effect on phosphorus removal. Some plants in the U.S.A. and Europe were designed specifically for the removal of phosphorus (Bartlett, 1972). Therefore, a grass filtration system may reduce nitrogen levels of the influent but not necessarily have the same level of treatment efficiency for phosphorus.

(a) Orthophosphate Phosphorus (OP-P)

Reactive phosphates, simple phosphates and sodium phosphates are the various forms of orthophosphates. Orthophosphates are the only form of phosphates that could be tested in the laboratories and they are the form that bacteria uses directly for metabolic processes.

(b) Total Phosphorus (TP)

Total Phosphorus in a sample is the total amount of phosphorus after all forms have been converted to orthophosphate. Total phosphates are commonly tested for and regulated in treatment plant effluents.

2.6 NITROGEN REMOVAL MECHANISMS IN NATURAL WASTEWATER TREATMENT

In establishing licence agreements for the discharge of treated wastewater into Port Phillip Bay, one of the major items highlighted for further investigation by the EPA Victoria, was the need to minimise discharge of nitrogen loading from WTP to the Bay. Paspaliaris (1996) reported that the WTP is the largest source of nitrogen to Port Phillip Bay, discharging around 2500 tons of nitrogen in winter while the grass filtration is in operation.

Due to the diversity of nitrogen forms, the removal of nitrogen from wastewater is a complex process involving several mechanisms as described in this section. A number of factors such as wastewater temperature, availability of dissolved oxygen (DO), the initial concentration of ammonia nitrogen in the wastewater, etc affect the nitrogen removal. Several rate coefficients govern the removal of various forms of nitrogen. Temperature affects every rate constant, changing the rate of reaction (Schnoor, 1996).

Basically, nitrogen removal can be achieved by physical, chemical or biological processes depending on the treatment process stage and the level of treatment required. However, it must be noted that many of these processes used for nitrogen removal in environmental technology are the same as the processes that take place in nature (Sorensen and Jorgensen, 1993). The biological process is commonly used as a tertiary treatment process for wastewater (Russell et al., 1978).

The main source of nitrogen entering a biological treatment such as grass filtration bays is from the influent, but there may be other sources including organic nitrogen from microbes, soil and decaying plant organic matter. The influent nitrogen includes mainly organic and ammonia nitrogen as well as small amounts of the oxidised forms comprising $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$.

Different forms of nitrogen undergo different processes (i.e. mineralisation, nitrification, denitrification, plant uptake, etc) as shown in Figure 2.1. The main nitrogen removal mechanisms are described in Sections 2.6.1-2.6.3, however, the remaining processes are described below.

Nitrogen fixation is the process by which atmospheric nitrogen gas is converted into ammonia. The ammonia is subsequently available for many important biological molecules such as amino acids, proteins, vitamins, and nucleic acids (Burns and Hardy, 1975). On the other hand, bacterial reduction is the process by which nitrate is converted to atmospheric nitrogen gas. This usually occurs as bacteria converts to bacteroids and begins to form enzymes. Another process in the nitrogen cycle is mineralisation. Mineralisation is the microbial conversion of organic nitrogen to ammonium. In this process, decomposition of organic nitrogen, proteins and amino acids occur. Eventually, nitrogen leaching occurs by downward movement of soluble nitrate into percolating water.

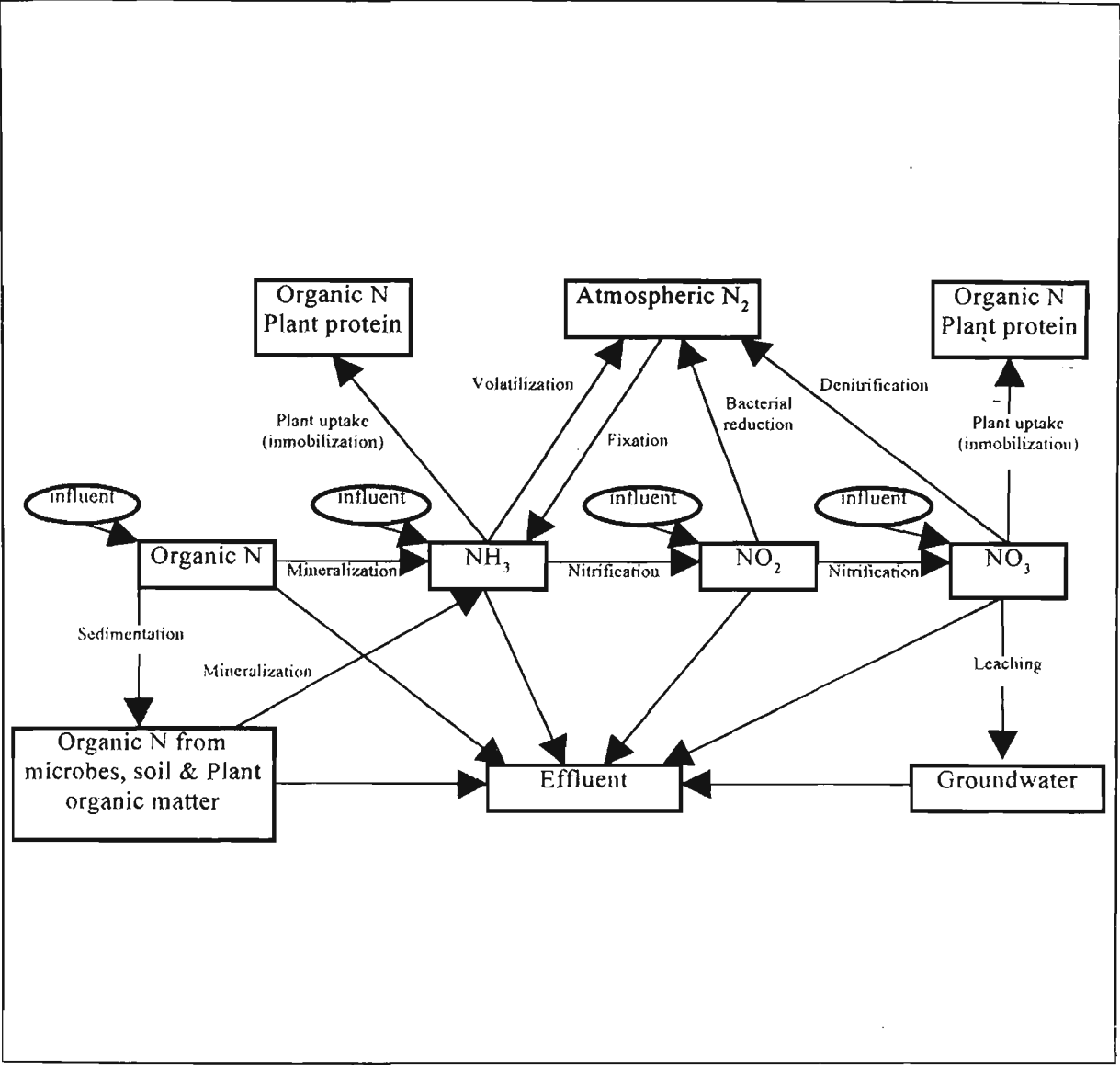


Figure 2.1 Nitrogen cycle on a grass filtration bay

Generally, the main nitrogen removal mechanisms in natural treatment processes (i.e. grass filtration) are believed to be nutrient uptake, volatilisation and nitrification/denitrification (Barnes and Bliss, 1983). While all of these mechanisms take place at grass filtration bays, the proportions of nitrogen loads removed from the wastewater are not necessarily the same through each mechanism. Green et al. (1997) studied the nitrification process in wetlands and concluded that nitrogen compounds removal in wetlands is governed mainly by microbial nitrification and denitrification, whilst other mechanisms such as plant uptake and ammonia volatilisation are of less importance. For these reasons it is necessary to understand the possible nitrogen removal mechanisms from wastewaters, so that strategies to improve their removal could be implemented.

2.6.1 Plant Uptake

Plant uptake is a process in which nitrogen is assimilated by the crops as a food source. Plant uptake of nitrogen is a common mechanism, although its contribution is generally minimal. Brix and Schierup (1989), who studied the SSF systems for municipal sewage treatment, stated that 'direct assimilation of nutrients by vegetation is considered to be of no significance for the purification ability of the system because the maximum amount of nutrients which can be removed by harvesting the above-ground biomass is less than 5% of the load on a yearly basis'.

Kerry et al. (1995) suggested that there is a need to ensure that the percentage of nutrients taken up by plants is maximised to decrease the potential for leaching, soil storage or surface runoff of nutrients. For wastewater irrigation, this is particularly important because of the high loads of nutrients applied. The chemical form of nutrients will alter the uptake of the nutrients and hence the chance of groundwater accessions.

2.6.2 Volatilisation

Volatilisation is the direct loss of ammonia to the atmosphere, usually in the form of volatile ammonia gas ($\text{NH}_3\text{-N}$). This process occurs when the pH is relatively high (greater than 7). Factors affecting volatilisation are temperature, soil pH and rainfall.

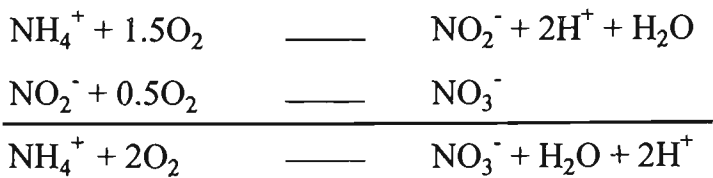
The rate of volatilization increases as pH increases until pH values of 10.5 to 11.5 when theoretically all ammonia in the wastewater is lost. Furthermore, volatilization is expected to increase with increasing $\text{NH}_3\text{-N}$ concentration. Laboratory tests on wastewater from grass filtration bays at WTP indicate that at 22°C, pH 7.3, and a detention time of 3 to 6 days, about 2 to 9 mg/l of $\text{NH}_3\text{-N}$ volatilises from an initial $\text{NH}_3\text{-N}$ concentration of 39 mg/l (Paspaliaris, 1996). Also, the work done on temperature effects on $\text{NH}_3\text{-N}$ removal (O'Farrell et al., 1973) indicates that the maximum $\text{NH}_3\text{-N}$ volatilization efficiency drops by about 25%, if temperature drops from 22.2 °C to 5 °C.

2.6.3 Nitrification/Denitrification

Nitrogen entering a biological treatment system in the organic or NH₃-N form can be either removed or transformed to another form. Removal of nitrogen is obtained by assimilation and by conversion to nitrogen gas. Sorensen and Jorgensen (1993) stated that the removal of nitrogen by biological nitrification and denitrification is a two step process. In the nitrification process NH₃-N is converted aerobically to NO₃-N, while in the second step (i.e. denitrification) NO₃-N is converted to nitrogen gas (N₂) and nitrous oxide (N₂O) by denitrifying bacteria under anoxic conditions. Nitrification first converts ammonium to NO₂-N and finally to NO₃-N, by nitrifying bacteria. The extent of nitrification that occurs during treatment is dependent on the extent to which nitrifying bacteria are present.

NO₃-N removal depends on the absence of oxygen and a good degree of contact between the wastewater and the soil (Nichols, 1983). Armstrong and Armstrong (1988) suggested that the matrix of aerobic and anaerobic environments which can develop in a wetland can achieve denitrification at the same time as nitrification.

Nitrification is an autotrophic process which means that the energy for bacterial growth is derived from the oxidation of nitrogen compounds, especially NH₃-N. Therefore, the availability of oxygen is very important for nitrification to take place. Green et al. (1997) indicated that oxidation of each mole of ammonium (NH₄-N) to NO₃-N requires two moles of oxygen. The stoichiometric equation for this reaction was hence given as:



On the grass filtration bays, DO from photosynthetic activity, which leaks to the bays through the root-water interface is the main source of oxygen for mineralisation and nitrification. At low DO levels, some nitrification still occurs producing gaseous nitrous and nitric oxides which diffuse to the atmosphere. Since the grass filtration

systems operate under low DO concentrations, it is important to understand nitrification at low DO levels. Paspaliaris (1996) reviewed the work done by Goreau et al. (1980) and stated that at low oxygen concentrations, the production rate of $\text{NO}_2\text{-N}$ reduced significantly, while the bacterial growth rate did not change severely. This meant that the production of N_2O relative to NO_2 was increased as the oxygen level was reduced.

Barnes and Bliss (1983) stated that nitrite oxidation is affected by members of the genera *Nitrobacter* and *Nitrosocystis*. *Nitrobacters* are the most extensively studied species with *Nitroagilis* being the most commonly encountered in wastewater treatment systems.

2.7 PHOSPHORUS REMOVAL MECHANISMS IN NATURAL WASTEWATER TREATMENT

Controlling phosphorus discharged from municipal and industrial wastewater treatment plants is a key factor in preventing eutrophication of receiving waters (Park et al., 1997). Therefore, monitoring of phosphorus levels in wastewaters is just as important as that of nitrogen. While both of these nutrients can significantly affect the quality of wastewaters, the cited literature in the context of wastewater treatment provides little information on phosphorus removal mechanisms in grass filtration processes. Wentzel et al. (1991) outlined plant uptake of phosphorus as the primary biological phosphorus removal mechanism. Phosphorus removal in grass filtration can also occur through sedimentation and adsorption in the soil matrix. This can lead to accumulation of phosphates within the soil matrix if wastewaters are irrigated on the same bays for a long period of time. Phosphorus removal efficiency of the system is usually affected by loading rate and detention time.

Dissolved phosphorus may be present in organic or inorganic forms and is rapidly transformed between the two. Plant uptake of dissolved inorganic phosphorus is rapid and following plant death, phosphorus may be quickly recycled to the water column or deposited in the sediments (WEF Manual of Practice, 1990). Hence the extent of

phosphorus removal is somewhat uncertain in both constructed FWS and SFS wetlands.

Matsch and Drnevich (1979) indicated that discharge of excessive amounts of phosphorus into receiving waters is particularly harmful, and probably more harmful than nitrogen. For this reason, phosphorus should be removed wherever possible in treatment plants, and especially in effluent polishing processes. Matsch and Drnevich (1979) stated that the ability of micro-organisms to accumulate phosphorus in their bodies is not a new observation. Organisms such as *E. coli* are capable of producing polyphosphate granules under proper environmental conditions. However, the bacteria is not sufficient to reduce the phosphorus content of wastewater to the desired levels (i.e. a 90th percentile of <15mg/l of phosphorus discharge to the Port Phillip Bay allowed by EPA Victoria). The work of Matsch and Drnevich (1979) also showed that the lack of DO was a major cause of orthophosphate release.

World wide experience (Bartlett, 1972; Matsch and Drnevich, 1979; Winkler, 1981; Brix and Schierup, 1989, Juwarkar et al, 1995) on grass filtration systems has shown little efficiency in TP and OP-P removals using FWS systems. Phosphorus removal in most FWS wetlands is not very effective because of the limited contact between the wastewater and the soil (U.S. EPA, 1981). However, for SFS wetlands the potential for phosphorus removal is greater, depending on the media and the detention time (Water Environment Federation, 1992). Therefore, grass filtration treatment may reduce nitrogen levels of wastewater, but will not necessarily be effective in phosphorus removal. Swindell and Jackson (1990) reported that a study conducted in Orlando, Florida, U.S.A., from 1988 to 1989 showed that over 98% of the phosphorus removal in a SFS wetland occurs within the first 11% of the system from the inlet. In the remaining section, the phosphorus removal was insignificant and in some strata there was an increase in phosphorus concentrations. Pilot studies conducted by Greenway and Simpson (1996) at Ingham and Townsville in Queensland, Australia, showed 8 and 6% reduction of total phosphorus (TP) between the influent and effluent of the wetlands, respectively.

Bartlett (1972) stated that sewage treatment works in the U.K. were not usually designed to remove phosphorus. However, some plants in the U.S. and Europe were designed specifically for removal of phosphorus. As with nitrogen, phosphorus must be present in wastewater if it is to be treated successfully by biological processes. Bartlett (1972) reported that phosphorus content should be about 1% of the BOD for efficient biological treatment of sewage. Barnes et al. (1981) concluded that biological treatment results in a transformation such that 50 to 90% of the total phosphorus remaining in the effluent is in soluble orthophosphate form, which is more readily available for growth and more easily removed by chemical treatment.

2.8 NUTRIENT REMOVAL MODELLING

Nutrient modelling is an important aspect in the design of grass filtration bays. Variables such as the detention time of the bays, influent to effluent concentration ratio of the wastewater and temperature are important parameters that should be considered in nutrient modelling. Since biological processes related to nutrient removal are similar in grass filtration bays and constructed wetlands, previous modelling work in relation to both systems are reviewed. The review is limited to two types of models, namely the first-order reaction kinetics models and the simple regression models. This is because the majority of reported work were based on these models.

In this thesis, the simple regression models are defined as linear regression models between influent and effluent, while the first-order reaction kinetics models are defined as models based on plug-flow theory and other regression type models which have parameters similar to plug-flow theory models, using data from many sites.

2.8.1 First-Order Reaction Kinetics Model

First-order reaction kinetics models based on plug flow theory depend on the hydraulic detention time and on the temperature in the system (Reed et al., 1995). The actual flow regime in constructed wetlands and grass filtration systems generally

appears to be a reasonable approximation to plug flow (Reed et al., 1995), where parameter concentrations are relatively uniform across any particular section perpendicular to the flow direction. Hence, first-order reaction kinetics models are widely used for these applications. Sometimes, they are also called plug flow models.

The general plug flow model is based on first-order reactions in which the reaction rate is proportional to the concentration of the reactant to the first power (Schnoor, 1996). This is shown below:

$$dC/dt = -k_T (C)^{1.0} \quad (2.1)$$

where C = pollutant concentration, mg/l

t = time, d

k_T = temperature-dependent first-order reaction rate constant, d⁻¹

Integrating Equation (2.1), gives the following:

$$\int_{C_o}^{C_e} dC/C = -k_T \int_0^t dt$$

$$\ln C_e - \ln C_o = -k_T t \quad (2.2)$$

where C_e = effluent nutrient concentration, mg/l

C_o = influent nutrient concentration, mg/l

t = hydraulic residence time, d

Therefore, the relationship for first-order plug flow reactors can be evaluated and re-written as shown in Equation (2.3) below:

$$C_e / C_o = \exp (-k_T t) \quad (2.3)$$

The value of k_T depends on the temperature and on the nutrient which is being removed. Reed et al. (1995) stated that the performance of constructed wetlands can be estimated with the above first-order plug flow kinetics, as shown by Equation (2.3), for nitrogen and phosphorus removal. It has been widely used in the works of Gilmour et al. (1977), Reddy et al. (1980), Moorehead et al. (1987), Tchobanoglous and Burton (1991), Kadlec et al. (1993), Altaf and Branion (1998) and Adelman et al. (1998). All authors suggested the use of the model for general design purposes. Although these authors have used the same form of the expression in their works, nutrient removal has not been the only area where it has been applied.

In Equation (2.3), the hydraulic residence time (t) can be estimated using Equation (2.4) below:

$$t = V/Q \quad (2.4)$$

where V = volume (m^3) of wastewater on the grass filtration bay (or constructed wetland), and

Q = average flow (m^3/d) through the bay, which can be considered as the average of influent and effluent flow.

It is necessary to compute the average flow through the bays so as to compensate for any losses or gains while the wastewater travels through the bays.

For general design purposes, where k_T is known, and any of the influent or effluent concentrations are to be determined, the use of Equation (2.3) is recommended as a preliminary modelling step (Reed et al., 1995). However, it can also be modified for specific sites or conditions, by evaluating the rate constant k_T via regression analyses using available data of these sites.

2.8.1.1 Nitrogen Removal Model

The complexity of the nitrogen removal processes does not allow the formulation of one universal nitrogen removal model. For example Reed et al. (1995) studied FWS

wetlands and evaluated separate expressions to model nitrification, denitrification and total nitrogen removal. Although separate expressions for these parameters have been developed, they are not independent of each other. For example denitrification is dependent to a significant degree on the level of nitrification.

The nitrification model is based on the assumption that all ammonia removal is due to nitrification without considering the plant uptake (Reed et al., 1995). The authors suggest that when the system requires ammonia removal, it is prudent to assume that all of the TKN entering the system will eventually be converted to ammonia. Although the authors have stated this assumption, it is not indicated clearly why the model is a nitrification model. Therefore, it is assumed that the primary reason for this may be due to the reduction in TKN and NH₃-N occurring by conversion of these to NO₂-N and NO₃-N. The model has the basic format of Equation (2.3). However, due to the specific nature of different influent and effluent parameters, specific temperature constants are defined for different temperature ranges. The nitrification model is shown below as Equation (2.5):

$$C_e / C_o = \exp (-k_T t) \tag{2.5}$$

where C_e = effluent ammonia concentration, mg/l
 C_o = influent TKN concentration, mg/l
 k_T = 0.0 (at 0°C)
0.1367(1.15)^(T-10) (for 1-10°C)
0.2187(1.046)^(T-20) (for 10°C +)
 t = hydraulic residence time, d

The rate constant (k_T) expressions for different temperature ranges were evaluated by plotting predicted versus actual effluent ammonia concentrations at an operational FWS system in Iowa with a detention time of 14 days. The predicted effluent ammonia was computed using Equation (2.5). However, Equation (2.5) will typically require detention time of 7 to 12 days for summer conditions and even longer for winter periods (Reed et al., 1995).. The detention time of the trial bays at WTP ranged

from 3 to 6 days. Therefore, the use of Equation (2.5) with the specified k_T ranges may not be suitable for the WTP trials.

Other nitrification models which have been derived from various sites are also presented below. These nitrification models have been developed using regression analysis of data from several sites. Hammer and Knight (1992), who studied 17 FWS systems, presented the following expression for modelling nitrification in FWS wetlands:

$$C_e = \frac{18.31C_oQ}{A_s} - 0.16 \quad (2.6)$$

where C_e = effluent ammonia concentration, mg/l

C_o = influent ammonia concentration, mg/l

A_s = surface area of wetland, m^2

Temperature adjustments are not possible with this model. However, the model does take into account the effects of hydraulic residence time and water depth, although these are not apparent from the equation itself. Since the terms Q and A_s are present in Equation (2.6), a relationship between hydraulic residence time and water depth is implicitly incorporated in the model. It must also be noted that the model is only useful for warm-weather conditions, and therefore this model is not suitable for use in the WTP trial sites. Furthermore, it does not model temperature.

It must be noted that the nitrification model reported by Reed et al (1995) (i.e. Equation (2.5)) considers influent TKN and effluent ammonia, while the other nitrification model (i.e. Equation (2.6)) as reported by Hammer and Knight (1992) only considers influent and effluent ammonia. However, the significance of incorporating TKN into the model or vice versa has not been clearly stated by neither of the authors.

The Water Environment Federation (1990) also presented an ammonia removal model which is shown as Equation (2.7) below:

$$\ln (C_e) = 0.688 \ln (C_o) + 0.655 \ln (HLR) - 1.107 \quad (2.7)$$

where C_o = influent ammonia concentration, mg/l

C_e = effluent ammonia concentration, mg/l

HLR = hydraulic loading rate, cm/d

This model was based on regression (with $R^2 = 0.77$) of performance data from 16 operational constructed and natural FWS wetlands in the U.S. This nitrification model does not allow for temperature correction. The authors do not state whether the temperature is implicitly incorporated in the model. Furthermore, the data used for the model do not show any details of temperature recorded. Therefore, it is assumed that temperature was not taken into account when deriving this model. Water Environment Federation (1990) suggest that this model should only be used in the range of ammonia concentrations of the data from which they have been derived and then only as a first approximation. The range of ammonia concentrations in the data used was between 7.2-35 mg/l and 1.5-28.2 mg/l at inlet and outlet respectively. Although the ammonia concentrations at the WTP trial bays are within these ranges, the use of the model may not be appropriate for final design since it does not allow for temperature effects to be modelled.

The denitrification model presented by Reed et al. (1995) has the same format as the nitrification model (i.e. Equation 2.5). However, the variables are different as shown in Equation (2.8) below:

$$C_e / C_o = \exp (-k_T t) \quad (2.8)$$

where C_e = effluent nitrate concentration, mg/l

C_o = influent nitrate concentration, mg/l

$k_T = 0, d^{-1}$ (at 0 °C)
 $1.000(1.15)^{(T - 20)}, d^{-1}$ (for 1 °C⁺)

t = hydraulic residence time, d

Once again, the k_T expressions for the different temperature ranges are obtained by regression as explained for the nitrification model earlier. The influent nitrate concentration used in Equation (2.8) is the difference between the influent and effluent concentrations determined with Equation (2.5) (Reed et al., 1995). This is because it was assumed that the reduction of TKN to $\text{NH}_3\text{-N}$ in Equation (2.5) was due to formation of $\text{NO}_3\text{-N}$ through nitrification. The authors suggest that the denitrification model (i.e. Equation 2.8) is only applicable for nitrate that is present within the bay. However, it is not clear whether the model is applicable to situations where excess nitrate enters the bay through the influent. For example, in most situations where grass filtration is used as the tertiary treatment process, the influent has already received secondary treatment and will contain some nitrate. This nitrate entering the system accumulates with the nitrate produced within the bays through nitrification. It must also be noted that the range of hydraulic residence time within which the model could produce accurate estimates is not specified.

The Water Environment Federation (1990), presented another model for FWS wetlands as a predictive relationship between C_e and C_o for TN. This relationship is listed as Equation (2.9) below and was developed based on regression analysis of data of wetlands described in the model of Equation (2.7).

$$C_e = 0.193 (C_o) + 1.55 (\text{HLR}) - 1.75 \quad (2.9)$$

Where C_e = effluent TN concentration, mgr/l

C_o = influent TN concentration, mgr/l

HLR = hydraulic loading rate, cm/d

Reed et al. (1995) also reviewed the work of the Water Environment Federation (1990) with regards to its denitrification model and stated that “it is not possible to adjust the results of the model for temperature, nor does it recognise the effect of depth and hydraulic residence time in the wetland, so its use for design is not recommended”. However, as it can be observed from Equation (2.9) itself, the term HLR (cm/d) is present in the model, suggesting that the water depth is implicitly accounted for. As for the temperature adjustments, the claim of Reed et al. (1995) is

correct, since the temperature does not seem to be associated within the model either explicitly or implicitly, nor it is shown within the system data.

2.8.1.2 Phosphorus Removal Model

The phosphorus removal models reported in the literature are basically models which were derived from the first-order reaction kinetics as shown in Equation 2.3. A number of investigations agree on the general first-order, area-specific model for phosphorus removal. However, there is a lack of consensus on the magnitude of the rate constant associated with the model (Reed et al., 1995). The model presented by Reed et al. (1995) is shown as Equation (2.10) below:

$$C_e / C_o = \exp (-k_p / \text{HLR}) \quad (2.10)$$

where C_e = effluent phosphorus, mg/l

C_o = influent phosphorus, mg/l

k_p = 2.73 cm/d

HLR = average annual hydraulic loading rate, cm/d

The model described by Equation (2.10) is based on the analyses of the North American Data Base of FWS constructed wetlands. The first-order rate constant (k_p) proposed for this model is 10 m/yr or 2.73 cm/d in units consistent with HLR. Although the model was developed from FWS wetland data, it should be valid for predicting the annual average phosphorus removal for both FWS and SFS wetlands (Reed et al., 1995).

Equation (2.10) has the basic form of Equation (2.3) and are implicitly related. This is because in Equation (2.3), the term t represents the detention time and is computed in accordance with Equation (2.4). In Equation (2.4), V is computed as $(A_s \times d)$. Therefore, substituting this into Equation (2.3) and considering the fact that HLR is computed as (Q / A_s) , the implicit relationship between the two equations can be observed.

2.8.2 Simple Regression Models

As mentioned in Section 2.8, simple regression models represent the second type of models, which can be applied to the grass filtration process. They are based on simple linear regression of influent and effluent concentrations. Not many such models are available in the literature. Only two models were found in the literature, but both of these were developed from experiments of a SSF system. Hence, the models, as they are, may not be relevant to FWS systems.

Breen (1990) presented such an empirical relationship of influent and effluent phosphorus loads, which is shown as Equation (2.11) below:

$$TP_{out} = 0.0155 TP_{in} + 0.0033 \quad (R^2 = 0.93) \quad (2.11)$$

where TP_{in} = influent total phosphorus, mg/bucket/day

TP_{out} = effluent total phosphorus, mg/bucket/day

A mass balance approach in Breen (1990) allowed the development of this simple predictive model. This relationship was based on experimental data from a subsurface up-flow hydraulic format and its relevance to a FWS wetland has not been established. This experimental system consisted of 10 litre plastic buckets filled with 12 kg of washed gravel with rhizome sections of *Typha orientalis* planted directly into the substratum. The influent was introduced via a central tube to the bottom of the bucket and the effluent was collected via three equidistant peripheral drainage tubes at the top of the bucket. The influent and effluent were measured in units (mg/bucket/day). Although the influent and effluent concentrations were also presented in Breen (1992) as well as mass loads, it is not clear whether the model can also be used for influent-effluent concentrations in addition to influent-effluent loads.

A similar relationship based on the same system was also developed for total nitrogen (Breen, 1990). This empirical relationship is shown as Equation (2.12) below:

$$\text{TN}_{\text{out}} = 0.0095 \text{ TN}_{\text{in}} + 0.9066 \quad (R^2 = 0.90) \quad (2.12)$$

where, TN_{in} = influent total nitrogen, mg/bucket/day

TN_{out} = effluent total nitrogen, mg/bucket/day

Using models such as Equations (2.11) and (2.12), the predictive results should be quite accurate as the variables (i.e. influent and effluent loads) are correlated closely with high R^2 values. However, these predictive models may not be valid for FWS systems as they were derived from SSF systems data.

2.9 SUMMARY

Natural wastewater treatment processes can be categorised as land treatment, wetland treatment and lagoon treatment. Land treatment methods include slow rate, rapid filtration and grass filtration, while wetland treatment consists of natural and constructed wetlands.

The review of the literature showed that natural wastewater treatment has been used and is currently being used in Australia as well as in other parts of the world. Since the processes governing wastewater treatment are similar in both constructed grass filtration systems and constructed FWS wetlands, both were considered in the review. Within Australia, the WTP located in Victoria is one of the largest treatment plants which makes use of natural treatment processes. The WTP uses land filtration in summer, grass filtration in winter and lagoon treatment all year around. Natural treatment processes such as grass filtration and wetland treatment are also being used in the other parts of Australia at various scales. In Queensland these processes are being used at the towns of Ingham, Townsville and Blackall at pilot scale and full scale operations.

Pilot and full scale projects using natural treatment processes in the other parts of the world (i.e. the U.S., Europe and Asia) have also been cited in the available literature. These include: Arcata and Alabama pilot projects, Oxelosund pilot project in Sweden and Sainik School pilot project in India. As well as these projects, there are hundreds

of other projects making use of natural treatment processes world-wide. Most of the case studies reviewed in this chapter deal with constructed wetlands.

This chapter also reviewed the significance of each water quality parameter measured in this trial study. The water quality parameters were categorised under physical, chemical, microbiological parameters and nutrients. The physical parameters consisted of wastewater temperature, pH, DO, redox potential, TSS and colour. Chemical parameters, however, included BOD, CBOD and COD. The only microbiological parameter considered was faecal coliforms. Nutrients included nitrogen and phosphorus. The review showed that all parameters were significant in grass filtration processes, especially the nutrients (i.e. nitrogen and phosphorus), since their excessive discharge directly contributes to the eutrophication of the receiving waters.

Nitrogen and phosphorus were the main focus of this project since they are the major contributors to the eutrophication of Port Phillip Bay. Therefore, the nitrogen and phosphorus removal mechanisms within grass filtration treatment process were reviewed. The review showed that nitrification/denitrification was the main nitrogen removal mechanism on grass filtration bays. Plant uptake and volatilisation were also identified as possible nitrogen removal mechanisms on the grass filtration bays. However, the magnitude of literature covering phosphorus removal mechanisms in grass filtration was not as great as for nitrogen. The major phosphorus removal mechanism cited in the literature was plant uptake. However, sedimentation and soil adsorption were also identified as possible phosphorus removal mechanisms. Although the review on nitrogen and phosphorus removal mechanisms was based on case studies on constructed FWS wetlands, the processes governing both constructed grass filtration systems and FWS wetlands are similar.

Finally the chapter examined the availability of nitrogen and phosphorus removal models in the literature in predicting the influent or the effluent concentrations in grass filtration processes. Two types of mathematical models, namely first-order reaction kinetics and simple regression models were found in the literature. The first-order reaction kinetics model uses first-order plug flow theory with appropriate decay

coefficients. The decay coefficients were determined under site specific conditions and therefore may not be transferable to other sites unless used under similar conditions. Several regression type models were also included in the review under the first-order reaction kinetics model category, since these models had similar parameters to the first-order kinetics models. The simple regression models were also derived for similar sites and simple linear regression equations were developed between influent and effluent concentrations. These models may not be transferable to other sites, for same reason as for first-order reaction kinetics models. The simple regression models reviewed were related to SSF systems and therefore may not be valid for FWS systems.

CHAPTER 3

DESCRIPTION OF THE STUDY AREA

3.1 SYSTEM DESCRIPTION FOR THE PILOT STUDY

The current grass filtration system treats approximately 300 Ml of wastewater per day in winter. In late summer, bays are given one or two irrigations of sedimented wastewater to promote germination of the grass seed deposited at the end of the previous season. The grass filtration areas need to be reconditioned every ten to fifteen years to remove weed growth, restore the vegetative filter and repair damage to the land caused by grazing animals.

The grass filtration trial site of this pilot study was operated under a system where wastewater was pumped onto the trial bays from Western Lagoon (Figure 3.1). This form of grass filtration is a tertiary treatment of wastewater which has already undergone a secondary treatment in lagoons, and significantly different from that of the current operation as previously discussed in Section 1.1 and outlined in the previous paragraph.

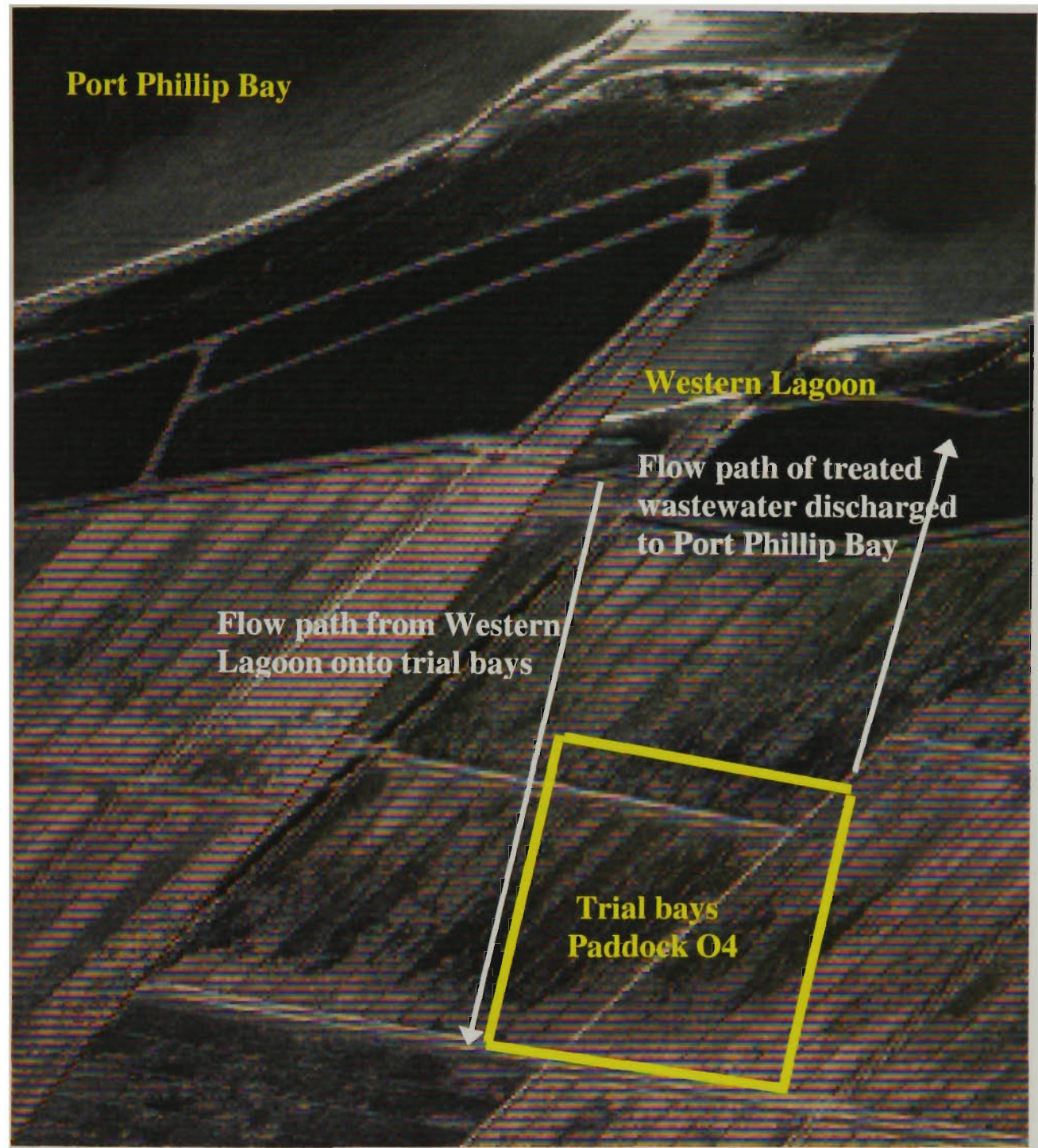


Figure 3.1 Areal Photograph of Grass Filtration Trial Site

Water from the Western Lagoon was used as the latter was relatively close to the trial bays and thus pumping and delivery costs could be minimised. However, the full scale treatment system will be irrigated with wastewater delivered via a drain called 15-East drain from an aerated lagoon further upstream in the plant flow system. Since the results from this project will assist in the design of a full scale treatment system using grass filtration as a tertiary treatment, comparisons of the wastewater quality in the Western Lagoon and 15-East Drain were carried out. These comparisons were mainly

undertaken using statistical tools to ensure that the samples came from the same populations. Results of these comparisons are presented in Chapter 7.

The wastewater was pumped continuously from the Western Lagoon to the trial bays at the rate of 10 l/s via a 300Ø mm PE pipeline (See Figure 3.2) and trapezoidal concrete lined inlet channel. It was then delivered from this channel to individual bays through rectangular concrete inlet structures which were controlled by orifice plates with 2 or 3 holes of varying diameters.



Figure 3.2 Wastewater Pumping from Western Lagoon to the Grass Filtration Trial Bays

The wastewater travelled down the bays through the Italian Ryegrass. It was then discharged to one common drainage channel via rectangular concrete lined outlet structures, which were controlled by 90° V-notch weirs. The effluent from the drainage was then discharged to Port Phillip Bay via one of the five licensed outlets (Murtcaim outlet: Figures 1.1 and 1.2). The grass filtration bays, bay outlets and common drainage channels are shown in Figure 3.3.



Figure 3.3 Grass Filtration Bays - Bay Outlets - Common Drainage Channel: (From Left to Right)

Wooden walkways constructed across the width of the bays at 200 and 300m longitudinal sections allowed easy collection of composite wastewater samples (see Figure 3.4) with minimal disturbance to the vegetation on the bays. This figure shows a photograph taken at the end of the season when the grass was grazed.



Figure 3.4 Wooden Walkways Across the Width of the Bays

3.2 SITE DESCRIPTION

The grass filtration trials were carried out on bays 1-7 of paddock O4 (Figure 3.1). Each of the bays is rectangular with checkbanks to prevent transverse flows to adjacent bays and to direct effluent outflow to a controlled section. However, the widths and longitudinal slopes are different in each bay, as shown in Table 3.1. A plan view of the bays is shown in a schematic diagram in Figure 3.5.

Table 3.1 Details of the trial bays

Bay No	Length (m)	Width (m)	Area (m ²)	Orifice type*	Longitudinal slope	Hydraulic loading rate (mm/d)
1	380	41.5	15770	A	1 in 300	20
2	380	37.5	14174	B1	1 in 300	30
3	380	38.2	14516	B1	1 in 300	30
4	380	37.4	14212	B2	1 in 300	40
5	380	34.0	12920	B2	1 in 250	40
6	380	32.0	12160	C	1 in 250	50
7	380	32.3	12274	C	1 in 250	50

*See Section 4.2 for details of orifice types

Excess wastewater in the supply channel was directed to an eighth bay (Bay 8). The inlet flow to each of the first seven bays was controlled by a different orifice plate design, while the bay outlets were controlled by identical 90° V-notch weirs. The orifice plates were designed to produce loading rates ranging from 20 mm/d to 50 mm/d. The loading rates for the trial bays are also presented in Table 3.1. Four design hydraulic loading rates of 20, 30, 40, and 50 mm/day were adopted. Each of the three higher loading rates was duplicated, while only one bay (i.e. bay 1) was loaded at 20 mm/day.

Common Inlet

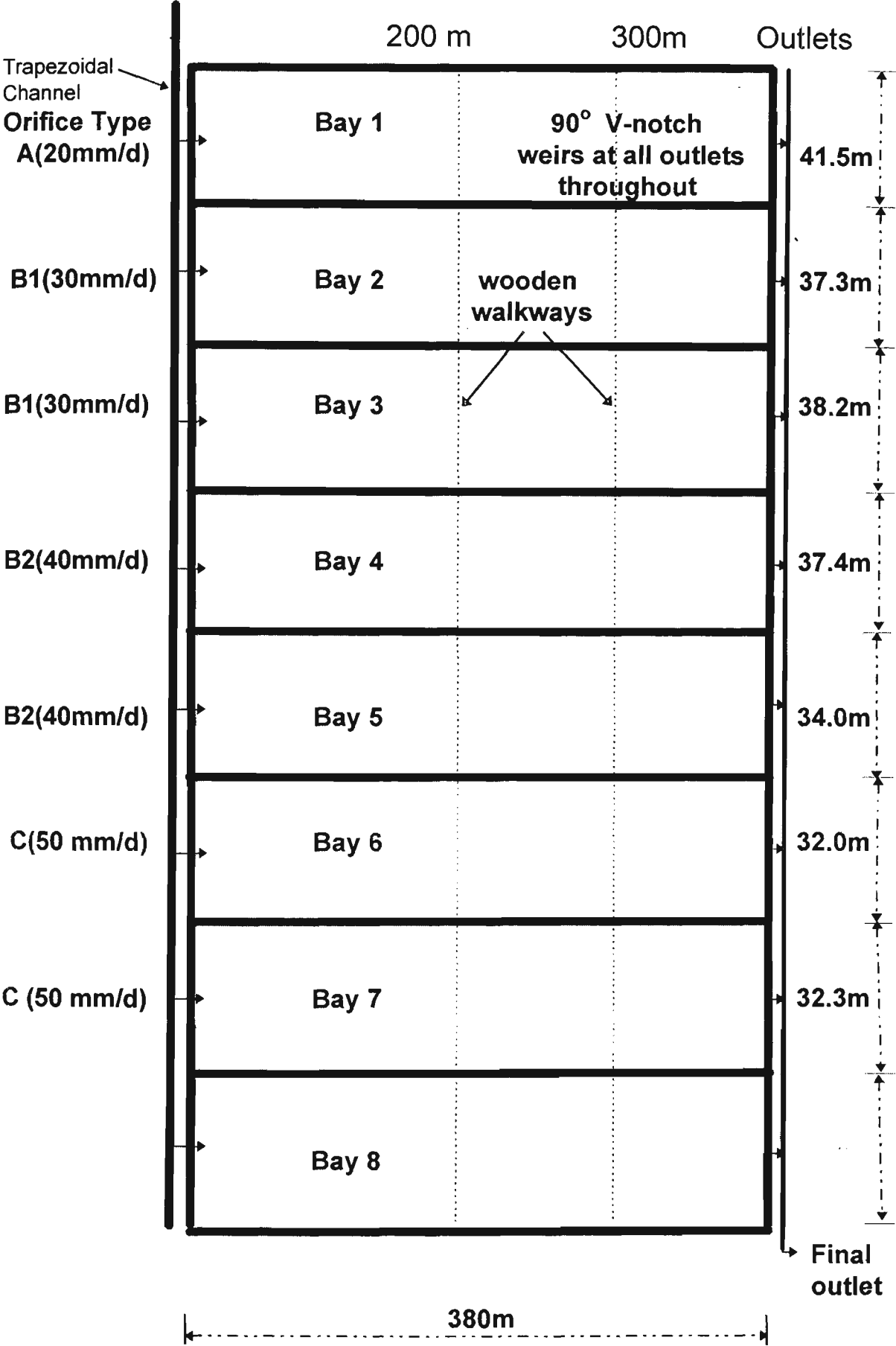


Figure 3.5 Plan of WTP Grass Filtration Trial Bays

3.2.1 Geology and Soils

In general, the geology and soils of the WTP area are closely related, resulting largely from lava flows on the western side of the WTP and the formation of the Werribee river delta plains on the eastern side of the plant. However, other features such as coastal and river alluvial deposits are also present at scattered locations. On the western side of the WTP where most of the grass filtration sites including the trial site are situated, extensive low relief basalt plains were formed during the upper Pliocene Age by large volumes of relatively fluid lava. The extensive basalt plains are generally fairly flat and gently sloping with deep soil cover and minimal rock outcrops. The soils formed on the basalt plains include light to heavy red-brown clays derived from the basalt, plus loams and sands from materials transported by wind and water.

An extensive investigation of soil types at the WTP concluded that a considerable range of soils exists within the farm area (Maher and Martin, 1952). Top soil at the site selected for the grass filtration trials consists of 250 mm of light clay with a dark brownish grey colour and a moderate medium sub-angular blocky structure. The soil has a pH of about 7.0 and permeability less than 1 mm/day. These soils have rusty root channels and are underlain by progressively heavier clays (Maher and Martin, 1952). The clays from 250 to 450 mm depth are dull and greyish in colour with a moderate amount of soft calcium carbonate and traces of fine angular and round quartz with a field pH of about 9.0 (Maher and Martin, 1952).

3.2.2 Vegetation

The vegetation used on the grass filtration trial site is Italian Ryegrass. This type of grass has traditionally been used at the WTP for many years. It produces a dense sward of stems which do not clump and require minimal maintenance. The bays are planted out two to three weeks before the start of the grass filtration season. The Italian Ryegrass produces a prolific growth of leaves that generally become denser as the filtration season progresses. The rapid growth of the grass results in its reaching

heights of up to 150 mm by mid-autumn (April) and by the end of the filtration season it grows to heights of about 500 mm with thick stems about 20mm wide.

3.2.3 Climate

The grass filtration trial site is located about 1 km from the ocean and the prevailing wind directions have a westerly component. Northerly winds are more common in the winter than in the summer when southerlies predominate with an average of 10 knots. During the winter when the grass filtration is in operation, the site usually experiences cool and windy weather with frosty mornings. A thick layer of frost often forms on the grass leaves overnight. During the period of trials (mid April - October) an average ambient temperature of 10°C and an inlet to outlet average water column temperature of 10.5°C were noted. These temperature readings were obtained at around 7 am throughout the trial period, when the in-situ samples were collected. The average annual rainfall at the WTP is 515 mm. The rainfall is generally distributed evenly throughout the year. The average annual evaporation however, is 1340 mm with peak evaporation levels occurring in summer.

CHAPTER 4

HYDRAULIC DATA COLLECTION AND ANALYSIS

4.1 INTRODUCTION

One of the major objectives of this project was to study the efficiency of the trial bays in nutrient removal from wastewater that entered the bays. This was achieved by conducting flow and nutrient mass balance computations for the bays. The purpose of the flow balance computations was to estimate the volume of water that entered the trial bays, the volume of water that left the bays and the amount which was lost through evapotranspiration, percolation and cross flows. Flow balance was necessary to conduct the nutrient mass balances which allowed estimation of nutrient removal rates. In order to perform accurate flow and mass balance computations, a substantial amount of hydraulic data had to be collected.

The hydraulic data were collected for the entire trial period (i.e. May 13 to October 9, 1997). During this period, data related to flow were collected from the inlet and outlet of each of the 7 trial bays of paddock O4 at WTP (Figure 3.5). Although inflows and outflows were constantly monitored throughout the entire trial period, on some occasions the flow data were incomplete due to either lack of equipment or its malfunction. The missing data were estimated using regression analysis. Rainfall and evaporation data were also collected at the site. Flow balance analysis was then conducted to estimate water losses and gains from the bays. In addition, tracer studies were conducted to determine the detention time of the bays in order to relate the bay inflows to the bay outflows.

This chapter presents a description of the hydraulic data collection program, the data collected from the site, flow balance analysis and tracer study results.

4.2 FLOW MEASUREMENTS

Continuous influent and effluent flow measurements are important to ensure that the trial bays are loaded at the desired levels, and to allow for more accurate estimation of wastewater losses and gains, computation of pollutant mass balances and determination of pollutant removal efficiency. These measurements also assist in explaining the behaviour of flow on the bays. Furthermore, the flow measurements at the inlet of the bays can be used to identify any cessation of wastewater flow from the supply lagoon due to failure of the inlet pump system.

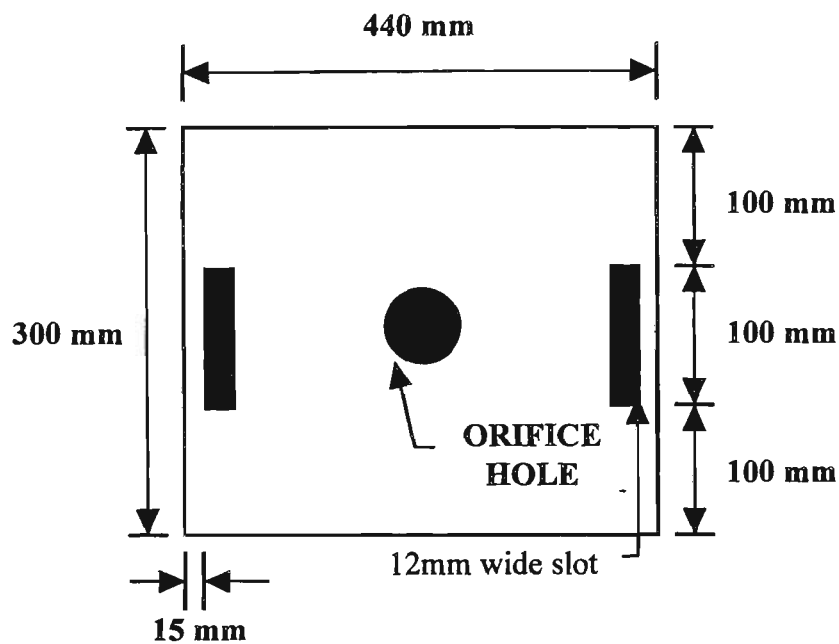
Wastewater flow onto and out of each trial bay was controlled by similar rectangular precast concrete structures (Figures 4.1 and 4.2). In order to determine the optimal hydraulic loading rate for the trial bays, four different rates (i.e. 20, 30, 40 and 50 mm/d) were considered after consultation with Melbourne Water. Therefore, to achieve these desired flow rates, the flows onto the bays were controlled by orifice plates. Each flow rate required a differently-designed orifice plate. The orifice plates were designed by Melbourne Water, with either one, two or three holes of different diameters. These orifice plate designs and details are shown in Figure 4.3 and Table 4.1 respectively. The orifice plates were made of stainless steel. They were bolted onto wooden drop boards. The bolts allowed up and down movement of the steel orifice plates, which was useful in adjusting their height relative to the bottom of the concrete inlet structure. Hence by adjusting the heights, different flows could be obtained. Similarly, the outlet flows required strict control in order to measure the outflow depths for flow balance computations. Therefore, all outlets were controlled by identical 90° V-notch weirs which were also designed by Melbourne Water. The V-notch weirs were also made of stainless steel attached to wooden drop boards and adjustable in height. The V-Notch weir design is also shown in Figure 4.3.



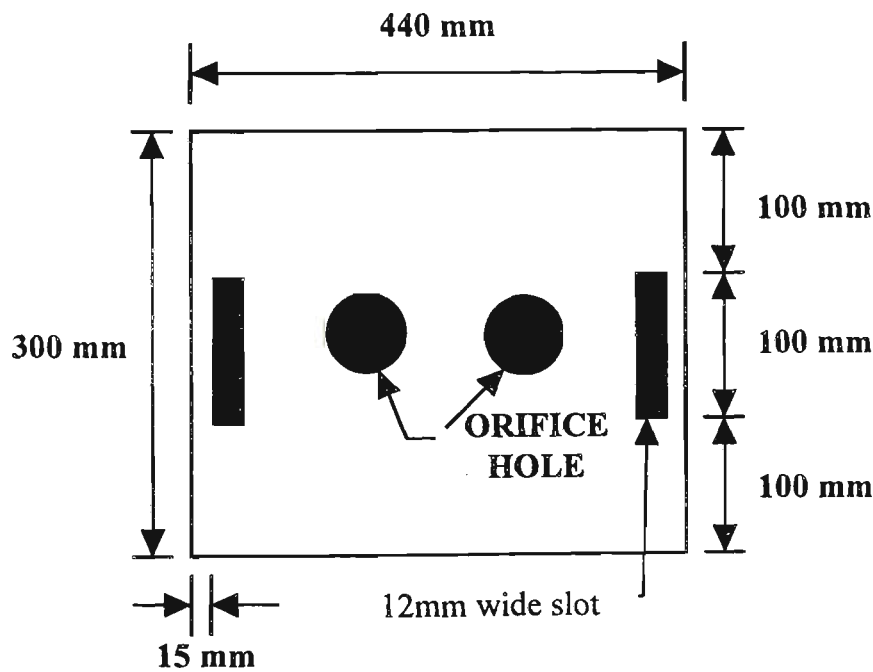
Figure 4.1 Rectangular Concrete Lined Inlet Structure with Depth Logger Installed Inside



Figure 4.2 Rectangular Concrete Lined Outlet Structure

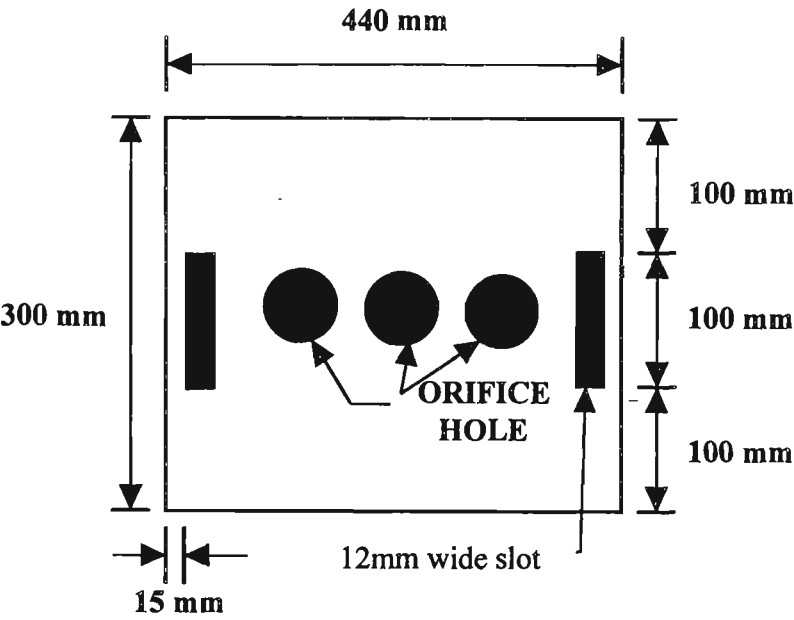


(a) Orifice Plate Type A

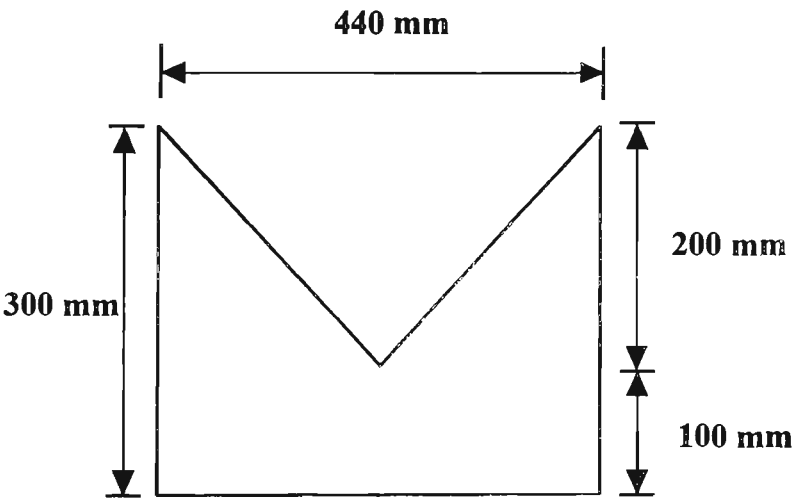


(b) Orifice Plate Type B1 and B2

Figure 4.3 Orifice Plate and V-Notch Weir Designs



(c) Orifice Plate Type C



(d) 90° V-Notch Weir

Figure 4.3 Orifice Plate and V-Notch Weir Designs (Contd...)

Table 4.1 Orifice Plate Details

Plate Type	No of Plates	Hole Size (mm)			Notes
		Left	Centre	Right	
A	1	N/A	65	N/A	One hole only
B1	2	55	NA	55	Hole centres located on horizontal centreline 80mm either side of vertical centreline
B2	2	64	NA	64	Hole centres located on horizontal centreline 80mm either side of vertical centreline
C	2	55	55	55	Hole centres located on horizontal centreline outside hole centres 105mm either side of vertical centreline

N/A = Not Applicable

Notes:

1.

Plates made from 1.6mm stainless steel (or nearest thicker size)
2.

Holes are sharp edged and free of burrs

To measure inflows and outflows of the bays, the water depths inside the control structures were measured using the Innovonics Depth Loggers that were installed at every inlet and outlet structure. These depth measurements were corrected to allow for temperature variations using temperature correction equations obtained by calibration of depth loggers. The details of this calibration procedure are given in Section 4.2.1. These depth measurements were converted into flow data using the rating curves established through in-situ field measurements of depth and flow. These rating curves for the orifice plates and the V-notch weir are explained in Section 4.2.2.

4.2.1 Calibration of Depth Loggers

Depth loggers used at the site were provided by Melbourne Water. They were dual channel loggers measuring the water depths and water temperature. The depth loggers record the water depth via a pressure sensor. The water depths measured by the depth loggers are sensitive to water temperature. Therefore, calibration of these instruments was necessary to adjust the measured depths to reflect the water temperatures of the bays.

The calibration of the loggers (serial numbers of the loggers were 21839 to 21845 at

inlets and 210536 to 210542 at outlets) was carried out by King Tech Services Pty Ltd to determine the correction factors required to compensate for temperature variation effects. Once correction factors (i.e. a and b of Equation (4.1)) were determined from calibration, Equation (4.1) was used to compute the actual water depths from the water depth reading of the logger.

$$D = [d + (aT + b)] + f$$

(4.1)

- where
- D = actual water depth at the bay, mm
 - d = water depth (above the pressure sensor) measured by the logger, mm
 - a = correction equation slope (obtained from calibration)
 - T = water temperature measured by the logger, °C
 - b = correction equation offset (obtained from calibration), mm
 - f = field offset (i.e. the vertical gap between the logger sensor and the floor of the concrete inlet and outlet structures), mm

The results of the depth logger calibration are listed in Table 4.2, which shows the values of a and b for both inlet and outlet loggers. These correction factors are valid for temperatures between 5°C and 20°C.

Table 4.2 Data Loggers Calibration Results for Variations Due to Temperature Effects

Calibration Results for Inlet Loggers				Calibration Results for Outlet Loggers			
Inlet	Logger Serial Number	a	b	Outlet	Logger Serial Number	a	b
1	21839	-0.75	+15	1	210536	-1.55	+31
2	21840	0.64	+40	2	210537	-1.05	+21
3	21841	0.47	+6	3	210538	-1.05	+21
4	21842	0.99	+6	4	210539	-1.25	+25
5	21843	1.63	-5	5	210540	-1.50	+30
6	21844	0.21	+48	6	210541	-1.65	+33
7	21845	0.37	-3	7	210542	-1.20	+24

The field offset (f) in Equation (4.1) was slightly different at every inlet and outlet, and therefore, they were measured for use in Equation (4.1).

Once the calibration of depth loggers was finished, they were installed at the inlet and outlet of all bays. However, they were not in place at all inlets throughout the trials. The depth loggers were initially installed only at the inlet of bay 1 and all bay outlets. However, the depth loggers for the remaining bay inlets (i.e. inlets of bays 2 to 7) were installed on the 25th of August 1997 since the suppliers could not supply all depth loggers at the start of the trials. Significant effort was therefore made in computing these missing inflow data, the details of which are provided in Section 4.2.4.

4.2.2 Calibration of Orifice Plates and V-Notch Weirs

Since the approach conditions of all bay inlets and outlets were identical, only one bay inlet was used to calibrate all orifice plates and the V-notch weir. This eliminated carriage of all equipment between every bay inlet and outlet. An in-situ volumetric flow measurement method was used for the calibration. In this method, the time taken to collect a known volume of wastewater is recorded, and then the flowrate can be computed. The calibration process involved several measurements of flow released through the orifice plates and V-Notch weir at different heads of wastewater at the bay inlet.

Several pieces of equipment were used to perform the in-situ calibration of the orifice plates and V-Notch weirs. These included:

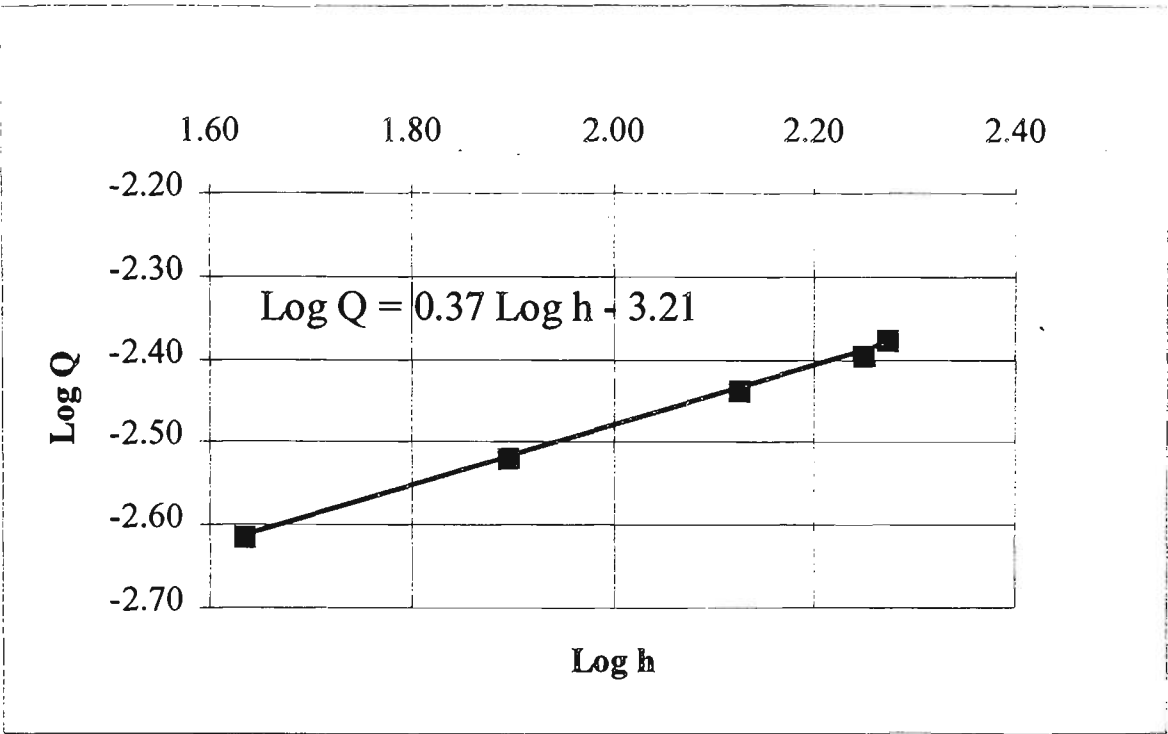
- a 400 litre capacity rectangular steel tank
 - a steel chute to transfer water from the controlled inlet of the bays to the tank
 - 2 pumps to pump water out of the tank after each measurement
 - a power generator to supply power for the pumps
 - an expandable control gate to block wastewater flow while plates were interchanged
-

The steel chute was positioned between the orifice plate (or V-notch weir) and the 400 litre steel tank, to transfer wastewater into the steel tank without any spillage. The tank was equipped with a glass sight tube on the side for accurate measurement of the depth and hence volume of wastewater in the tank. The power generator was positioned nearby for supply of power to the two pumps which pumped the collected wastewater out of the tank after each trial. However, due to the large volume of wastewater in the inlet channel, the wastewater initially flowed freely as the plates were interchanged and flooded the area where the 400 litre steel tank was positioned. This in turn made it impossible to read the volume gauge on the side of the steel tank and hence caused lengthy delays of calibration works. Therefore, an expandable control gate was designed and used to block the flow of wastewater while the orifice plates were interchanged. The gate was rectangular in shape, designed to fit freely inside the concrete inlet structure, and contained a rotating handle on top, connected to an expanding mechanism inside (Figure 4.1). Once the gate was vertically positioned, just upstream of the orifice plate in the inlet structure, the rotating handle on top was used to expand the sides, which completely stopped the flow.

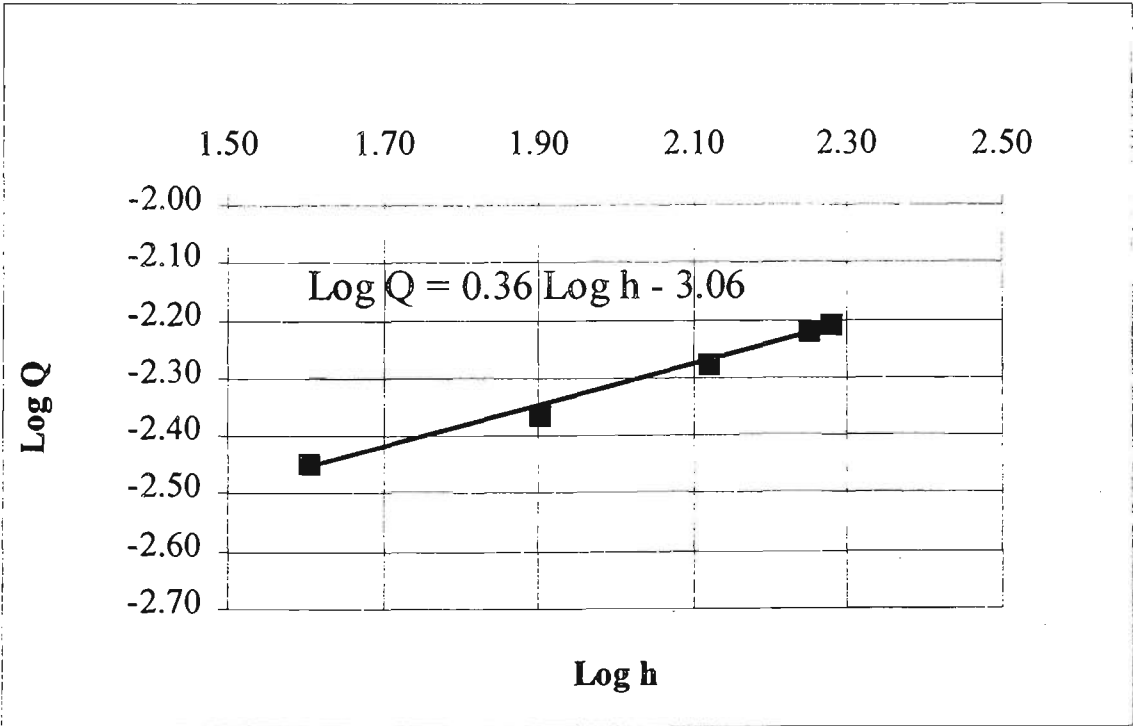
4.2.2.1 Orifice Plates

As stated earlier in Section 4.2, four different orifice plates (i.e. types A, B1, B2 and C) were used in the trials. Each type was calibrated separately for several flow depths (which correspond to different discharges), and the time taken (to the nearest second) to collect 300 litres of wastewater was recorded and discharge computed. Several such measurements were performed for each depth and an average flowrate was obtained for the depth.

The rating curves were produced by plotting the flowrate (Q) against the head (h) of wastewater above the orifice centreline. These are shown in Figure 4.4. The logarithmic values of Q and h were plotted since the equation for flow through an orifice plate is defined by a power function. The rating curves were then determined using linear regression analysis. In Figure 4.4, Q and h were expressed in m^3/s and mm respectively. The regression analysis showed the coefficient of correlation (R^2) values of 0.99 for all orifice plate types.

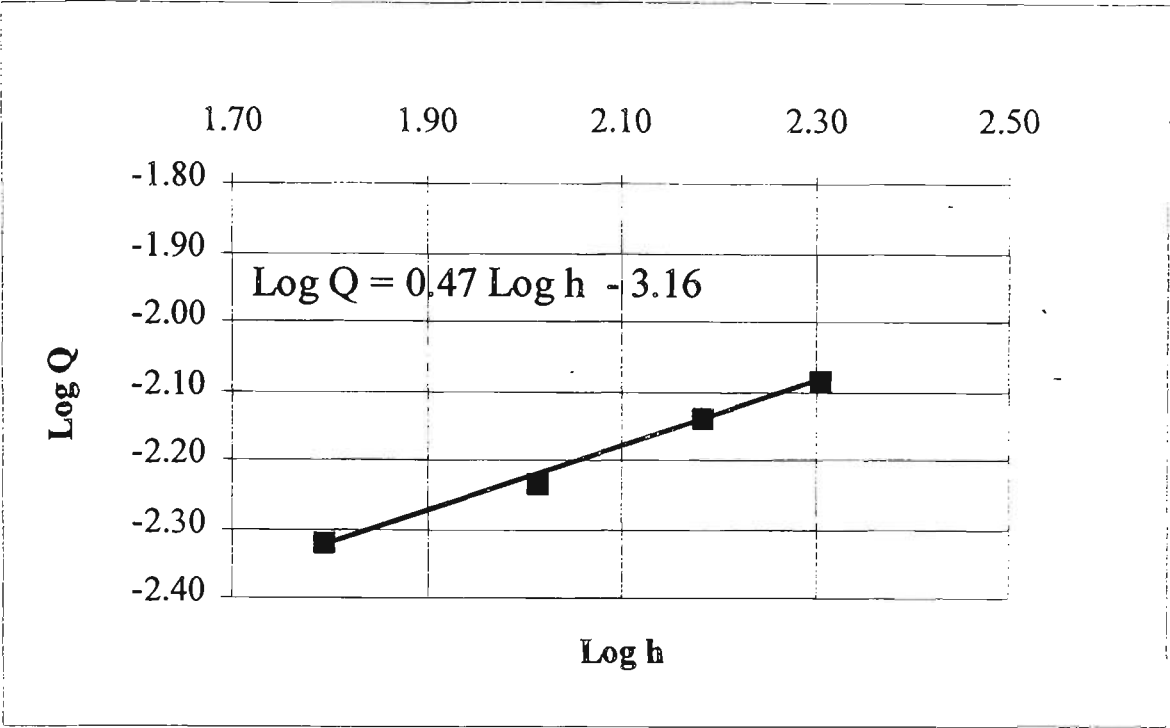


(a) Type A

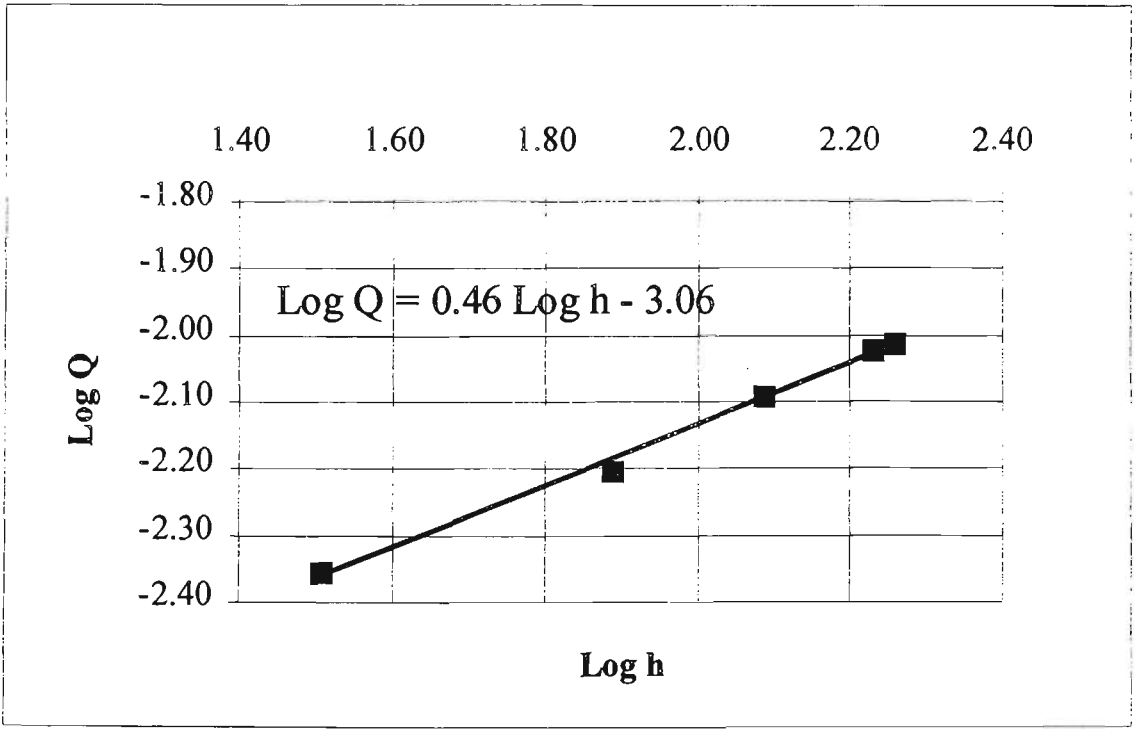


(b) Type B1

Figure 4.4 Rating Curves for Orifice Plates



(c) Type B2



(d) Type C

Figure 4.4 Rating Curves for Orifice Plates (Contd ...)

4.2.2.2 V-Notch Weir

The V-Notch weir calibration was carried out similarly to that for the orifice plates (Section 4.2.2.1), and the rating curve is shown in Figure 4.5. As seen in Figure 4.5, the points plotted show very minimal scatter. The R^2 value of 0.98 for this regression also confirms that the rating curve obtained from the calibration was accurate. Since the equation for flow through the V-notch weir is defined by a power function, the logarithmic values of Q and h were plotted. In Figure 4.5, Q and h were expressed in m^3/s and mm respectively.

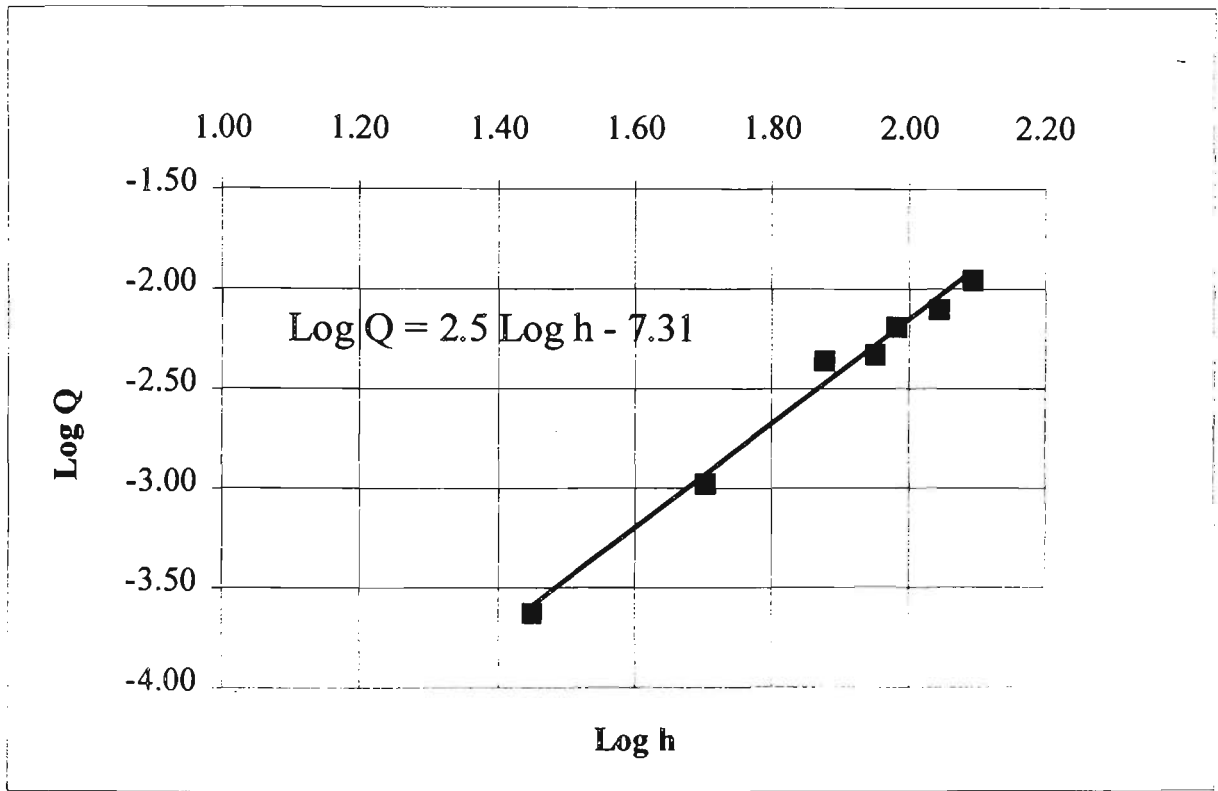


Figure 4.5 Rating Curve for V-notch Weir

4.2.3 Depth Measurements and Flow Computations

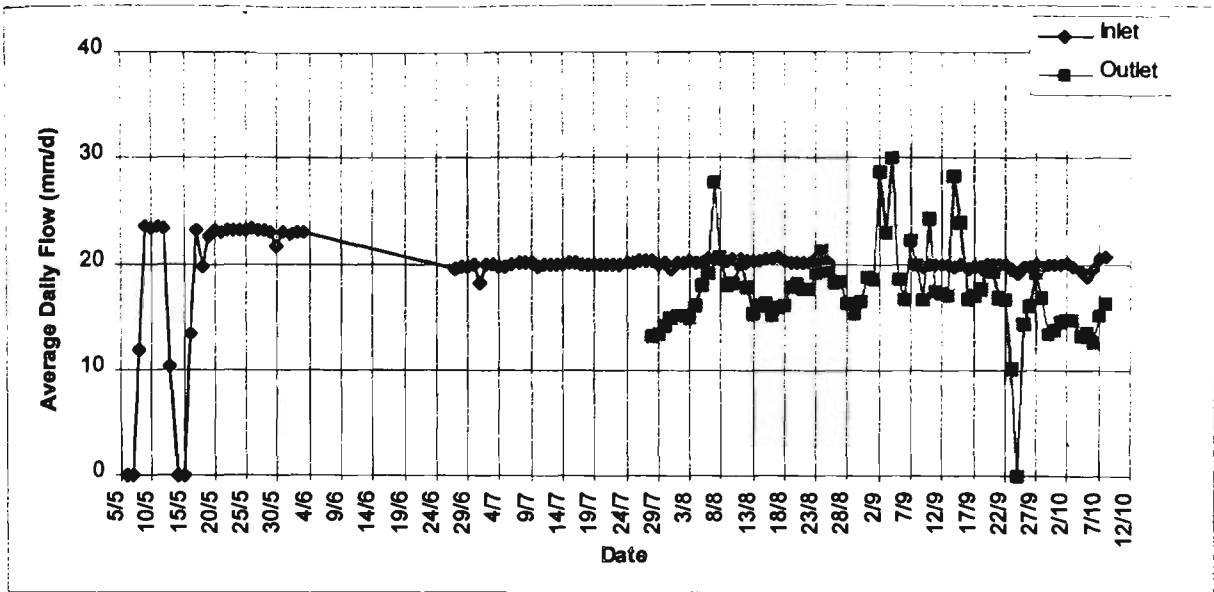
During the initial 4-week period of monitoring, it was found that all seven bays received more wastewater than required. Therefore, appropriate settings for the orifice plates were determined to achieve the desired hydraulic loading rates of 20, 30, 40 and 50 mm/day on the appropriate grass filtration bays. These settings were communicated to Melbourne Water staff (on 6 June 1997) who then adjusted the orifice heights relative to the bottom of the inlet structure in order to get the required

flow. The orifice plates were further adjusted on 9 June and 17 June, to account for some errors which occurred due to windy conditions during the setting of the plates in position.

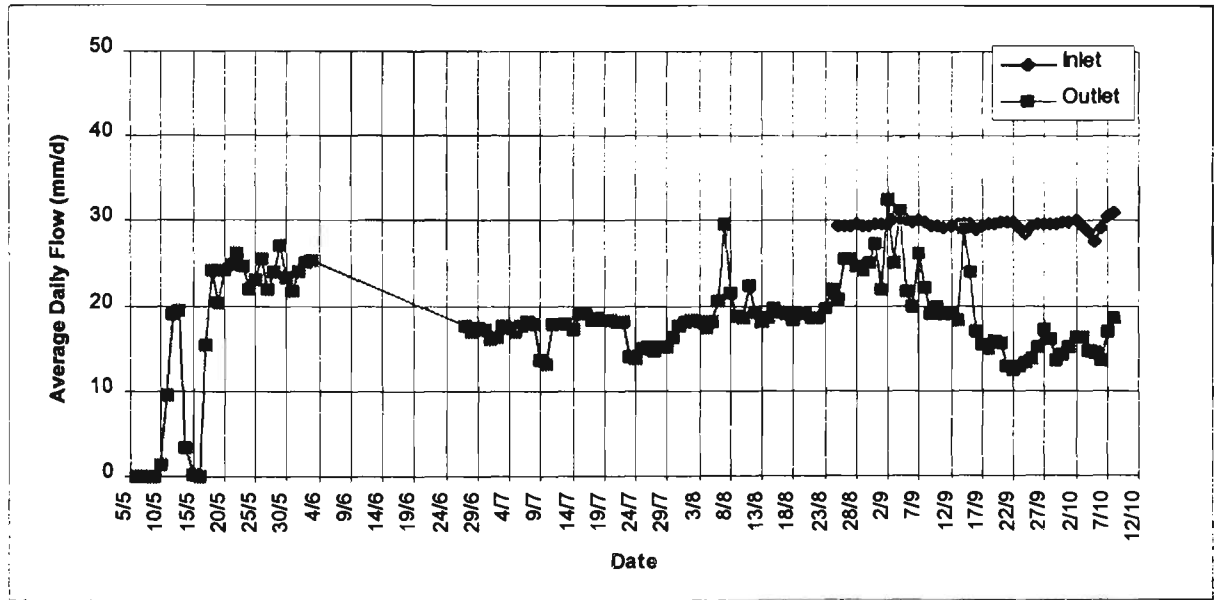
Water depths at each inlet and outlet were measured continuously at 10-minute intervals and the data from the loggers were downloaded using a lap top computer (Figure 4.1). Temperature corrections were applied to these depth measurements using Equation (4.1) and calibration values given in Table 4.2. The depths were then converted to flows using corresponding rating curves (i.e. Figures 4.4 and 4.5). These flow data were averaged for each day to represent average daily flow and are shown in Figure 4.6. The effects of temperature corrections were tested using data for bay 1 for the period from 9 September to 8 October and were found to be approximately 6% which is significant.

In addition to the data measurements from the data loggers, the depths at inlets and outlets were also measured manually using a steel ruler at every sample collection day. These manual measurements were then compared with the values recorded by the data logger to detect any equipment malfunction or errors.

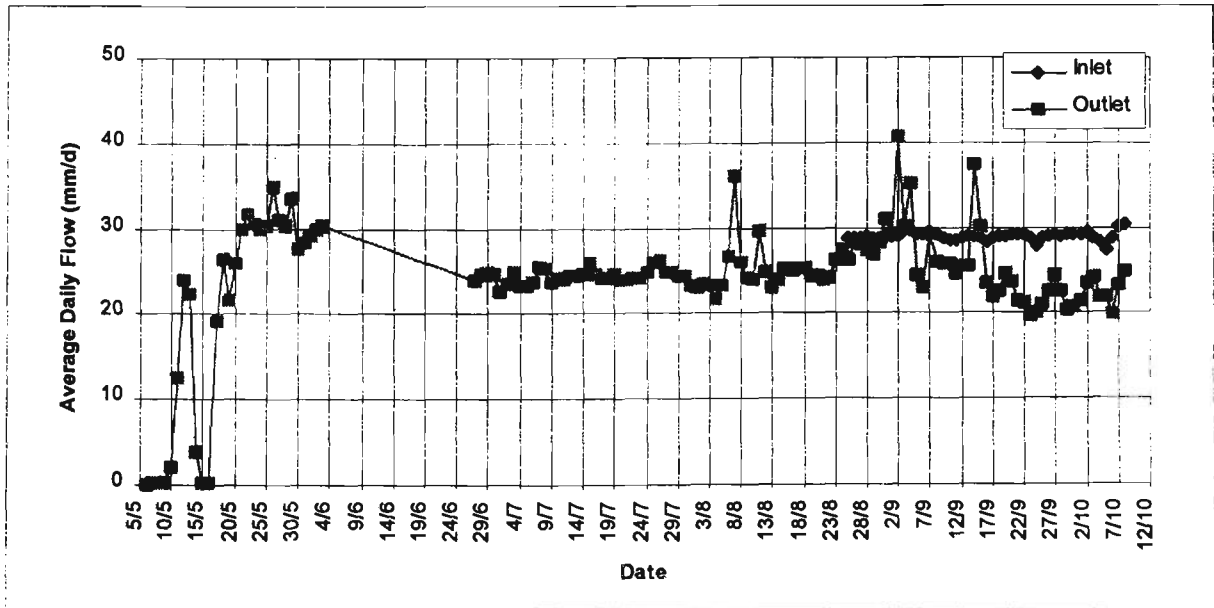
The inlet and outlet daily flow series in Figure 4.6 indicate missing data from 4 June to 26 June, which appears as a straight line. During this period the data logger memory of each logger was full, which automatically stopped the loggers from recording water depths. Also due to the delays in the supply of data loggers, water depths were recorded only at the inlet of bay 1 for the period between 5th of May and 25th of August. It can be seen in Figure 4.6 that the flow data at inlets of bays 2 to 7 are only available from 25th of August. Flow data were also not recorded at inlet of bay 1 for the period from 26 August to 8 September (Figure 4.6a) due to a faulty data logger. The missing flow data at the inlets were estimated by different means which are discussed in Section 4.2.4.



(a) Bay 1

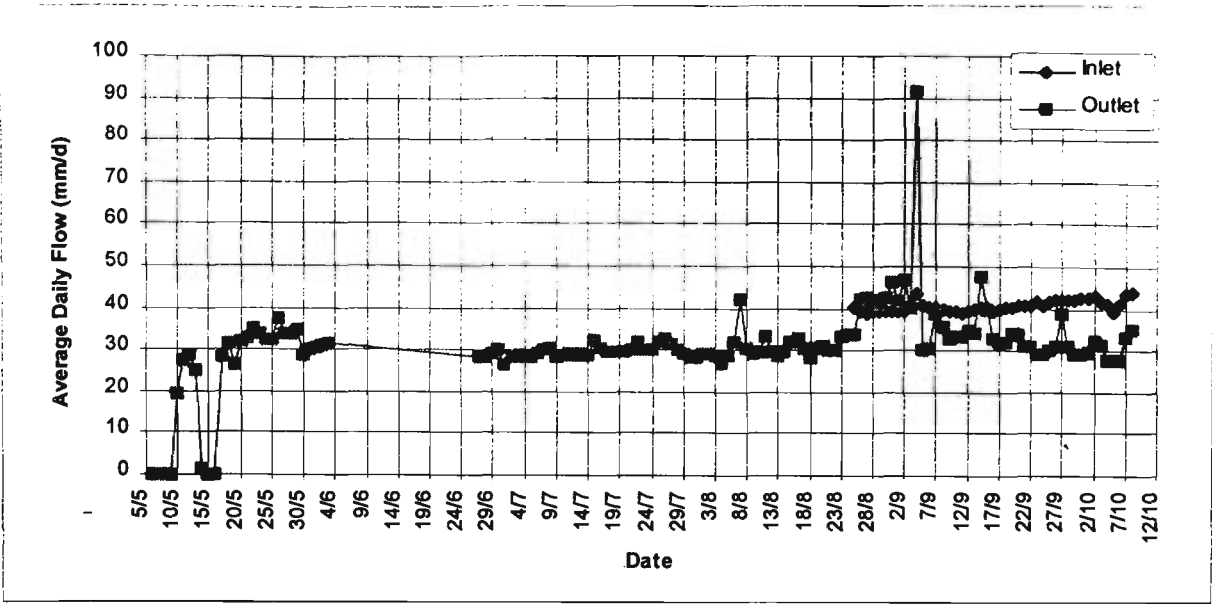


(b) Bay 2

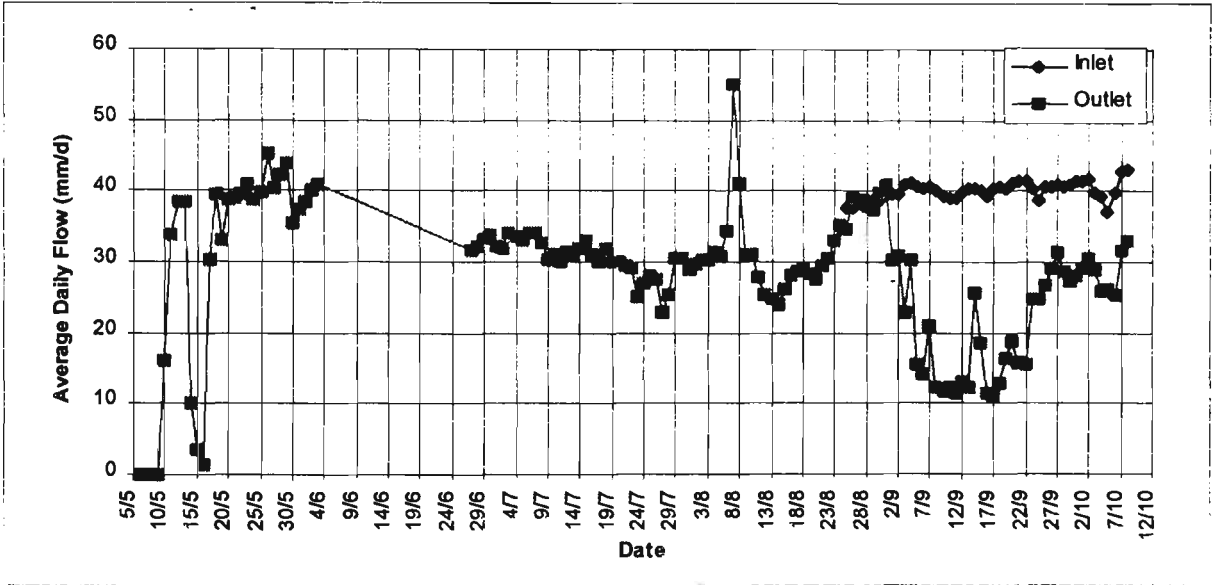


(c) Bay 3

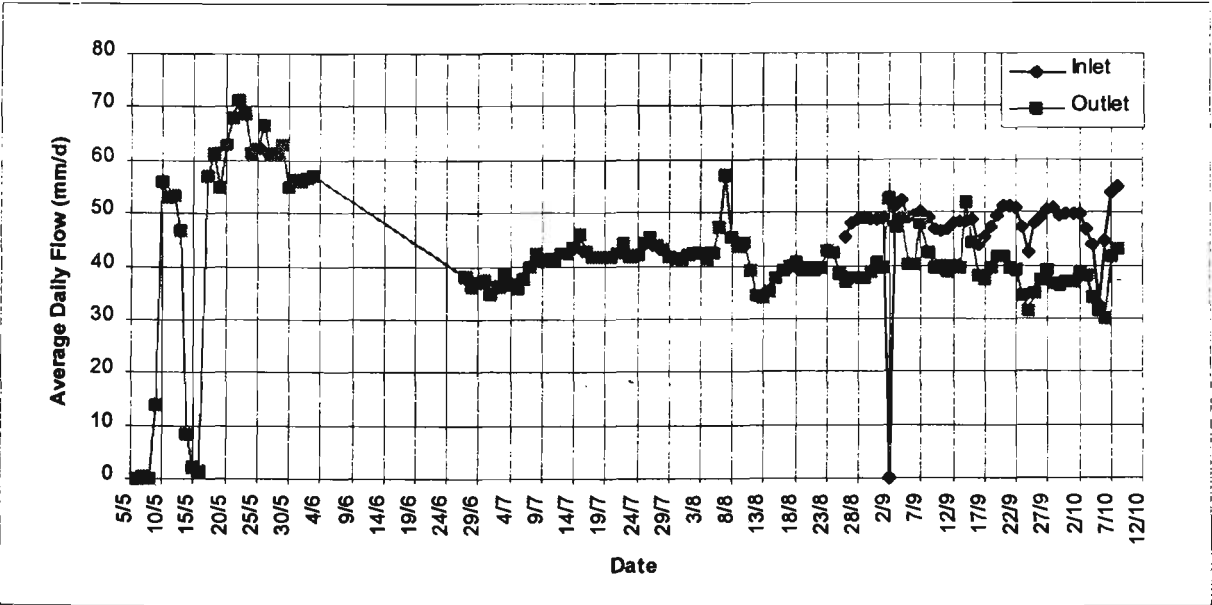
Figure 4.6 Inflows and Outflows of Bays



(d) Bay 4

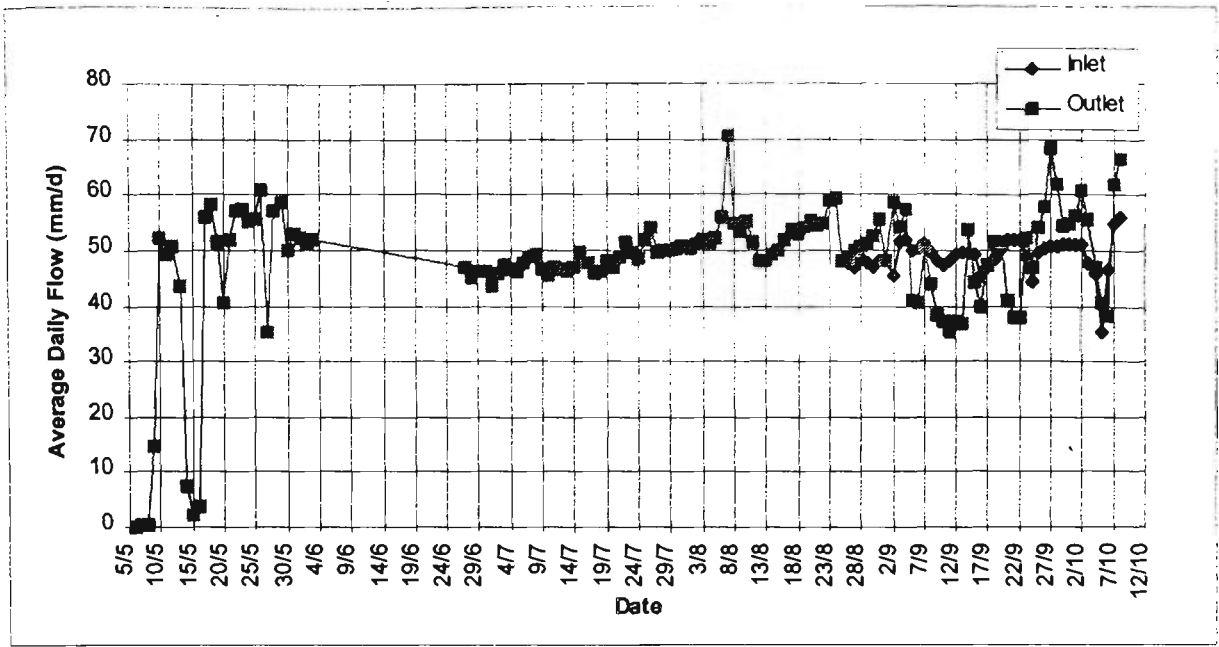


(e) Bay 5



(f) Bay 6

Figure 4.6 Inflows and Outflows of Bays (Contd...)



(g) Bay 7

Figure 4.6 Inflows and Outflows of Bays (Contd...)

The outflows showed more fluctuation than the inflows towards the end of the season particularly on bay 5 (see Figure 4.6). This fluctuation in bay 5 outflows could not be explained. Figure 4.6 also shows some flow rates which are out of range (i.e. as little as 0 and as high as 90 mm/d on bay 4). These flows may have resulted from errors in the measurements of depths by the data loggers.

4.2.4 Estimation of Missing Flow Data

As stated earlier in Section 4.2.3, due to delays from the suppliers, the data loggers were not installed at inlets of bays 2 to 7 between 5 May and 25 August (Figure 4.6). Similarly flow data were not recorded at inlet of bay 1 for the period from 26 August to 8 September (Figure 4.6a) due to a faulty data logger, which was sent away for repairs. Therefore, these missing flow data had to be estimated.

To obtain these missing flow records, regression analyses were performed between inlet flows of bay 1 and those of each of bays 2 to 7, using concurrent data from 9 September to 8 October. Inlet flows to bays 2 to 7 were plotted (Figure 4.7) against the inlet flow to bay 1 and regression equations developed. In computing these missing flow records, it was assumed that there was a strong correlation between the

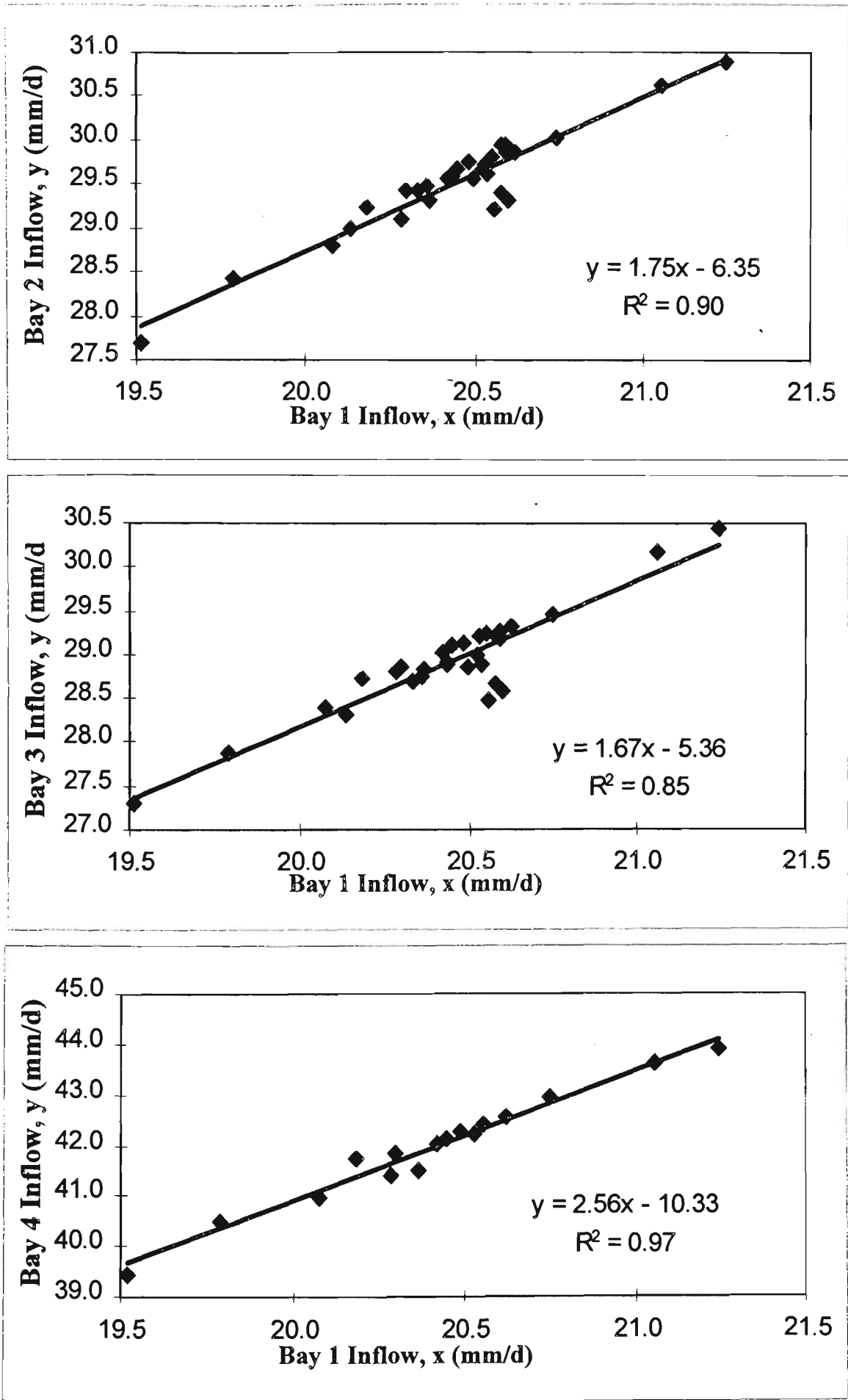


Figure 4.7 Regression Scatter Plots for Flows in Bays

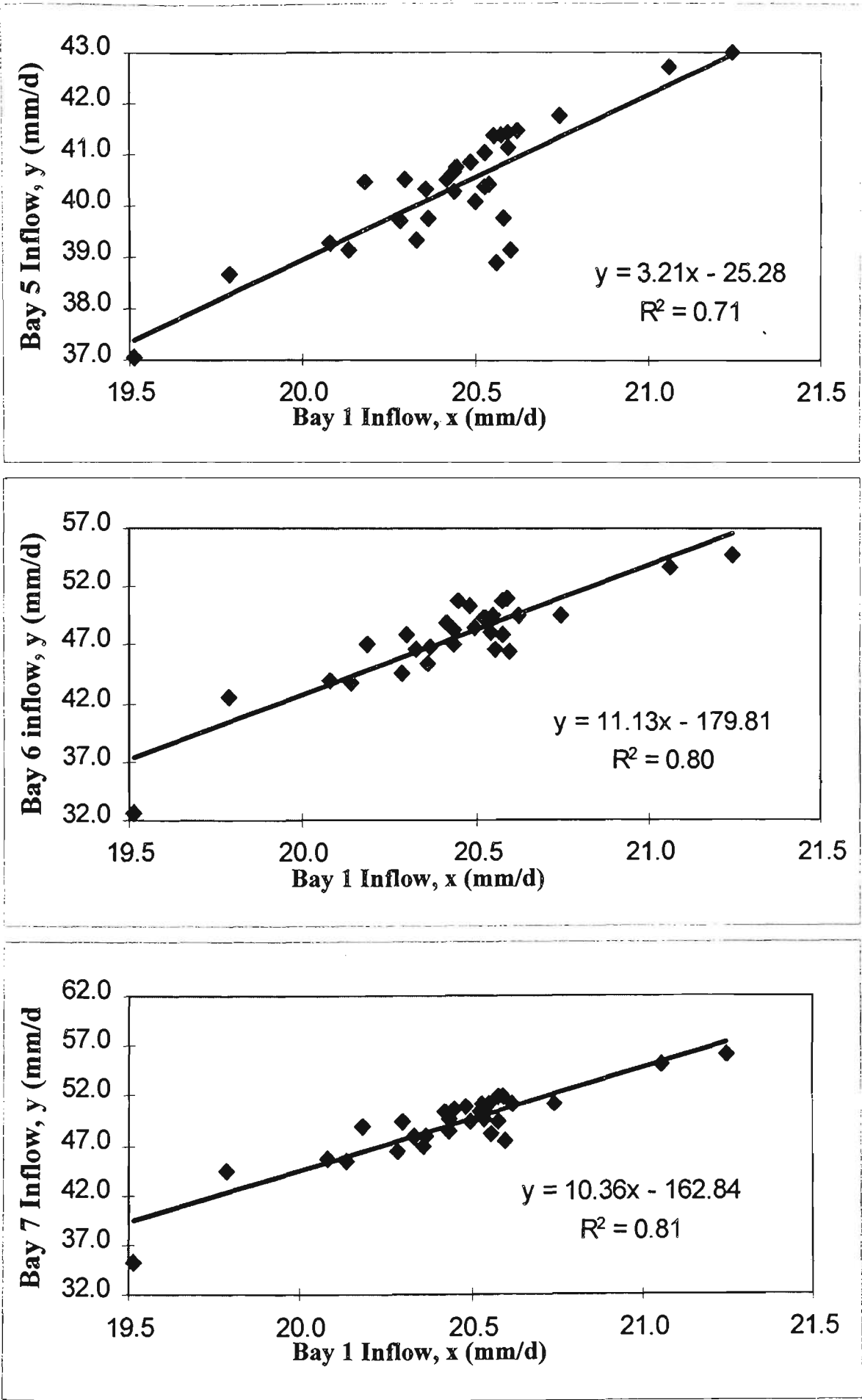


Figure 4.7 Regression Scatter Plots for Flows in Bays (Contd...)

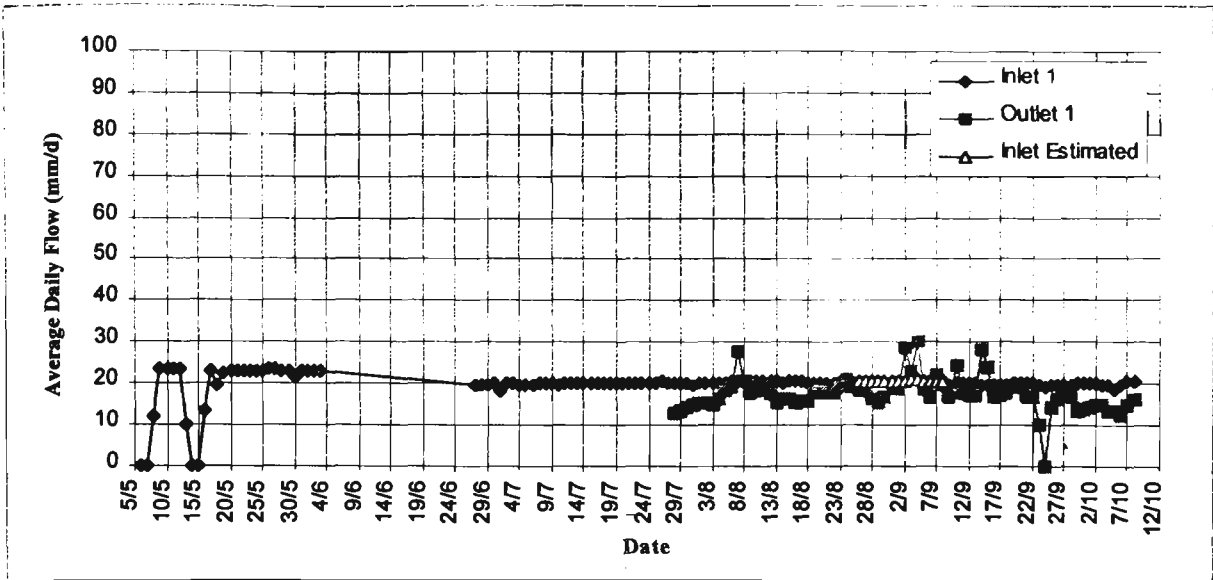
inlet flows of bay 1 and other bays. This assumption was reasonable since wastewater was fed from one supply channel to all bays simultaneously at all times during the trials, having a narrower range for the head above the orifice at each bay. The regression lines and their respective equations are also shown in Figure 4.7. This figure clearly shows that the regression analysis is quite acceptable as the R^2 values ranged from 0.71 to 0.97. The plotted points also show a low scatter.

Due to the malfunctioning of data loggers at the outlet of bay 1, no water depths were recorded from 5 May to 28 July at the outlet of this bay (Figure 4.6a). Outlet flows of bay 1 for this period could not be estimated since there was no correlation between the outflows of the bays. As seen in Figure 4.6, outlet flows for all bays are missing between 4 June and 26 June, appearing as a straight line on the figure. They could not be estimated for the same reason.

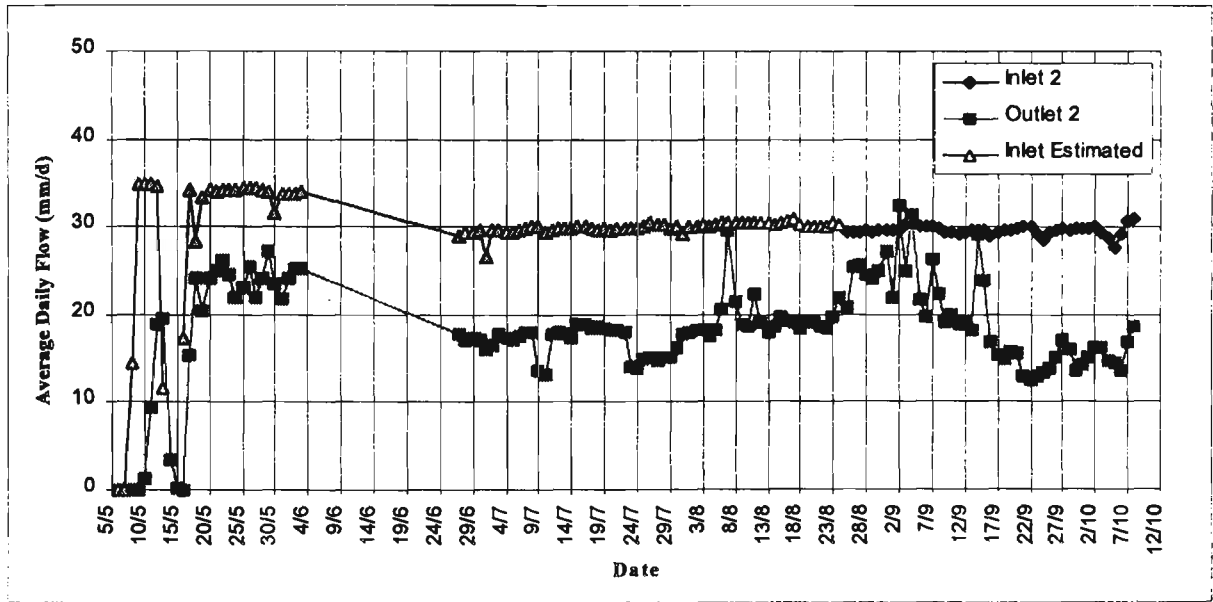
The regression equations derived from the analysis of the inlet flows (Figure 4.7), were then used to calculate the missing data from 5th of May to 25th of August for inlets of bays 2 to 7, using the flow data at the inlet of bay 1 for that period. The missing inflows of bay 1 from 26 August to 8 September were also estimated using these regression equations. All the missing inflows were then plotted on the same axis as the flows recorded by the data loggers, as shown in Figure 4.8.

4.3 RAINFALL AND EVAPORATION MEASUREMENTS

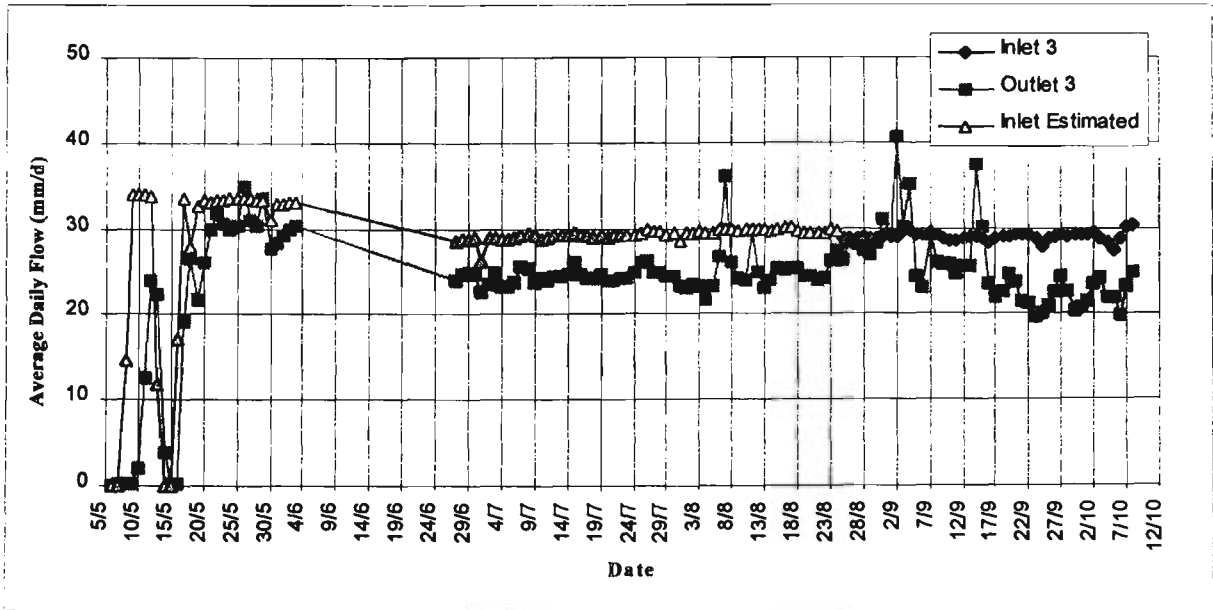
In order to perform flow balance computations, it is necessary to consider rainfall on the bays and actual evapotranspiration from the bays. Rainfall can be measured at the site. The actual evapotranspiration from the bays can be estimated as potential evapotranspiration (PET), since the bays are completely covered by grass and wastewater is present in the bays during the whole trial period, allowing maximised soil moisture. PET can be estimated from pan evaporation as described later in this section. Pan evaporation was measured at the site.



(a) Bay 1

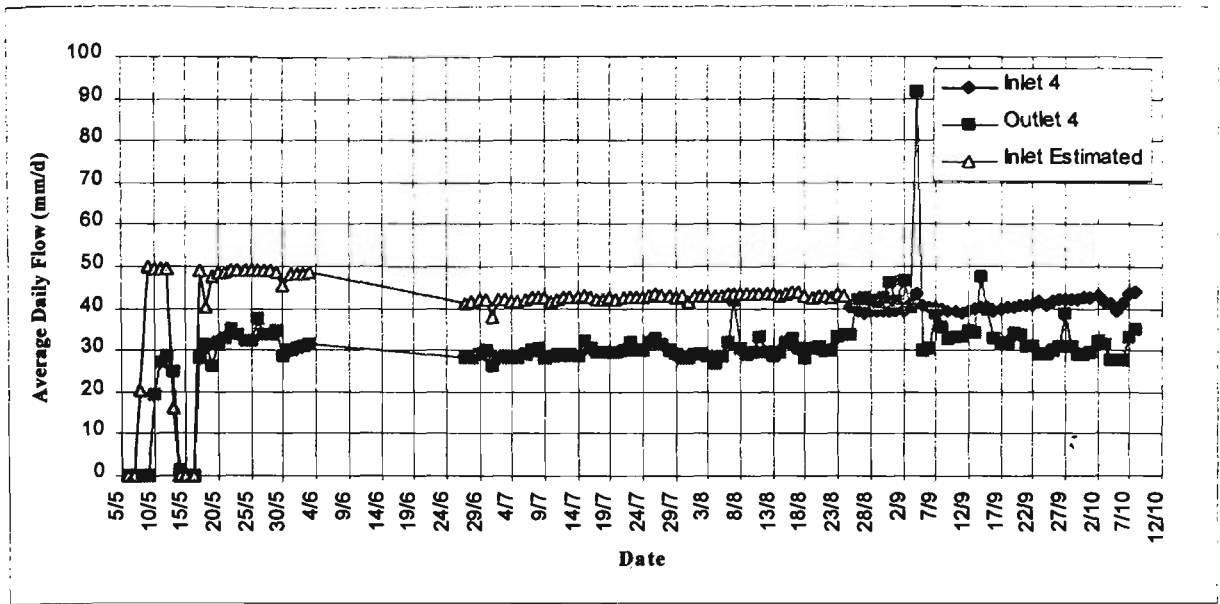


(b) Bay 2

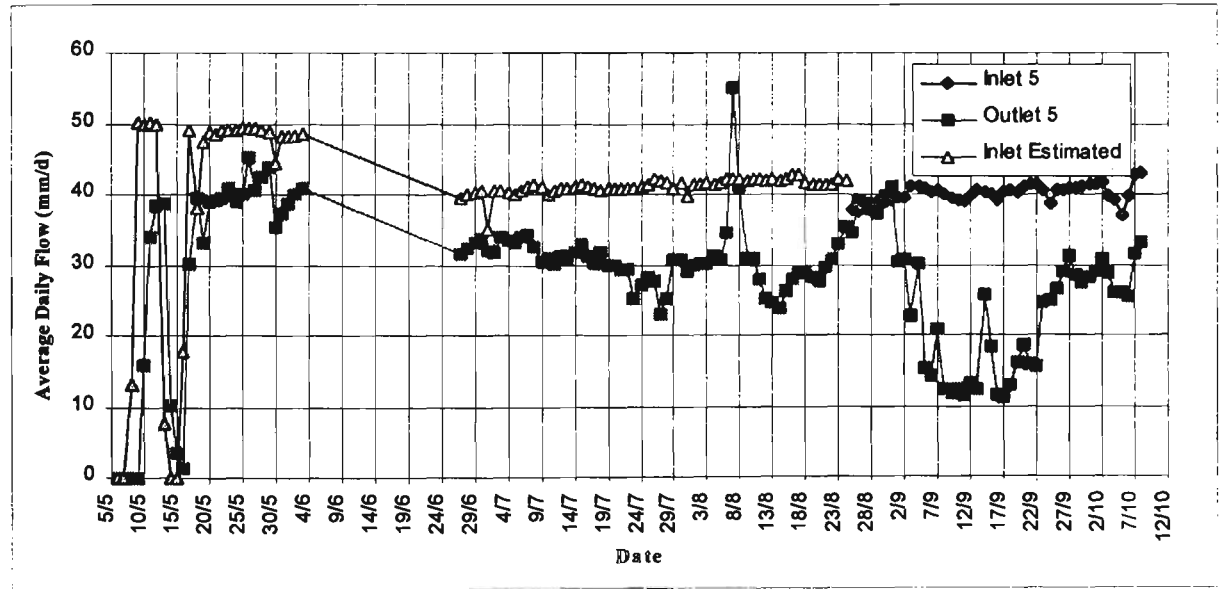


(c) Bay 3

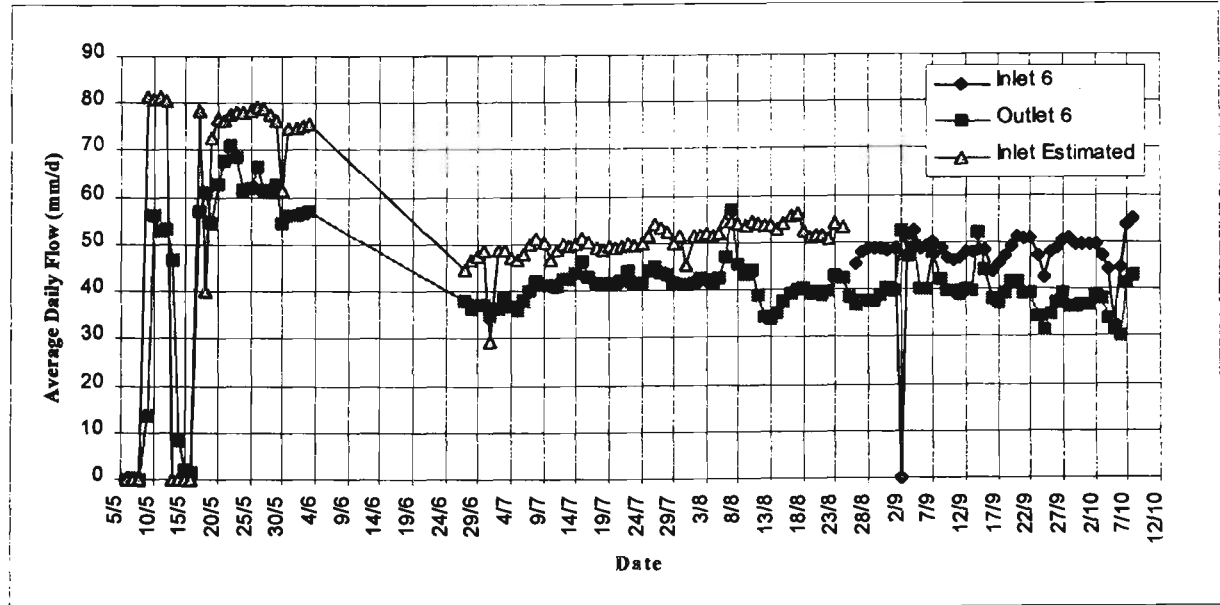
Figure 4.8 Time Series Plots of Inflows and Outflows



(d) Bay 4

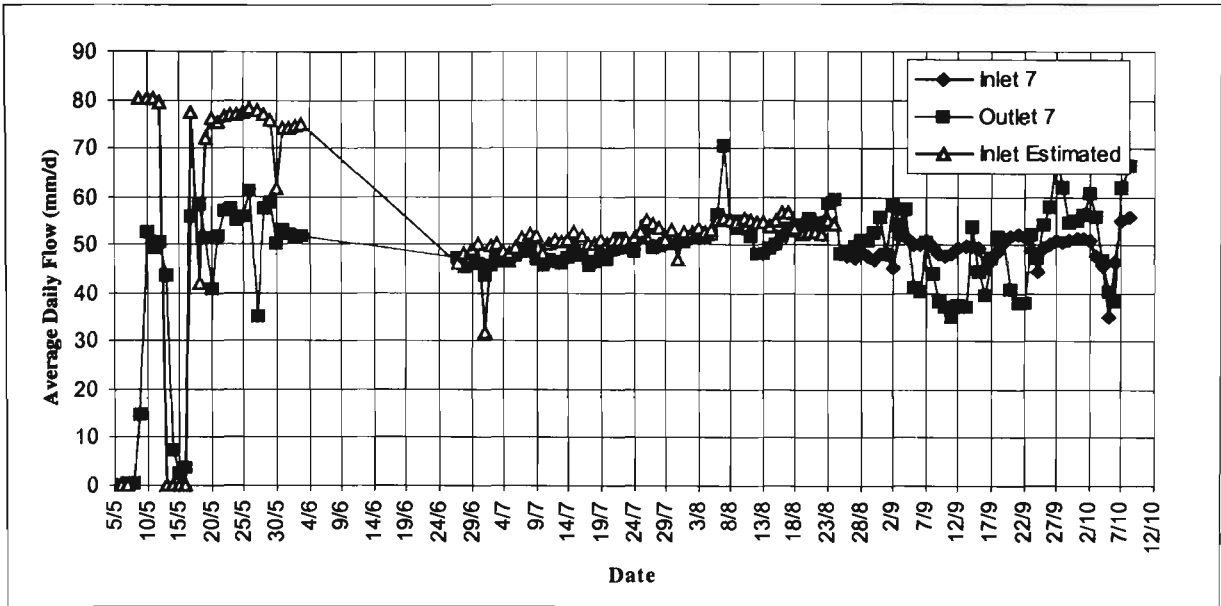


(e) Bay 5



(f) Bay 6

Figure 4.8 Time Series Plots of Inflows and Outflows (Contd...)



(g) Bay 7

Figure 4.8 Time Series Plots of Inflows and Outflows (Contd...)

Rainfall and evaporation were measured using a 0.2 mm pluviometer and a Class A evaporation pan respectively. The pluviometer was calibrated before installation using the procedures given by manufacturers. Both the pluviometer and the Class A evaporation pan were installed near the common inlet of the bays (Figure 4.9). Evaporation and rainfall information were not obtained throughout the entire trial period. Evaporation was measured between July 21 and October 9 1997 (Table 4.3), while rainfall measurements had commenced earlier. The time gap between rainfall and evaporation measurements was due to delays in the supply of the evaporation pan by the manufacturers.

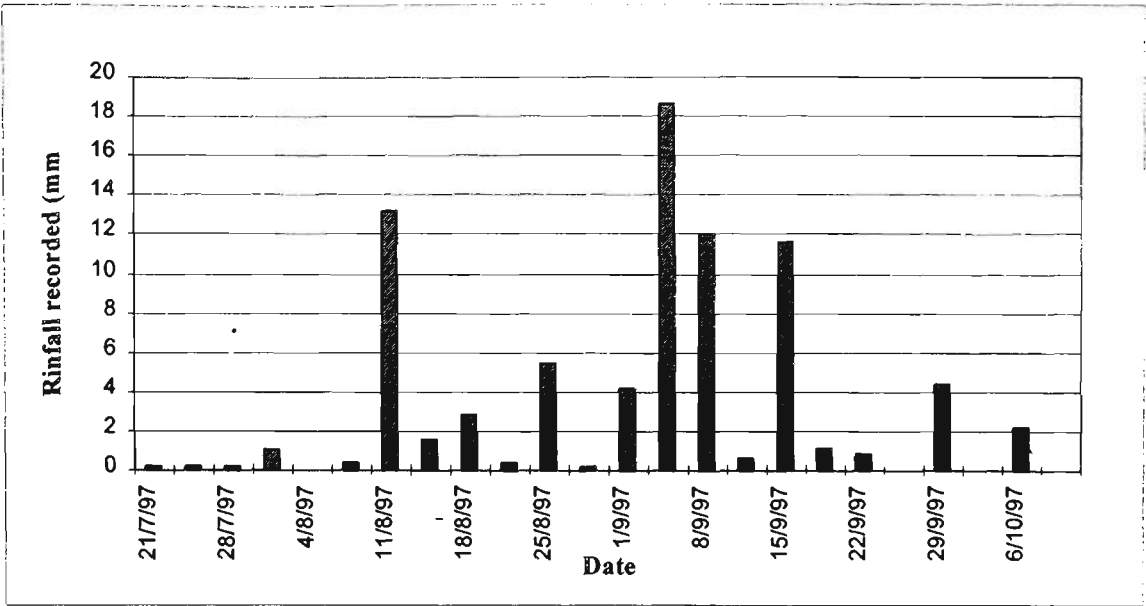


Figure 4.9 Class A Evaporation Pan and Pluviometer (Edge Right)

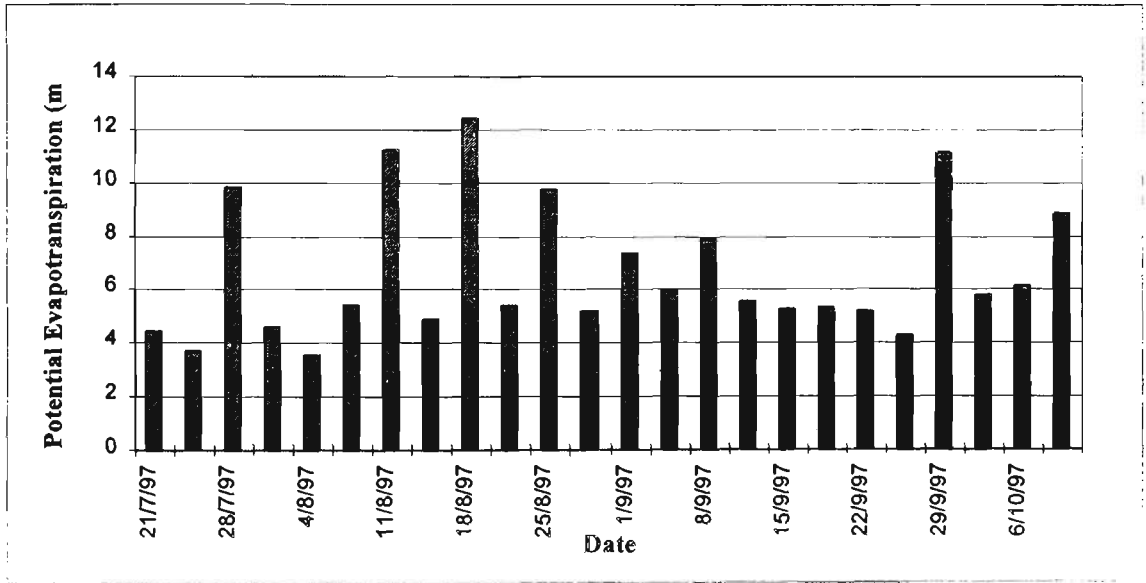
Table 4.3 Rainfall and Evaporation Data

Date	Rainfall (mm)	Net Evap (mm)	Pan Evap (E_{pan}) (mm)	Potential Evapotranspiration (PET) (mm)	Net Loss (mm)
21/7/97	0.2	4.37	4.57	4.48	4.28
24/7/97	0.2	3.50	3.70	3.72	3.52
28/7/97	0.2	10.49	10.69	9.80	9.60
31/7/97	1	3.72	4.72	4.60	3.60
4/8/97	0	3.50	3.50	3.54	3.54
7/8/97	0.4	5.24	5.64	5.41	5.01
11/8/97	13.2	-0.87	12.33	11.23	-1.97
14/8/97	1.6	3.50	5.10	4.93	3.33
18/8/97	2.8	10.93	13.73	12.44	9.64
21/8/97	0.4	5.24	5.64	5.41	5.01
25/8/97	5.4	5.24	10.64	9.76	4.36
28/8/97	0.2	5.24	5.44	5.24	5.04
1/9/97	4.2	3.72	7.92	7.39	3.19
4/9/97	18.6	-12.24	6.36	6.03	-12.57
8/9/97	12	-3.50	8.50	7.90	-4.11
11/9/97	0.6	5.24	5.84	5.58	4.98
15/9/97	11.6	-6.12	5.48	5.27	-6.33
18/9/97	1.2	4.37	5.57	5.35	4.15
22/9/97	0.8	4.59	5.39	5.19	4.39
25/9/97	0	4.37	4.37	4.30	4.30
29/9/97	4.4	7.87	12.27	11.17	6.77
2/10/97	0	6.12	6.12	5.82	5.82
6/10/97	2.2	4.37	6.57	6.22	4.02
9/10/97	0	9.62	9.62	8.87	8.87
Average net loss between 21/7/97 to 9/10/97 (mm/d)					1.05

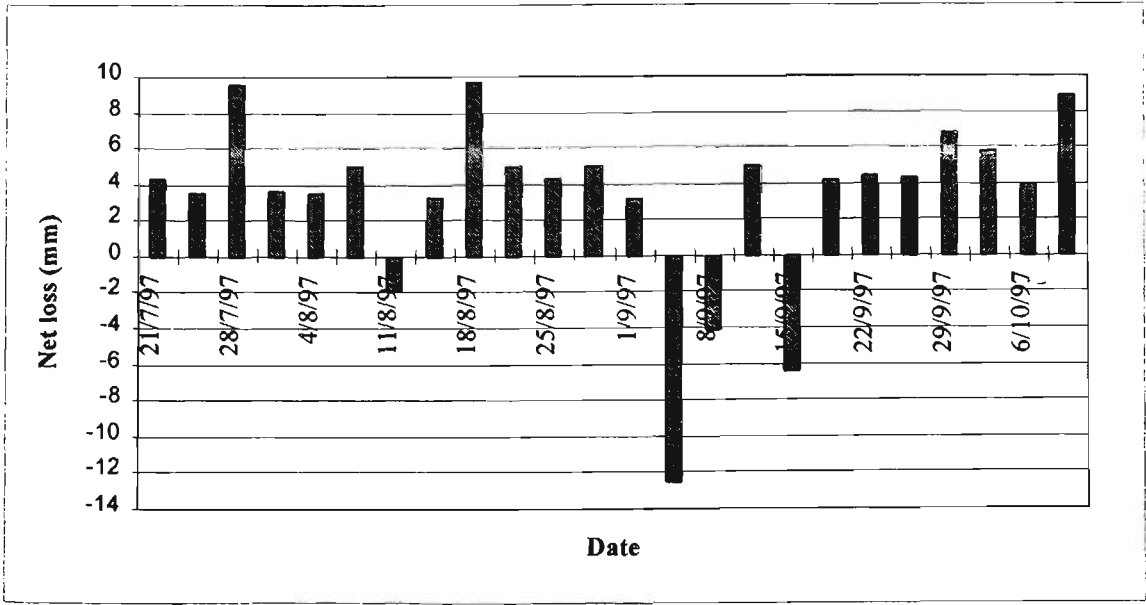
The evaporation measurements were obtained at around 9.00 A.M after the collection of water quality samples on water quality sampling days. They were obtained by monitoring the level of water in the evaporation pan between each measurement. At each measurement day, the level of water that was marked in the evaporation pan, was maintained by manually adding or taking out water from the pan using a cup of known volume. The number of cups added or taken out were recorded, and this gave the amount evaporated from the pan over that period (i.e. between the current and the previous measurement day). However, this amount includes the rainfall over this period and therefore is the net evaporation from the pan. This is shown as Net Evap in Table 4.3.



(a) Rainfall Recorded (mm)



(b) Potential Evapotranspiration (mm)



(c) Net Loss (mm)

Figure 4.10 Rainfall, Potential Evapotranspiration and Net Loss Plots

4.4 FLOW BALANCES

A flow balance is essential to give an estimate of any losses or gains which may have occurred on the trial bays and should be based on collected hydraulic information. The hydraulic information includes inflows, outflows, evaporation and rainfall. The flow balances in conjunction with water quality analyses are also essential for computing the nutrient loads into and out of the bays. Nutrient removal is also affected by water losses from the bays.

4.4.1 Water Losses From Bays

According to the time series plots shown on Figure 4.8, the inflows for all bays stayed fairly constant during the trial period, after the adjustments were made to get the required flows onto the bays (Section 4.2.3), while the outflows were more variable, reflecting the effects of rainfall and losses on the bays. It can also be observed from these time series plots that, unlike the other bays, bays 1 and 7 often have higher outflows than inflows, suggesting that they received additional flows from sources other than their inlets. On the other hand, bay 5 shows large wastewater losses between 1 and 25 September, while bay 4 indicates much lower losses even some modest flow gain during the same period. Flow losses may be due to the effects of percolation or cross flows to adjacent bays. The above effects have been considered in previous studies. For example, Bartlett (1972) suggested that care must be taken in the design of grass filtration bays to avoid channelling between the bays.

A detailed investigation of flow gains and losses on the bays was undertaken by dividing the trial period into five sub-periods. Table 4.4 summarises the inflows, outflows and wastewater losses during each of the five sub-periods. In this table, the losses for each sub-period were computed as the difference in the average inflows and average outflows of that sub-period. The losses could not be computed for bay 1 for the first two sub-periods (i.e. 6 May - 24 July), since the outflows were not available. The average inflows and outflows of each bay over the whole study period were computed using Equation (4.3):

Table 4.4 Summary of Average Flows and Losses

Periods	BAY 1 (Area, 15770 m ²)			BAY 2 (Area, 14174 m ²)			BAY 3 (Area, 14516 m ²)			BAY 4 (Area, 14212 m ²)		
	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)
1 6 May - 3 June	-	-	-	29	16	13	26	20	4	35	23	12
2 27 Jun - 24 Jul	-	-	-	30	17	13	29	24	6	42	29	13
3 25 Jul - 25 Aug	21	17	4	30	19	11	30	25	5	43	31	12
4 26 Aug - 25 Sep	20	19	1	30	21	9	29	27	3	40	38	2
5 26 Sep - 8 Oct	20	15	5	30	16	14	29	22	8	42	31	11
Average over whole period (mm/d)	21	17	3	30	18	12	28	24	5	40	31	10
Periods	BAY 5 (Area, 12920 m ²)			BAY 6 (Area, 12160 m ²)			BAY 7 (Area, 12274 m ²)			Average across all bays		
	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	Avg Inflow (mm/d)	Avg Outflow (mm/d)	Losses (mm/d)	In (mm/d)	Out (mm/d)	Losses (mm/d)
1 6 May - 3 June	39	29	10	42	46	-4	42	41	1	35	29	6
2 27 Jun - 24 Jul	41	31	10	48	40	8	50	47	3	39	31	9
3 25 Jul - 25 Aug	42	30	12	53	41	12	54	53	1	38	30	8
4 26 Aug - 25 Sep	40	22	18	47	41	6	49	47	2	36	30	6
5 26 Sep - 8 Oct	41	29	12	48	37	11	49	56	-7	36	28	8
Average over whole period (mm/d)	40	28	12	48	42	6	49	48	1	37	30	7

Note - All figures shown in the table above, are rounded off to the nearest 1.

$$\text{Average over whole period} = \sum[(X \times Y)] / Z \quad (4.3)$$

where X = average flow (i.e. inflow or outflow) of each sub-period, mm/d
 Y = duration of each sub-period, days
 Z = total duration of all sub-periods, days

The average total inflows and outflows across all bays during each sub-period (as shown in grey on Table 4.4) are computed using the areal average as shown in Equation (4.4):

$$\text{Average across all bays} = \sum[(X \times A)] / T \quad (4.4)$$

where A = area of each bay, m²
 T = total area of all bays, m²

Table 4.4 suggests that, in general, there are significant differences between the wastewater losses on different bays. Also, for a given bay, there could be large variations of flow losses during different sub-periods of the trial. For example, the losses on bays 4 and 5 from 26 August to 25 September are 2 and 18 mm/day respectively, compared with the values of about 12 and 11 mm/day respectively, during all other periods of the trial. Flow balance analysis considering all seven bays suggests that, on average, 7 mm/day of wastewater is lost from the bays over the whole period of the trials (Table 4.4).

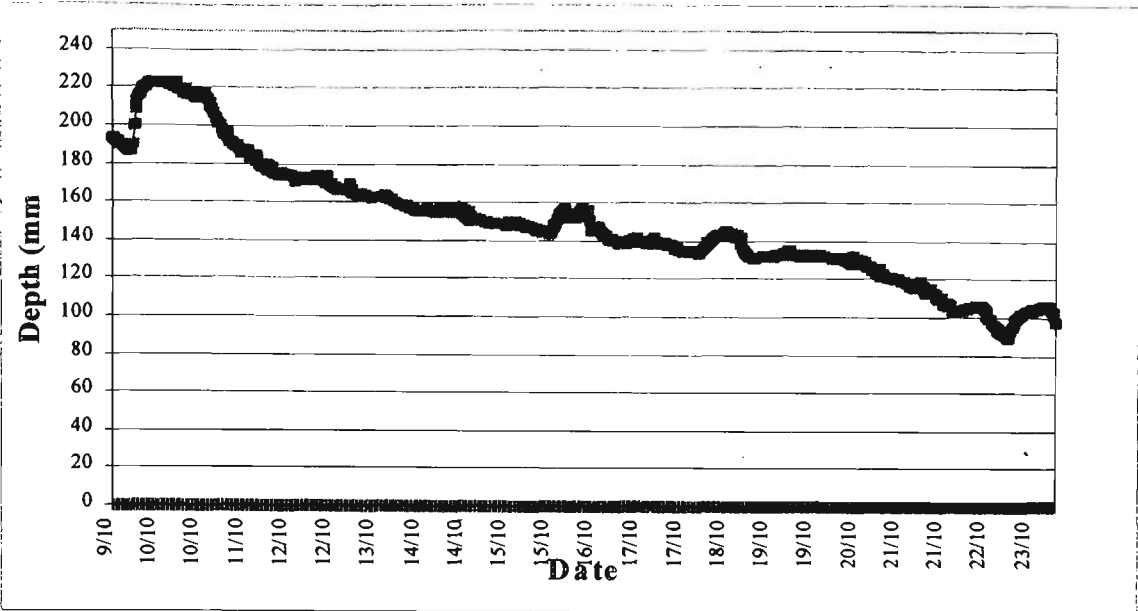
The losses are due to a combination of effects such as seepage into the soil profile and evapotranspiration. Water loss due to the net effect of evapotranspiration (ET) and rainfall is 1 mm/day (Section 4.3). This figure was based on measurements of rainfall and evaporation between 21/7/97 and 9/10/97. However, it is reasonable to assume the net effect of ET and rainfall as 1 mm/day even for the earlier part of this study. The remaining water losses of 6 mm/day occur through infiltration. This seems high compared with expected infiltration based on a permeability of less than 1 mm/day (Maher and Martin, 1952) associated with the soils at the site. However, these soils

are noted to have rusty root channels, which can increase the infiltration rate up to several orders of magnitude.

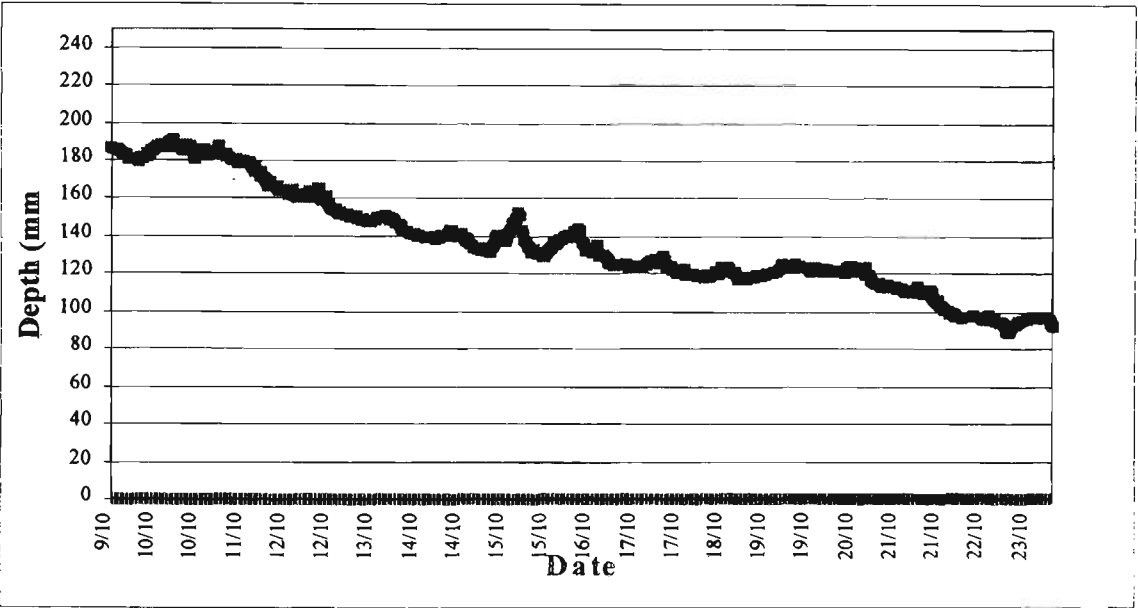
Table 4.4 also shows that bays 1 and 7 may receive additional flows from groundwater or adjacent bays. This may be concluded from minimal losses (i.e. average losses of 3 and 1 mm/d for bays 1 and 7 respectively) of these bays in comparison to the losses from the other bays. Furthermore, the bays adjacent to bays 1 and 7 (i.e. bays 2 and 6 respectively) have suffered from more than double the losses of bays 1 and 7, which indicates that bays 1 and 7 may have received some flows from bays 2 and 6.

After the inflows to the bays were stopped at the end of the trials (i.e. 9 October 1997), water depth measurements at the outlet of the bays were continued for 14 days to investigate if there were cross flows between adjacent bays. The time series plots of water depth during this period are shown in Figure 4.11. The similarity of water depth trends at the outlet of these bays 1 and 2 fell below the 100 mm level (i.e. the minimum depth for flow at outlet) about 12 days after the trials were stopped. The similarity of water depth trends in the outlet of these two bays suggests an inter-connection and complements the observation of water gain by bay 1 to indicate that there was some cross flow from bay 2 to bay 1.

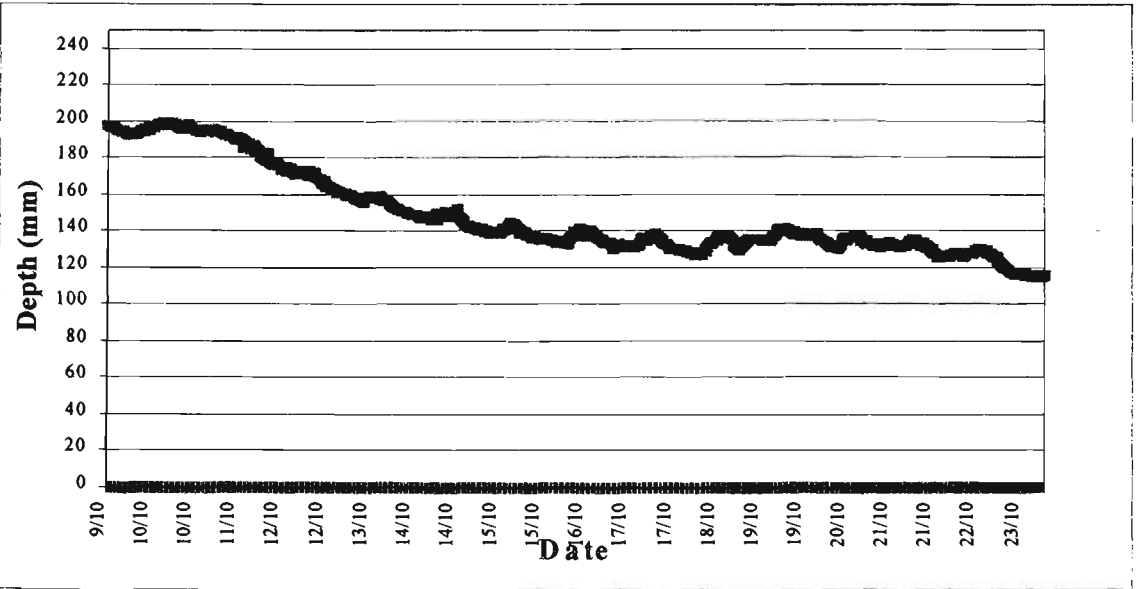
The water depth at the outlet of bay 5 also fell below the 100 mm level after 12 days while a gradual rise in water depth was observed for bay 4 during the same period. This suggests that bay 5 could have lost some flow to bay 4 somewhere upstream part of the bay. However, the extent to which the water depth dropped at the outlet of bay 5 suggests that it lost a significant part of its flow through infiltration, probably through fractures or root channels which characterise the site. The water depth plots do not clearly indicate cross flows between bays 6 and 7. However, the water depth of bay 7 was about 10mm higher than that of bay 6 during this period.



(a) Bay 1

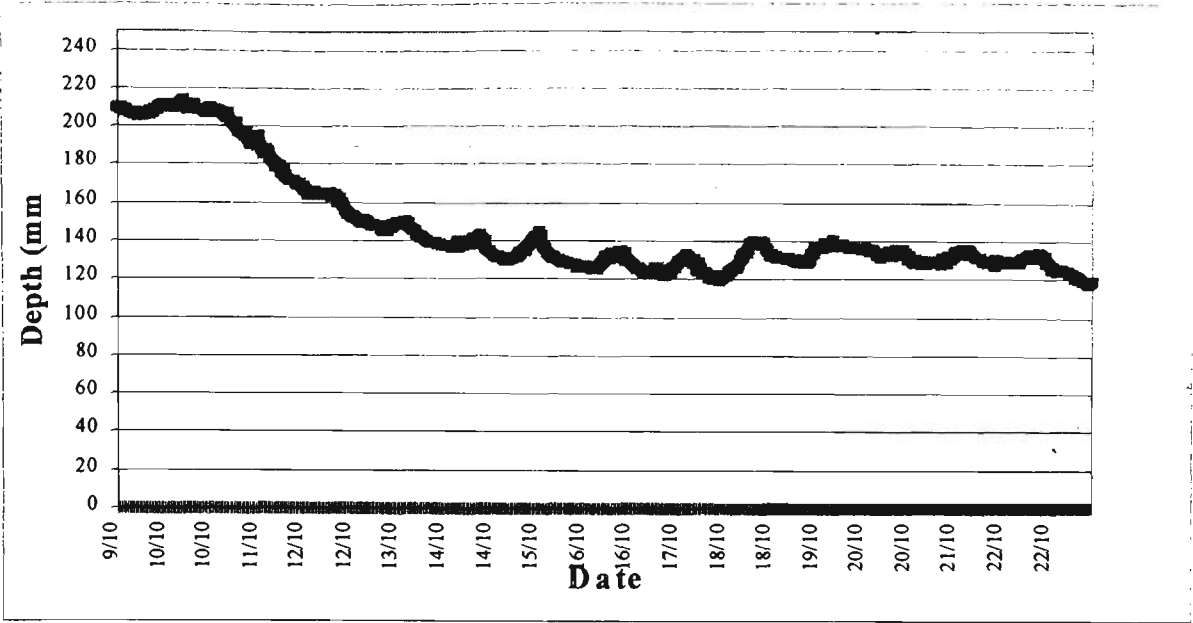


(b) Bay 2

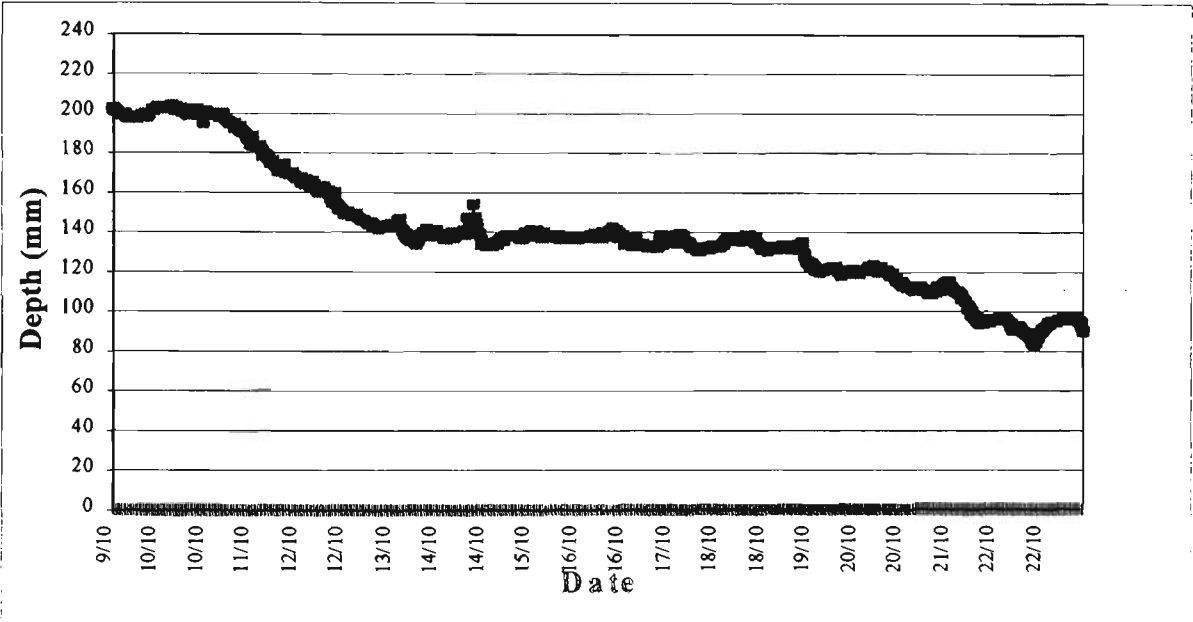


(c) Bay 3

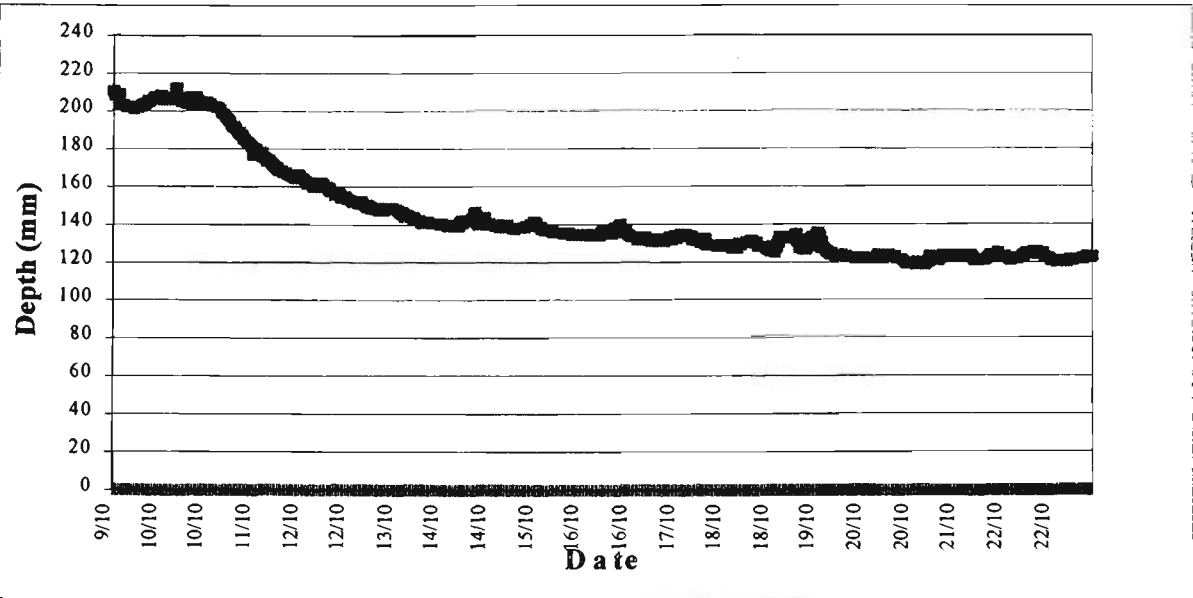
Figure 4.11 Time Series Plot of Water Depths at the End of Trials



(d) Bay 4



(e) Bay 5



(f) Bay 6

Figure 4.11 Time Series Plot of Water Depths at the End of Trials (Contd...)

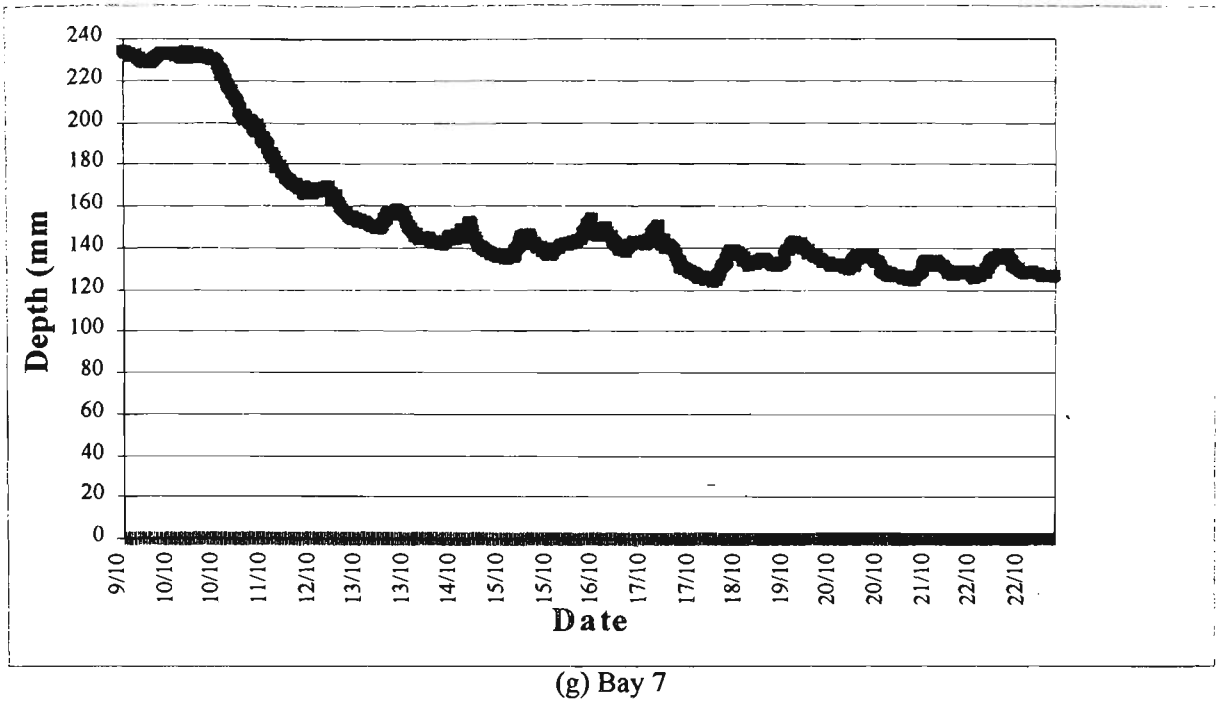


Figure 4.11 Time Series Plot of Water Depths at the End of Trials (Contd...)

The outflows for bays 4, 5, 6 and 7 are reproduced in Figure 4.12 to further discuss the issue of water losses in these bays. This plot shows that between 22 September and 7 October, increased outlet flows for bay 7 occurred when outlet flows for bay 6 decreased, suggesting some amount of cross flow from bay 6 to bay 7. This figure also shows a decrease in the outlet flows of bay 5 between 2 September and 22 September and a moderate increase in the outflows of bay 4 during this period, showing some cross flows from bay 5 to bay 4. Bays 4 and 5 should have the same outlet flows if there are no cross flows between the bays and the losses are the same, since inflows are the same. The same argument is true for bays 6 and 7.

Cross flows occurred between some bays mainly through breaches in the check banks and probably through burrows. Field inspection of the site after application of wastewater had been stopped clearly showed that portions of some of the check banks were very low and sometimes almost flat with a high potential for cross flows at these locations. For instance some cross flow from bay 8 to bay 7 was observed near the outlet of the bays on August 28, indicating that some of the flow gain by bay 7 is from bay 8. It was noted that a leaking checkbank resulted in this cross flow. The leaking check bank was later repaired by Melbourne Water.

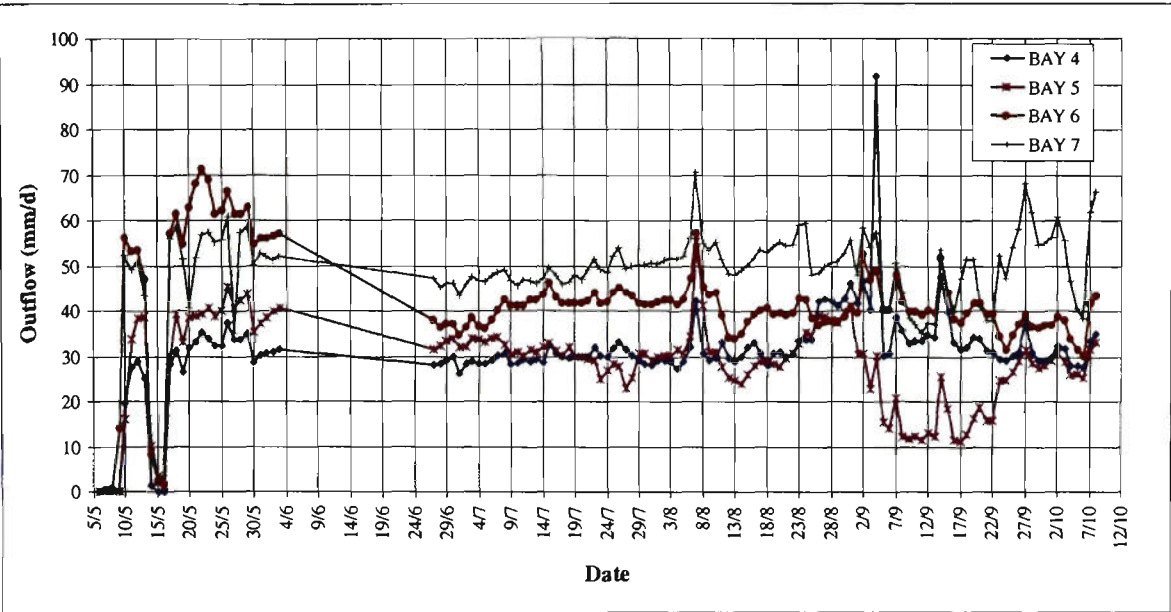


Figure 4.12 Outflows for Bays 4, 5, 6 and 7 throughout the Trial

Based on the above analysis, there is some evidence to suggest that bay 2 lost some flow to bay 1, bay 5 lost a small amount of flow to bay 4, and bays 6 and 8 lost some flow to bay 7. In most cases, the cross flows may not have occurred continuously over the whole period of the grass filtration trials.

4.5 DETENTION TIME MEASUREMENTS AND ANALYSIS

Detention time provides a measure of the average time wastewater remains on the bay. It also provides valuable information on the type of flow which occurs on the bays along with an approximate relationship between the influent and effluent from the bays. The detention time is also one of the most important design parameters which can assist in providing information on the flow characteristics and the chemical, biological, and microbiological processes on a grass filtration bay. It has a significant role in modelling of nutrient removals as it forms one of the variables in the first-order reaction kinetics model (Chapter 6). The detention time depends on a number of factors including the hydraulic loading rate, physical characteristics of the

bay such as vegetation and volume available for water storage on the bay, flow type and longitudinal slope of the bay.

Vorrath (1989), who conducted hydraulic studies of the grass filtration bays of the WTP, stated that different elements of fluid take different flow paths and times to travel the length of the bay. Therefore, to obtain a meaningful result, the average time the fluid spends in the bay (i.e. detention time) must be found. In order to achieve this, the Residence Time Distribution (RTD) was used in Vorrath (1989). The centroid of the distribution gives the detention time. The RTD is essentially a plot of concentration against time, which is determined by a stimulus response experiment. This stimulus response experiment involves dosing with a tracer chemical. The response is then recorded in the form of time records of the tracer concentrations at the bay outlet during the experiment. After conducting the tracer studies, the detention times of the bays were computed using three independent methods, namely best fit, centroid of distribution, and 50% mass out or rule of thumb methods (Vorrath, 1989). These methods are described in detail in Section 4.5.2.2.

4.5.1 Tracer Measurements

Two tracer studies using lithium (Li) were conducted to determine the detention time of the grass filtration bays at the WTP. The first study was conducted on bays 1, 3, 4 and 6 between 7 and 20 July. These bays were selected to ensure that one of each of the hydraulic loading rates of 20, 30, 40 and 50 mm/day were represented. The second tracer study was undertaken from 24 September to 6 October on bays 4, 5, 6 and 7 to investigate some abnormal water losses and gains recorded earlier during the trials and to check the accuracy of the first tracer study, especially for bay 4 which produced a shorter detention time than expected. It should also be noted that tracer measurements can help in explaining the hydraulic losses and gains which may occur within the bays. This can be achieved by computing the Li losses using a mass balance approach, which is explained in detail in Section 4.5.2.3.

The chemical used for the study was lithium hydroxide (LiOH). The chemical was delivered in 25 kg bags. According to laboratory analysis, the bags contained 53%

pure LiOH. Accounting for the purity of the chemical, the masses of LiOH shown in Table 4.5 were used to dose the bays. These masses were used to produce higher concentrations of Li for higher loaded bays.

Table 4.5 Lithium Hydroxide Dosage Used for the Bays

Study No.	Bay No.	Water Volume (litres)	Mass of LiOH (kg)	Average Li Concentration (mg/l)
1	1	300	5.6	2600
	3	300	7.5	3400
	4	400	10.0	4466
	6	400	13.7	6400
2	4	400	10.0	3667
	5	400	10.0	3700
	6	400	13.8	5300
	7	400	13.8	5333

The dosing was carried out for each of the above bays using a 400 litre galvanised steel tank, which was positioned over the top of the concrete inlet structures and filled with the water volumes shown in Table 4.5. The chemical was dissolved in water by stirring gently for about 10 minutes. The solution was then released into the bay inlet over a period of 30 minutes at a steady rate with continuous stirring as recommended by the Water Ecoscience laboratories staff. While the solution was being released into the bay from the tank, samples were taken from the outlet of the tank at the beginning, halfway (i.e. after 15 minutes) and end of dosing in order to check uniformity of concentration. Laboratory analysis of these samples gave concentrations of Li, the averages of which are shown in Table 4.5. As can be seen from Table 4.5, the average Li concentrations of the two studies even with the same volume of water and mass of LiOH were different. This can be clearly seen in bay 6 dosage. The difference is due to different percentage of impurities of LiOH in the two studies. The study 2 had a higher percentage of impurities.

Safety precautions were taken throughout the experiment by use of protective gloves, facemasks, overalls, gumboots and making available fresh water for quick wash of eyes or skin in case of Li contact with the body.

Li concentrations were also monitored at the bay outlets by continuous sample collection using automatic samplers. For the first detention time study, samples were collected at 3-hour intervals for the first 11 days and at 6-hour intervals for the last 5 days. However, based on the experience of the first study, samples were collected at 3-hour intervals for the first 6 days and at 6-hour intervals for the last 5 days during the second detention time study. The frequency of sample collection was changed from every 3 hours to every 6 hours to minimise the number of samples, due to the costs incurred for laboratory analysis of each sample, without compromising the accuracy of results of the detention time studies.

To trace the path of Li through the grass filtration bays, areal samples were also taken at five equally spaced transverse points at the 200 and 300m mark (i.e. Figure 3.5) over a four-day period for the first tracer study. This period was chosen as it was assumed that four days after the dosing, most of the tracer chemical would have travelled past the 300m mark. The sampling grid (i.e. at five equally spaced transverse points at the 200 and 300m mark) was chosen to cover the whole width of each bay to investigate the existence and effects of channelling to determine the type of flow. The areal sampling schedule is shown in Table 4.6.

Table 4.6 Schedule for Areal Sampling for the First Tracer Study

Days after Dosing	Date	Sample Location
1	8/7/97	200m
2	9/7/97	200m & 300m
3	10/7/97	300m
4	11/7/97*	300m

*only for bays 1 and 3 which have low hydraulic loading rates

4.5.2 Analysis, Results and Discussions

4.5.2.1 Flow Behaviour

One of the aims of the tracer studies was to determine the type of flows on the bays. This was achieved through the analysis of the RTD curves and areal plots of measured

Li concentrations on the bays. As stated in Section 4.5.1, areal sampling was done only for the first tracer study.

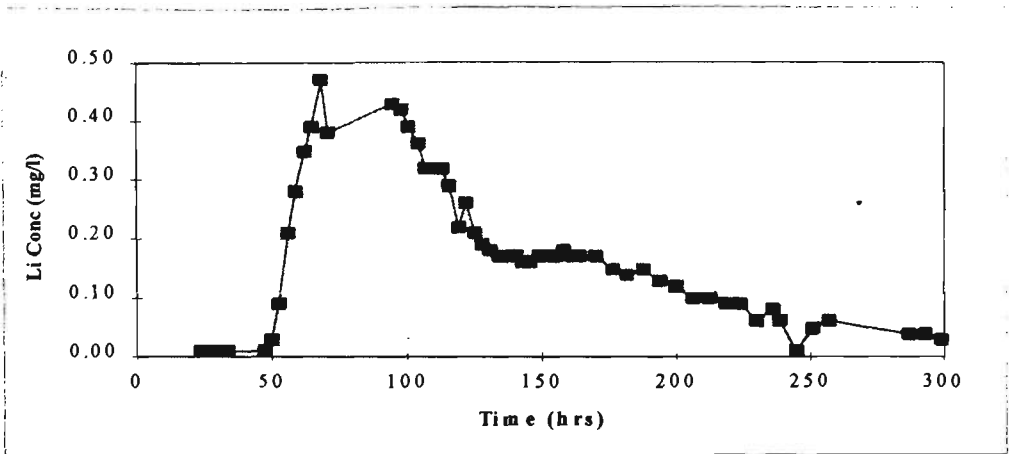
(a) First Tracer Study

The RTD curves are shown in Figure 4.13 for the first tracer study. According to Figure 4.13, no traces of Li were recorded for the first 24, 18, 12 and 12 hours for bays 1, 3, 4 and 6 respectively. This can be related to the hydraulic loading rates of the bays showing flow in lower loaded bays takes more time to reach the outlets than the higher loaded bays, as expected. The peak Li concentrations occurred between 30 and 110 hours for all four bays.

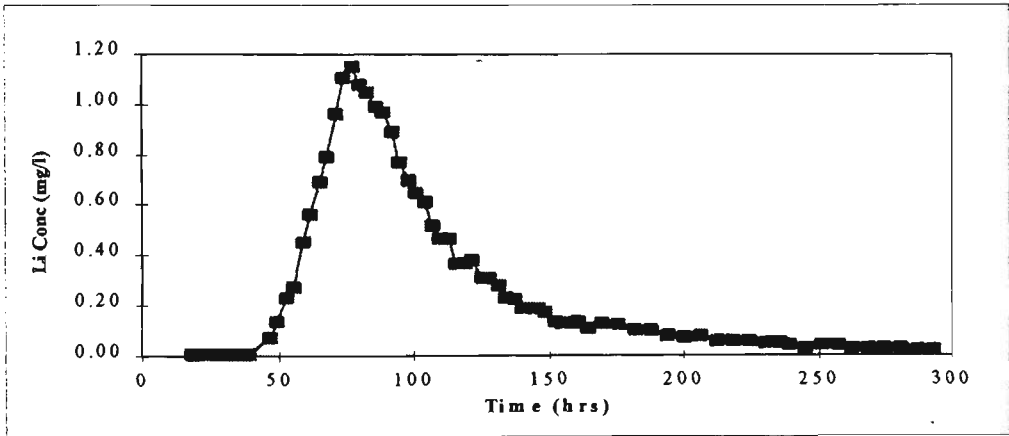
Traces of Li were detected until 132 to 239 hours after dosing, after which the concentration values had fallen below 0.04 mg/l.

The first tracer study showed some characteristics of both plug and laterally mixed (i.e. plug flow with some degree of lateral dispersion) flows for all four bays (i.e. bays 1, 3, 4 and 6). This can be observed from the shapes of the RTD curves (Figure 4.13). The results suggest that the flow through bays 1 and 3 have features of both mixed and plug flows with bays 4 and 6 showing more pronounced plug flow characteristics than bays 1 and 3. This is primarily due to the fact that the RTD curves for bays 1 and 3 show a gradual decrease in distribution of Li concentrations after the peak, in comparison to bays 4 and 6 which show a rapid decrease in concentration levels immediately after the peak. This can also be observed from the plots of areal samples shown in Figure 4.14. The following characteristics can be observed from Figure 4.14, although it is hard to draw definite conclusions.

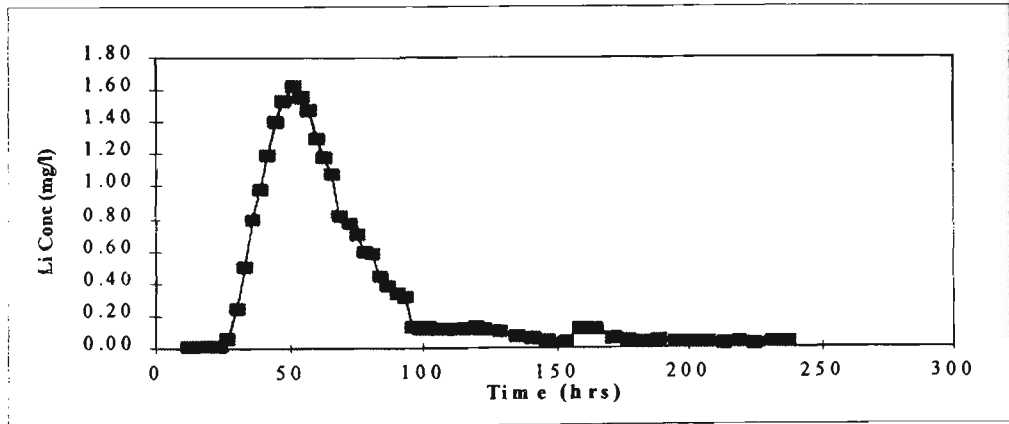
- fairly uniform flow across the 200 m mark of bay 1 with some dispersion (i.e. Figure 4.14a).
 - a peak in Li concentration at the centre of the bay at the 300 m mark on 9/7/97 showing some channelling of the flow between 200 m and 300 m mark. On later days, the spread is almost even across the width of the bay at the 300 m mark.
-



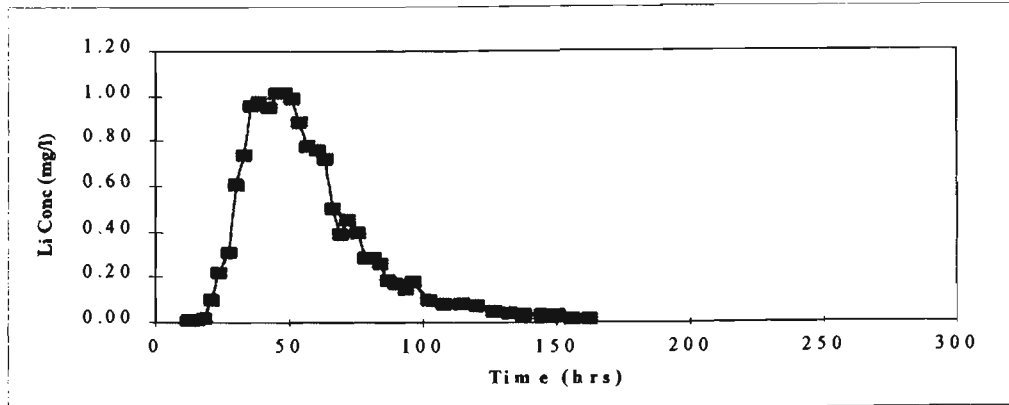
(a) Bay 1



(b) Bay 3

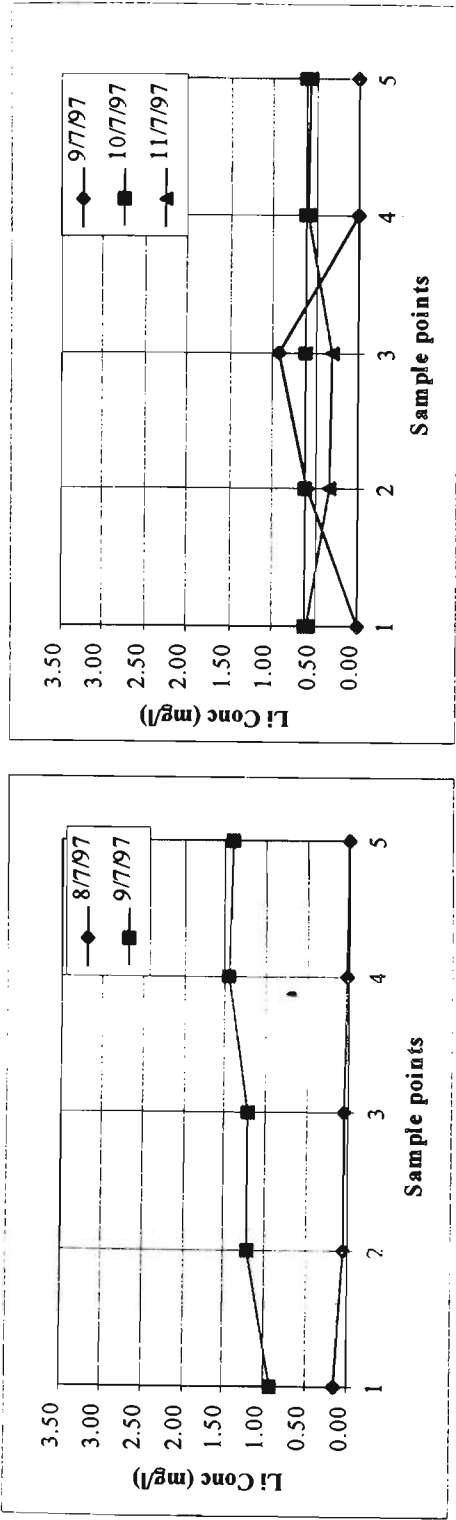


(c) Bay 4

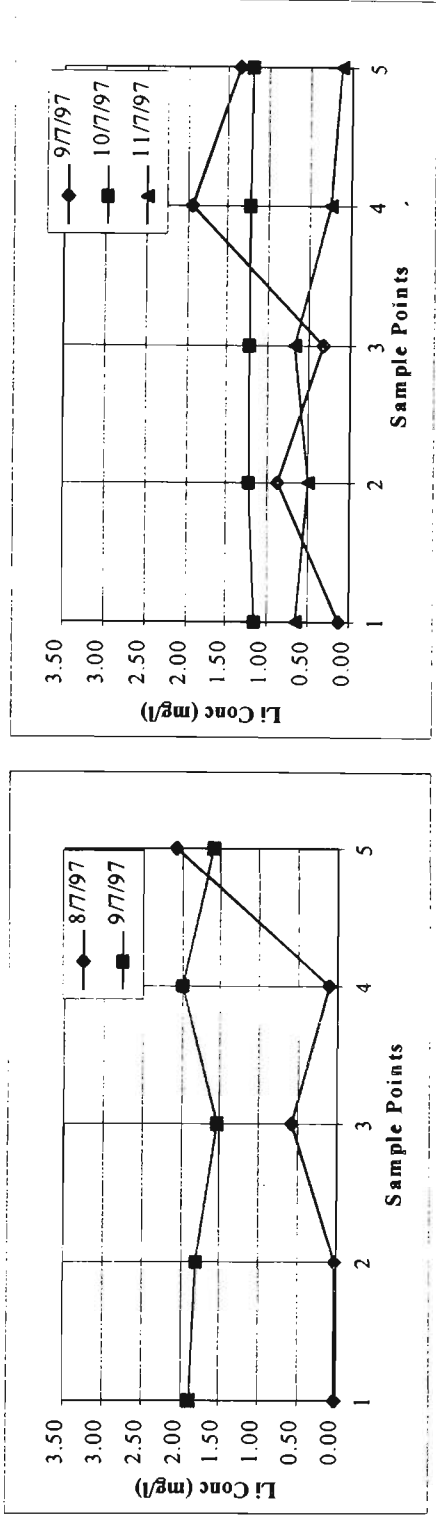


(d) Bay 6

Figure 4.13 RTD Curves for First Tracer Study

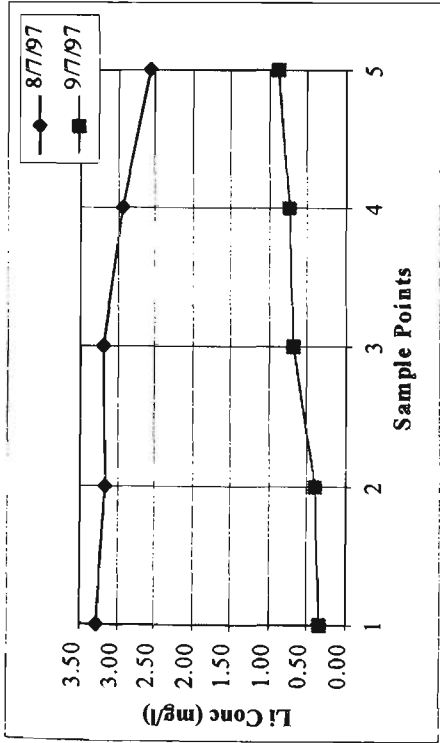


(a) Bay 1



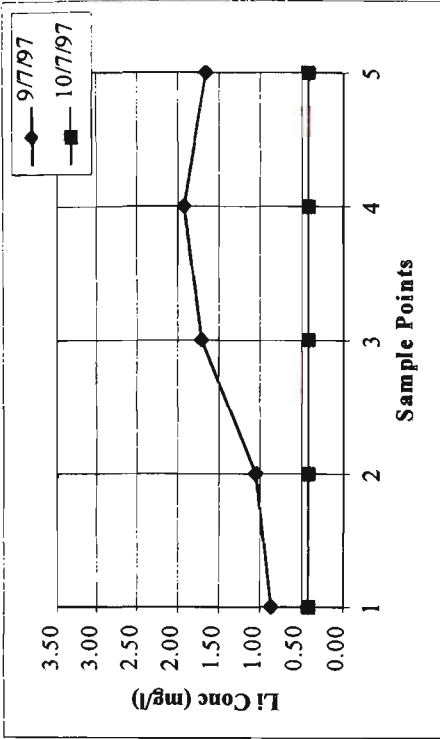
(b) Bay 3

Figure 4.14 Areal Plots of Lithium Concentrations at 200m and 300m points for Bays 1 and 3

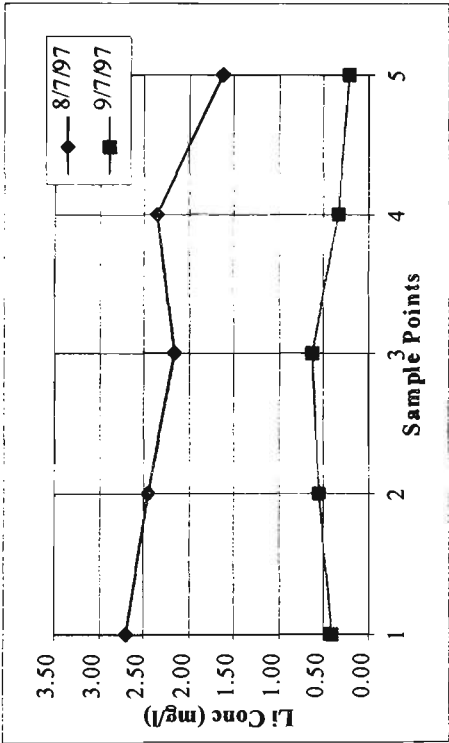


(200m)

(c) Bay 4

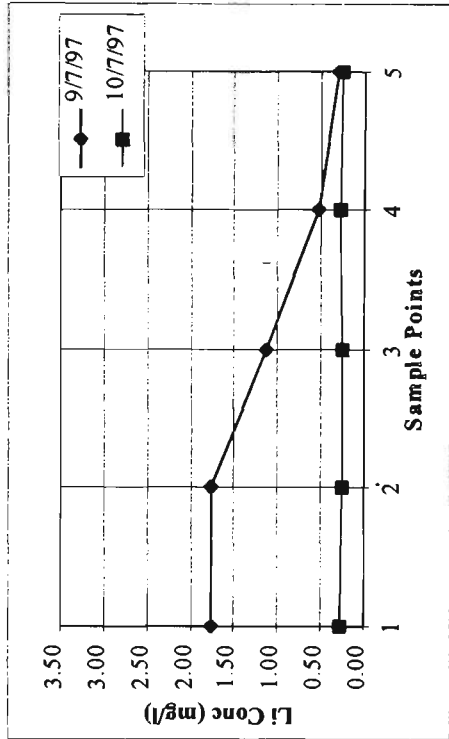


(300m)



(200m)

(d) Bay 6



(300m)

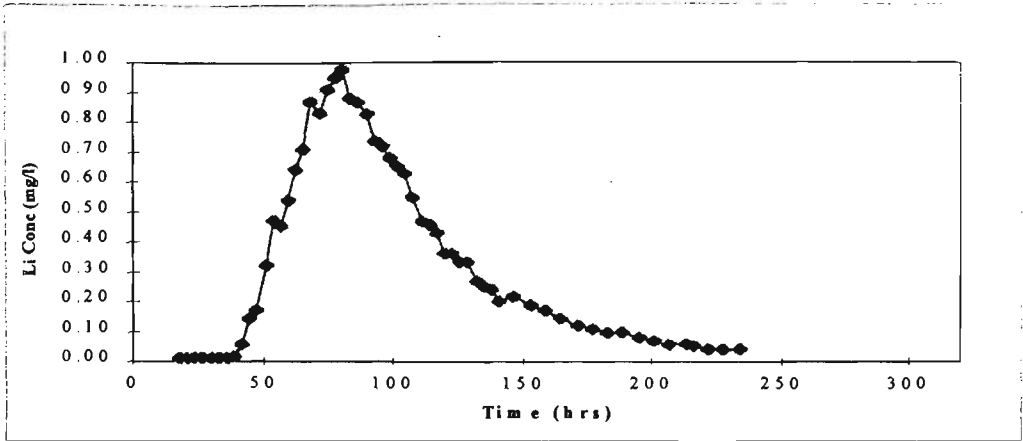
Figure 4.14 Areal Plots of Lithium Concentrations at 200m and 300m Points for Bays 4 and 6 (Contd...)

- high Li concentrations at the right edge of bay 3 (i.e. Figure 4.14b), suggesting channelling of flow between inlet and 200 m mark on 8/7/97.
- on 9/7/97, laterally mixed flow at 200 m point of bay 3 can be seen.
- on 9/7/97, higher concentrations of Li at the right edge of the bay 3 showing plug flow between 200 m and 300m mark.
- on 10/7/97 and 11/7/97, more uniform distribution of Li across the 300 m mark of bay 3 suggesting laterally mixed flow.
- generally, bays 4 and 6 (i.e. higher loaded bays) showed higher Li concentrations at initial stages (i.e. at 8/7/97) across the 200 m mark, which gradually decrease with time. This is in contrast to bays 1 and 3 (i.e. lower loaded bays) where the Li concentrations were lower at the initial stages (i.e. at 8/7/97) and higher on (9/7/97) across the 200 m mark. This contrast is mainly due to the hydraulic loading rate of the bays. The flow (on lower loaded bays) was slower, resulting in Li concentrations reaching their peak level later than the higher loaded bays. This can also be observed from the shapes of the RTD curves as shown in Figure 4.13.
- Once the peak concentration is passed, the Li concentration at all bay outlets has a uniform distribution across the bays.

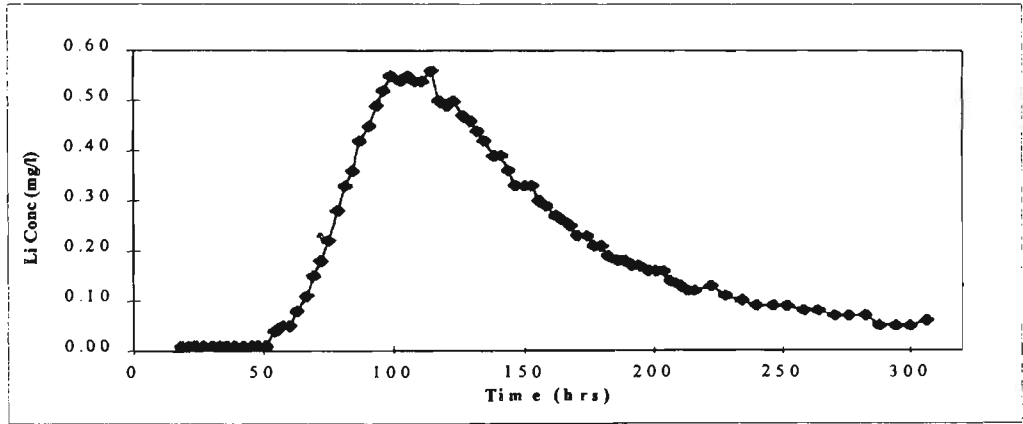
According to the areal plots (i.e. Figure 4.14), the variations in Li concentration across the width of the bays over different days show that the wastewater flow was different at different parts of the bay (i.e. between inlet and 200m and from 200m to 300m mark). This in turn suggests that some degree of channelling occurred on the trial bays. The degree of channelling depends on the hydraulic loading rate, the uniformity of topography and the density of vegetations on the bays.

(b) Second Tracer Study

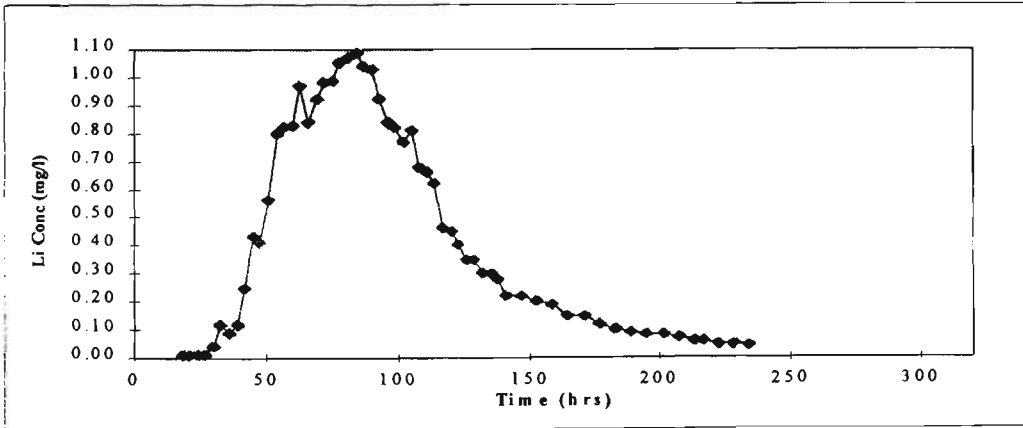
The second tracer study was performed on bays 4, 5, 6 and 7. Figure 4.15 shows the RTD curves for the second tracer study. As can be seen from Figure 4.15, no traces of Li were recorded for the first 18 hours. Between the 45th and the 114th hours the Li concentrations at the outlets peaked for all four bays. Traces of Li were detected until



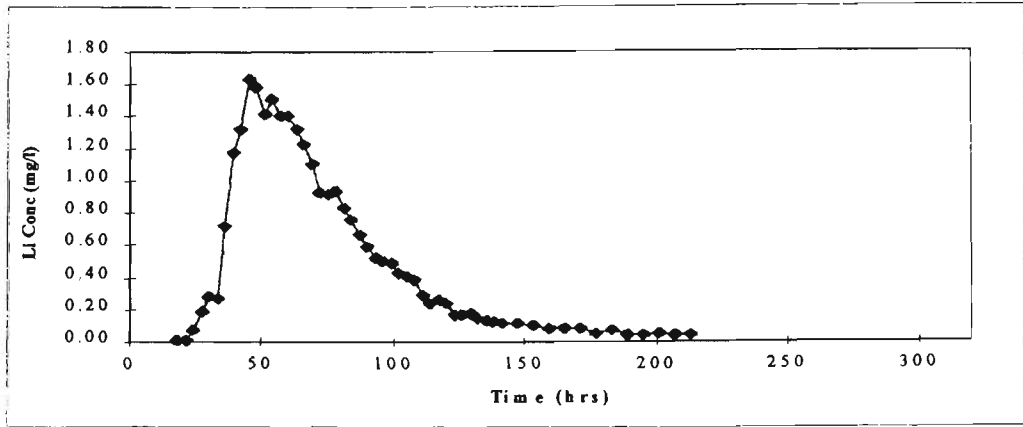
(a) Bay 4



(b) Bay 5



(c) Bay 6



(d) Bay 7

Figure 4.15 RTD Curves for Second Tracer Study

214 to 306 hours after dosing, after which the concentration values were below 0.04 mg/l.

The shapes of the RTD curves in this figure, suggest mixed flow type on all four bays. This is primarily due to the gradual and almost uniform decrease of Li concentrations after the peak of the curves. Although bays 4 and 5 had the same hydraulic loading rate (i.e. 40 mm/d), the Li concentration of bay 5 reached its peak at 114 hours compared to bay 4 which reached the peak at 82 hours after dosing (i.e. Figure 4.15(a) and (b)). This suggests a possible cross flow from bay 5 to bay 4, causing higher flows in bay 4, which in turn reduce the time to peak. Similarly, the Li concentrations of bay 6 peak after 85 hours, while Li concentrations of bay 7 peak after 45 hours, suggesting some cross flows from bay 6 to bay 7. Bays 6 and 7 have the same hydraulic loading rate of 50 mm/d.

Bays 4 and 6 were common in two studies. However, the time to peak almost doubled in the second study compared to the first study. This could be due to different flow losses during the two studies and the increased vegetation during the second study. This aspect is further discussed in Section 4.5.2.2 (d).

4.5.2.2 Estimation of Detention Times

As can be seen from Figures 4.13 and 4.15, the RTD curves show the flow in the bays are mixed flow. If it was plug flow (as opposed to mixed flow), the outlet should have a similar response to inlet in terms of the RTD curves, and the detention time can be estimated by the time difference between inlet and outlet pulses. However, since the bays produce mixed flow behaviour, this has to be accounted for in computing the detention time of the bays.

As stated earlier, three methods namely best fit, centroid of distribution, and 50% mass out (or rule of thumb), were used by Vorrath (1989) to compute the detention times of the grass filtration bays at the WTP. However, these bays were not the same bays that were used in this study. The best fit and centroid of distribution methods were based on the RTD curve, while the 50% mass out method was based on the

cumulative mass curve. It must be noted that the best fit and centroid of distribution methods are dependent on each other and are effectively based on the same mathematical concept. These two methods had produced similar results in Vorrath (1989), as would be expected. The centroid of distribution method makes more use of the measured data than the best fit method. Therefore, only the centroid of distribution and 50% mass out methods were used in this study to compute the detention times.

(a) Centroid of Distribution Method

Figure 4.16 shows the typical measured RTD curve at a bay. The mean value or the centroid of distribution of the RTD curve ($\bar{t}$) is given by Equation (4.5) (Levenspiel, 1972). This figure also shows an exponential curve. The reason for having this curve is explained later in this section.

$$\bar{t} = \frac{\int_0^{\infty} C t \, dt}{\int_0^{\infty} C \, dt} \tag{4.5}$$

where C is the Li concentration at the outlet t hours after start of dosing.

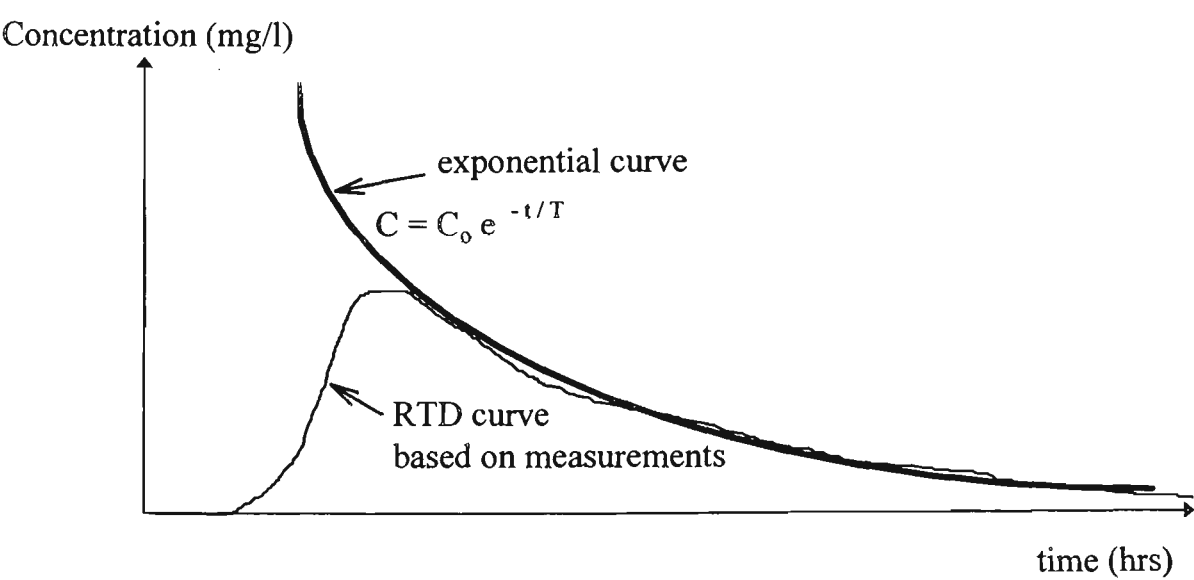


Figure 4.16 Residence Time Distribution (RTD) Curve

For the case where a large number of discrete values are known at different times, Equation (4.5) can be expressed in the form of Equation (4.6).

$$\bar{t} = \frac{\sum_{0}^{\infty} C t \Delta t}{\sum_{0}^{\infty} C \Delta t} \quad (4.6)$$

where Δt = time interval between Li samples collected from the outlet of bays

The RTD curves obtained from the tracer measurements (i.e. Figures 4.13 and 4.15) generally showed rapid increase of Li concentrations at the bay outlets with a gradual decrease after the peak which continued until concentrations less than 0.04 mg/l were reached. The concentrations less than 0.04 mg/l were not measured during the trials. However, it is expected that these low concentrations would occur for some time after measurements were stopped. This has to be accounted in computing the detention time. This is achieved by considering a theoretical curve for the recession limb of the RTD curve. Vorrath (1989) successfully used an exponential curve to model this recession limb. The fitted exponential curve for the recession limb is also shown in Figure 4.16. Although the concentrations less than 0.04 mg/l were not measured in the rising limb, it is not necessary to consider these concentrations because of the steep nature of the rising limb. The error due to the exclusion of these concentrations is negligible.

Considering the concentrations less than 0.04 mg/l, the recession limb of the RTD curve, Equation (4.6) can be expressed as Equation (4.7).

$$\bar{t} = \frac{\sum_{0}^{t_0} C t \Delta t + \int_{t_0}^{\infty} C t dt}{\sum_{0}^{t_0} C \Delta t + \int_{t_0}^{\infty} C dt} \quad (4.7)$$

where t_0 = time at which the last concentration reading just above 0.04 mg/l was taken at the outlet, hrs

The exponential curve has the following form (Vorrath, 1989).

$$C = C_0 e^{-t/T} \quad (4.8)$$

where C = tracer concentration at time t , mg/l
 C_0 = a theoretical initial concentration, mg/l
 t = time from the start of dosing, hrs
 T = time constant

Taking logarithms of Equation (4.8) produces:

$$\ln C = \ln C_0 - t/T \quad (4.9)$$

By plotting C against t , it is possible to estimate C_0 and T through the gradient and y-intercept of the regression line.

Substituting Equation (4.8) into Equation (4.7) and integrating the integrals results in:

$$\bar{t} = \frac{\sum_0^{t_0} C t \Delta t + C_0 T^2 e^{-t_0/T} (1 + t_0/T)}{\sum_0^{t_0} C \Delta t + T e^{-t_0/T} + T e^{-t_0/T}} \quad (4.10)$$

Equation (4.10) was then used to compute the detention time of the bays. Although the inclusion of the concentrations less than 0.04 mg/l on the recession limb of the RTD curve accounts only for 2% of the total detention time (Vorrath, 1989), it has been included in this study. It should be noted however, that this level of accuracy may not be justified in view of a number of other potential errors in this type of analysis.

The basic difference between the best fit method and the centroid of distribution method is that the former uses the fitted exponential curve for the whole recession

limb, while the latter uses the fitted curve only for the concentrations less than 0.04 mg/l.

(b) 50% Mass Out Method

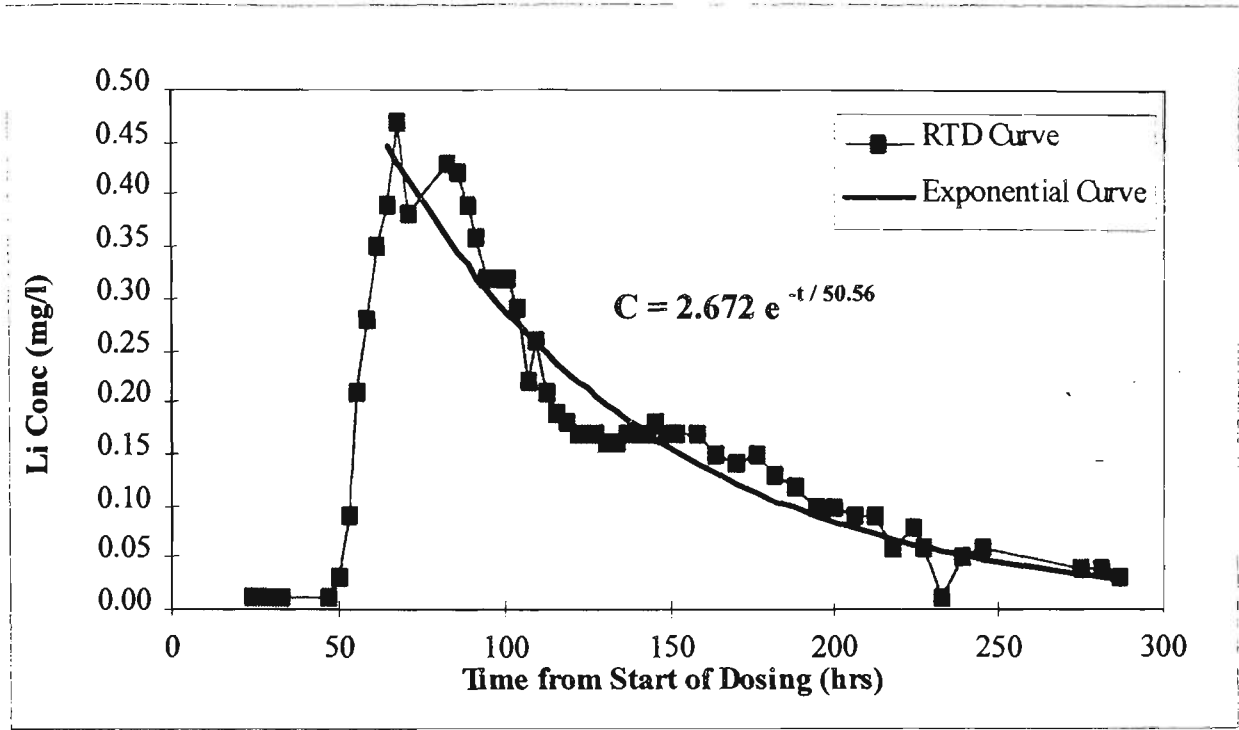
This method estimates the detention time as the time when 50% of the cumulative tracer mass has left the bay. To achieve this, it is necessary to plot the cumulative mass curve. The cumulative mass shows the mass of Li leaving the bay versus the time from the start of dosing. The cumulative mass is computed by summing up the product of flow leaving the bay by its concentration from the start of dosing. It is expressed in kg. As stated by Vorrath (1989), this method can be used as a rough check on the computations of the centroid of the distribution method, and can be considered as a 'rule of thumb' method.

(c) RTD Curves, Fitted Exponential Curves and Cumulative Mass Curves for the Tracer Studies

(i) First Tracer Study

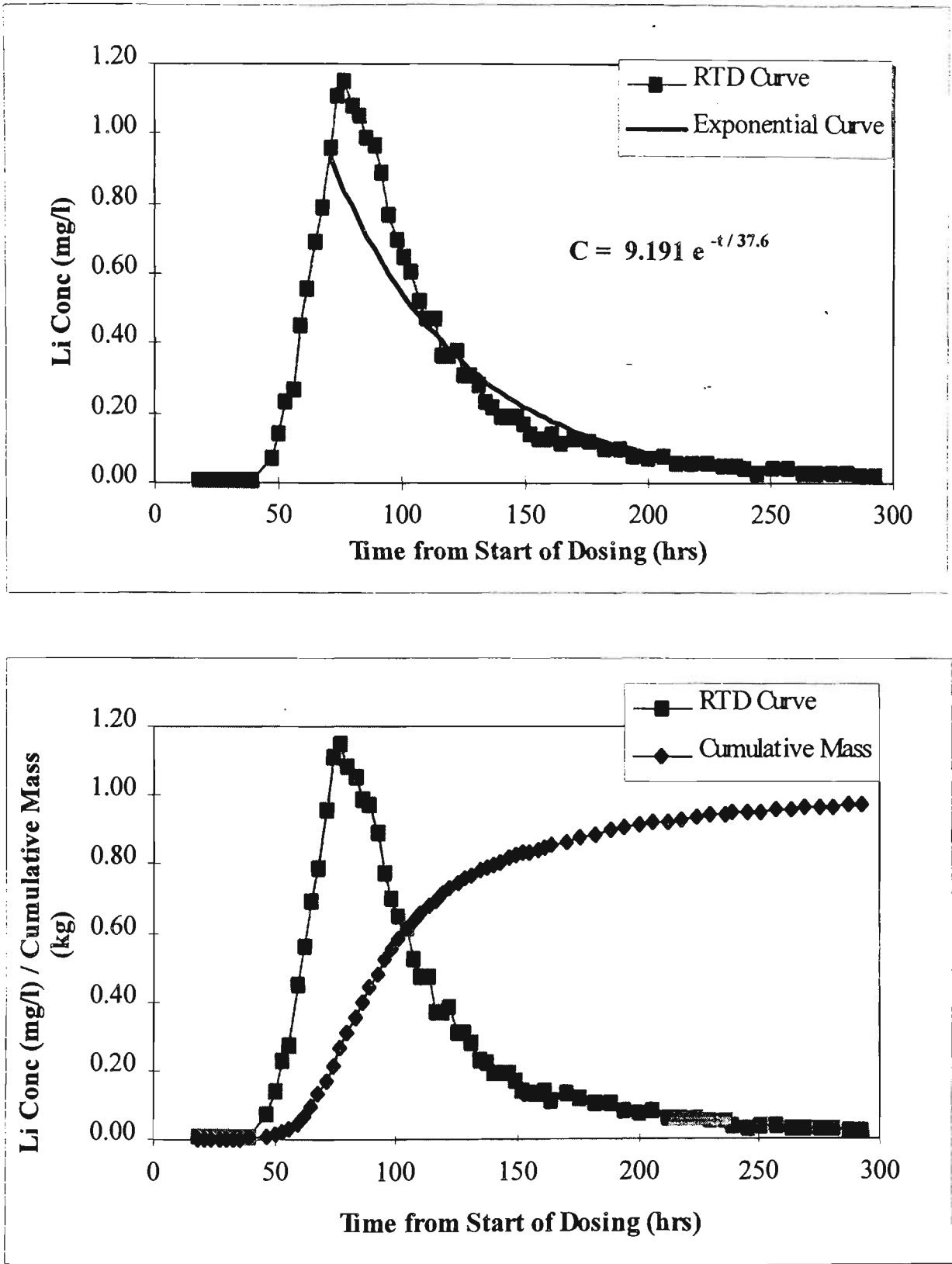
As explained in Section 4.5, the centroid of distribution and 50% mass out (or rule of thumb) were used to compute the detention times of the bays. Therefore, the plots of RTD, fitted exponential curve and cumulative mass curves were required. Figure 4.17 shows plots of RTD, fitted exponential curve and cumulative mass curve for all four bays (i.e. bays 1, 3, 4 and 6) used for the first tracer study. The RTD curves used the measured Li concentration data, while the exponential and cumulative mass curves were fitted using the procedures outlined in (a) and (b) above. The values of C_0 and T were also shown in Figure 4.17 for each bay.

Figure 4.17a does not show the cumulative mass plot of bay 1. This is due to the fact that the flow data at the outlet of bay 1 could not be obtained due to malfunctioning of the data logger (during tracer study 1, i.e. July 7 - July 20, 1997).



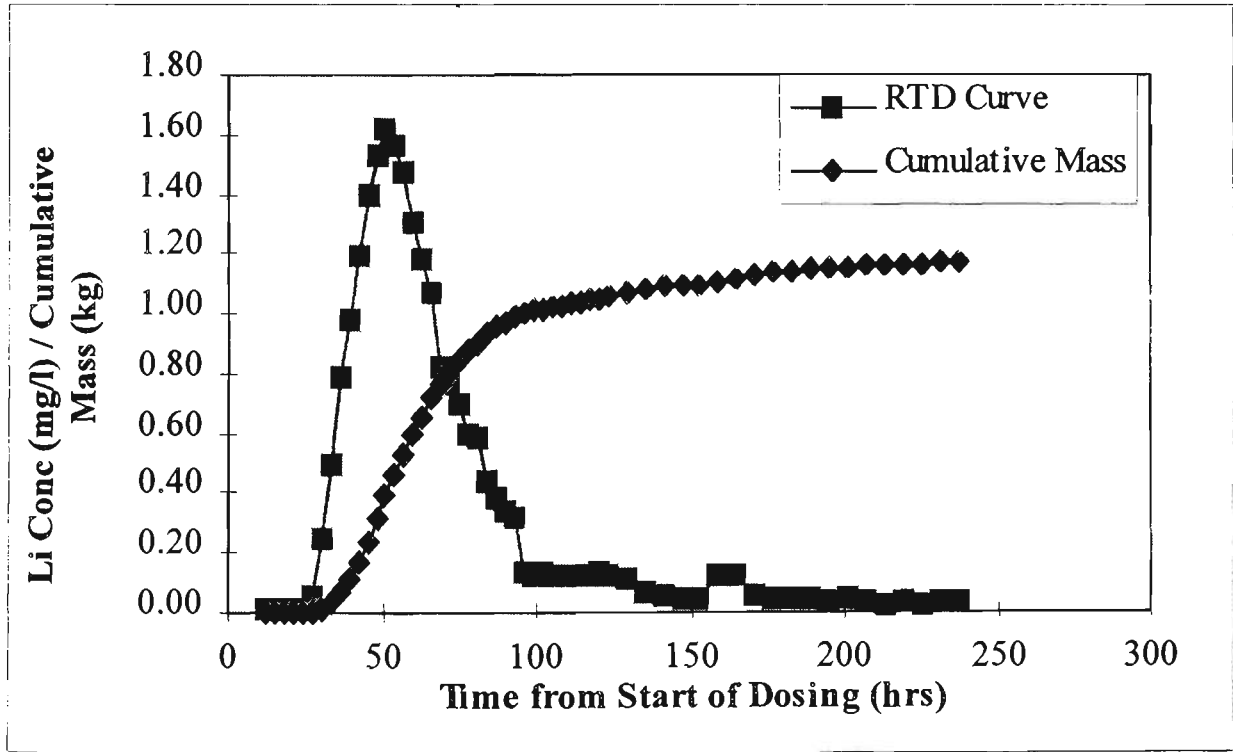
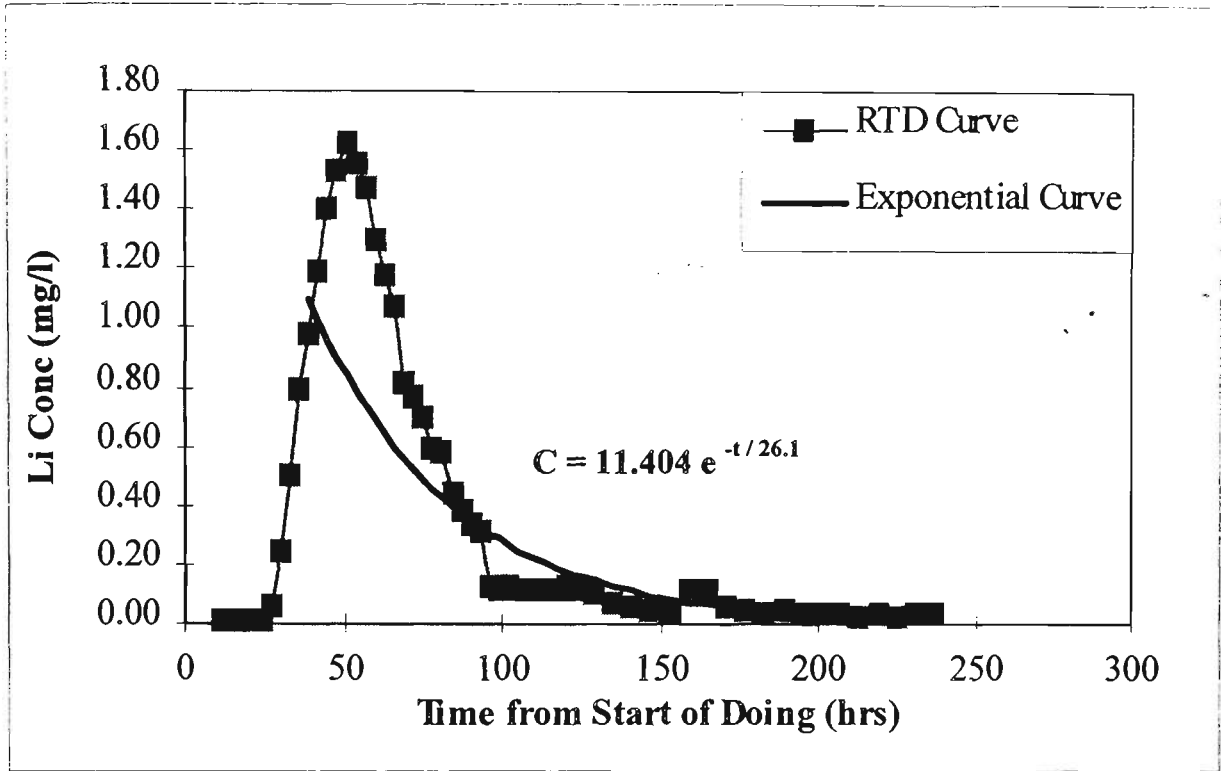
(a) Bay 1

Figure 4.17 RTD, Exponential and Cumulative Mass Curves for Tracer Study 1



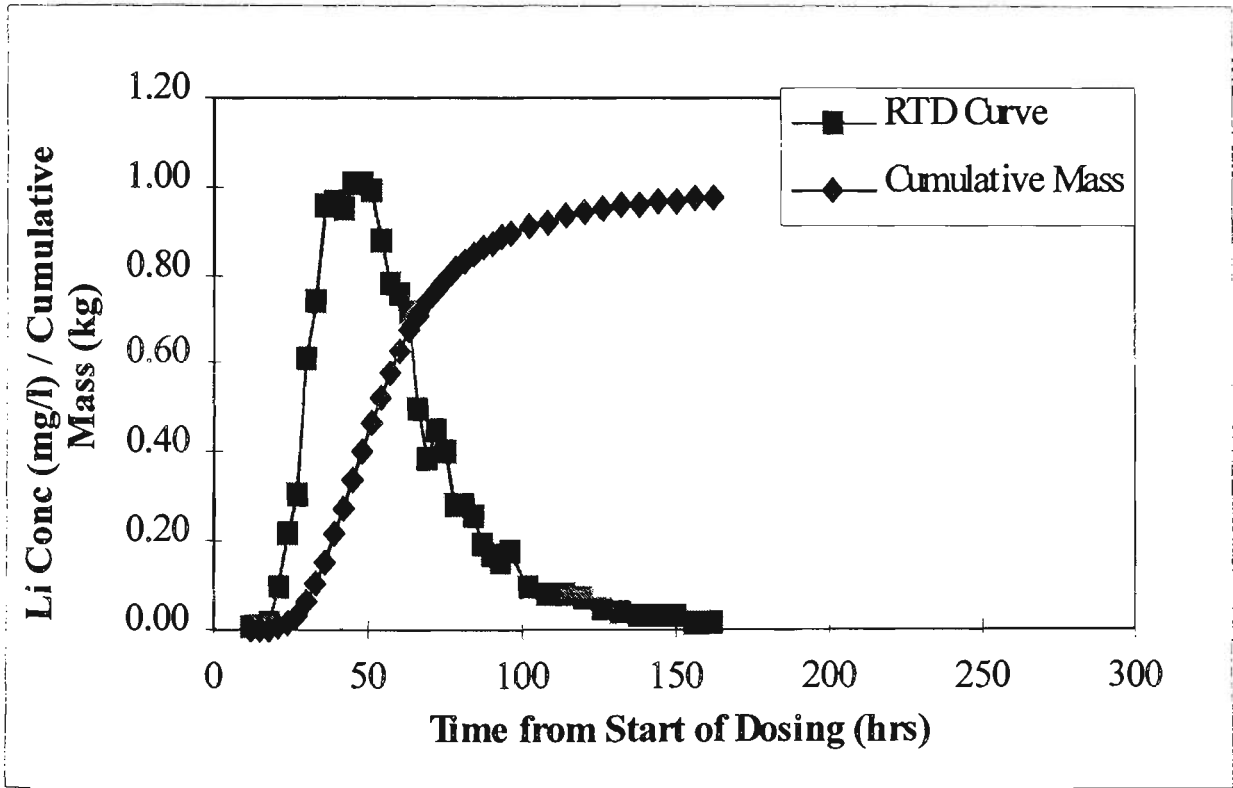
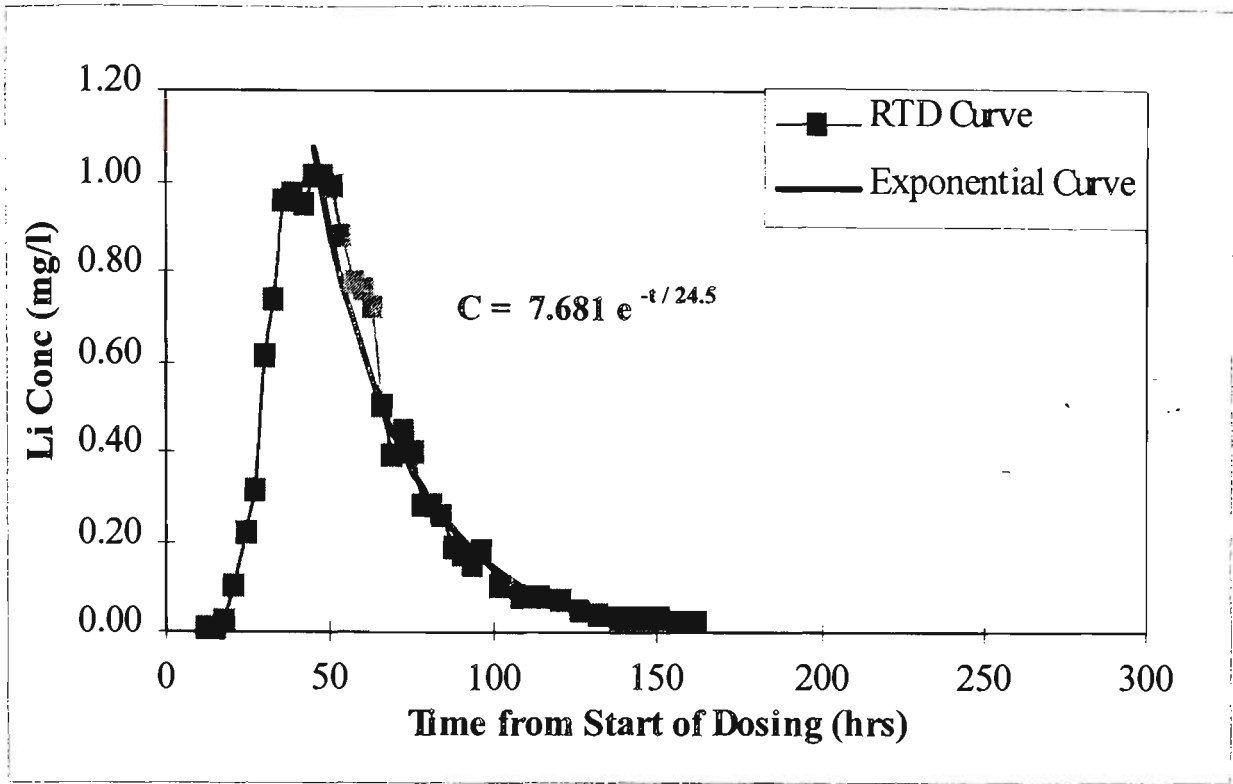
(b) Bay 3

Figure 4.17 RTD, Exponential and Cumulative Mass Curves for Tracer Study 1
(Contd...)



(c) Bay 4

Figure 4.17 RTD, Exponential and Cumulative Mass Curves for Tracer Study 1
(Contd...)



(d) Bay 6

Figure 4.17 RTD, Exponential and Cumulative Mass Curves for Tracer Study 1
(Contd...)

(ii) *Second Tracer Study*

Figure 4.18 shows the RTD, fitted exponential curve and cumulative mass curve for the second tracer study. This figure shows similar patterns to those of the first tracer study, except for bays 4 and 6 which were dosed in both studies. These bays had shown longer detention times in the second study. The longer detention times of bays 4 and 6 in the second study may be due to the following possible reasons:

- possible losses of flow from the bays, which are discussed in detail in Section 4.5.2.3. Losses can be different at different times of the tracer studies in the same bay.
- increase in the density of vegetation and formation of detritus material on the bays, which affect the flow path and hence the detention time of wastewater.

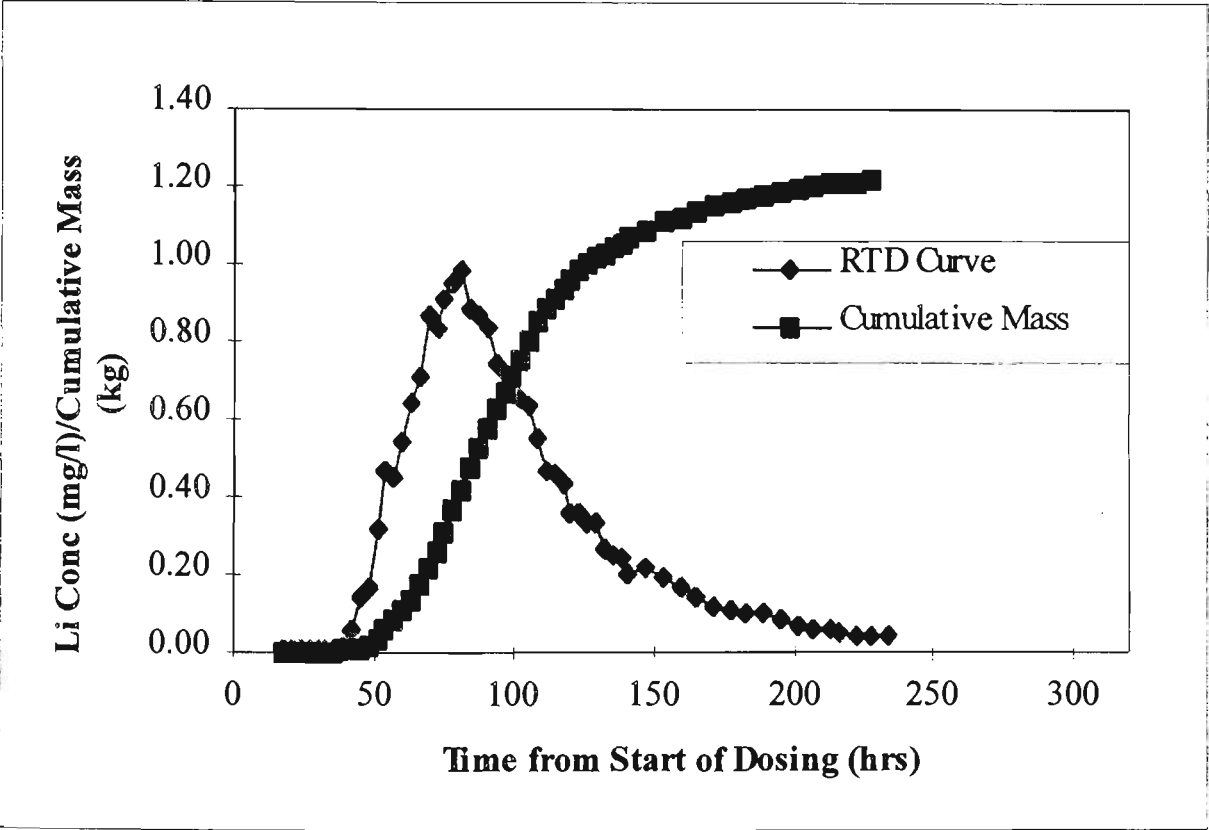
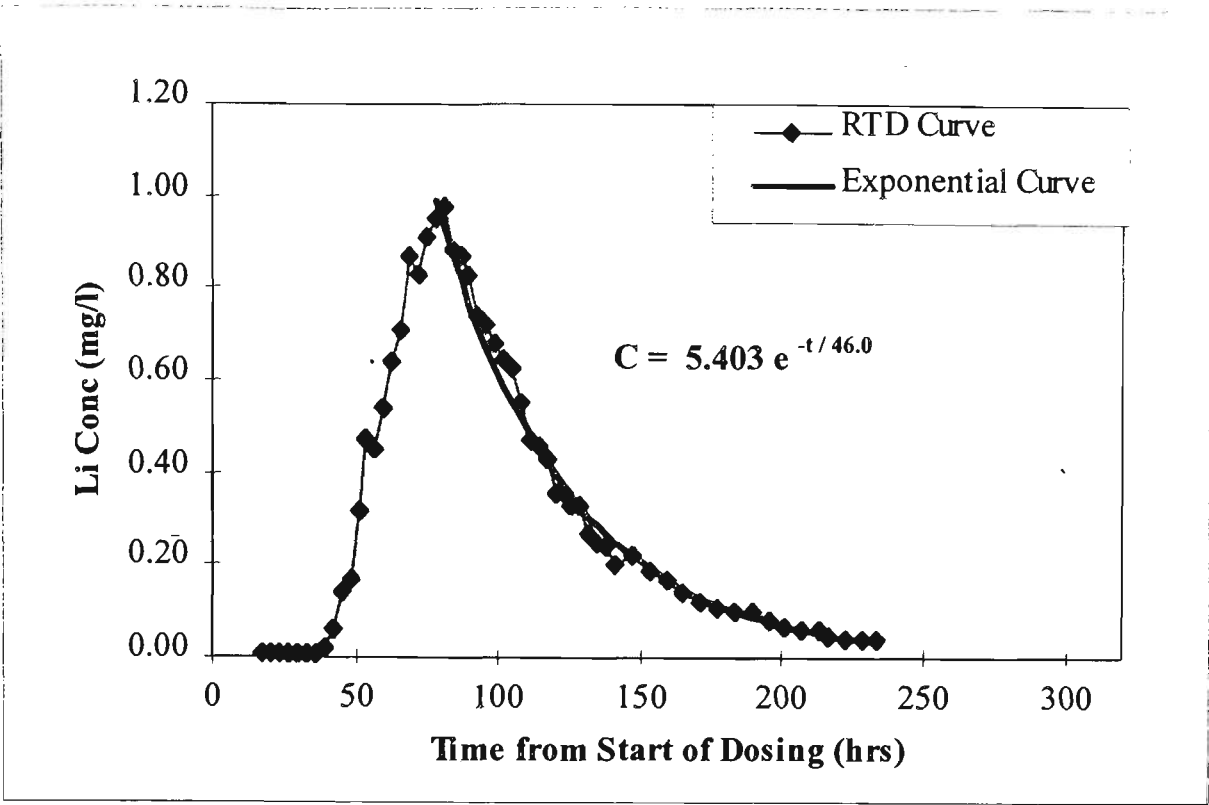
(d) **Detention Times**

The detention times of the bays for both tracer studies were computed using the centroid of distribution and 50% mass out methods described in Section 4.5.2.2, and are presented in Table 4.7. The 50% mass out method, as mentioned earlier, can be used as a rough check on the results of the centroid of distribution method.

Table 4.7 Detention Times From First and Second Tracer Studies

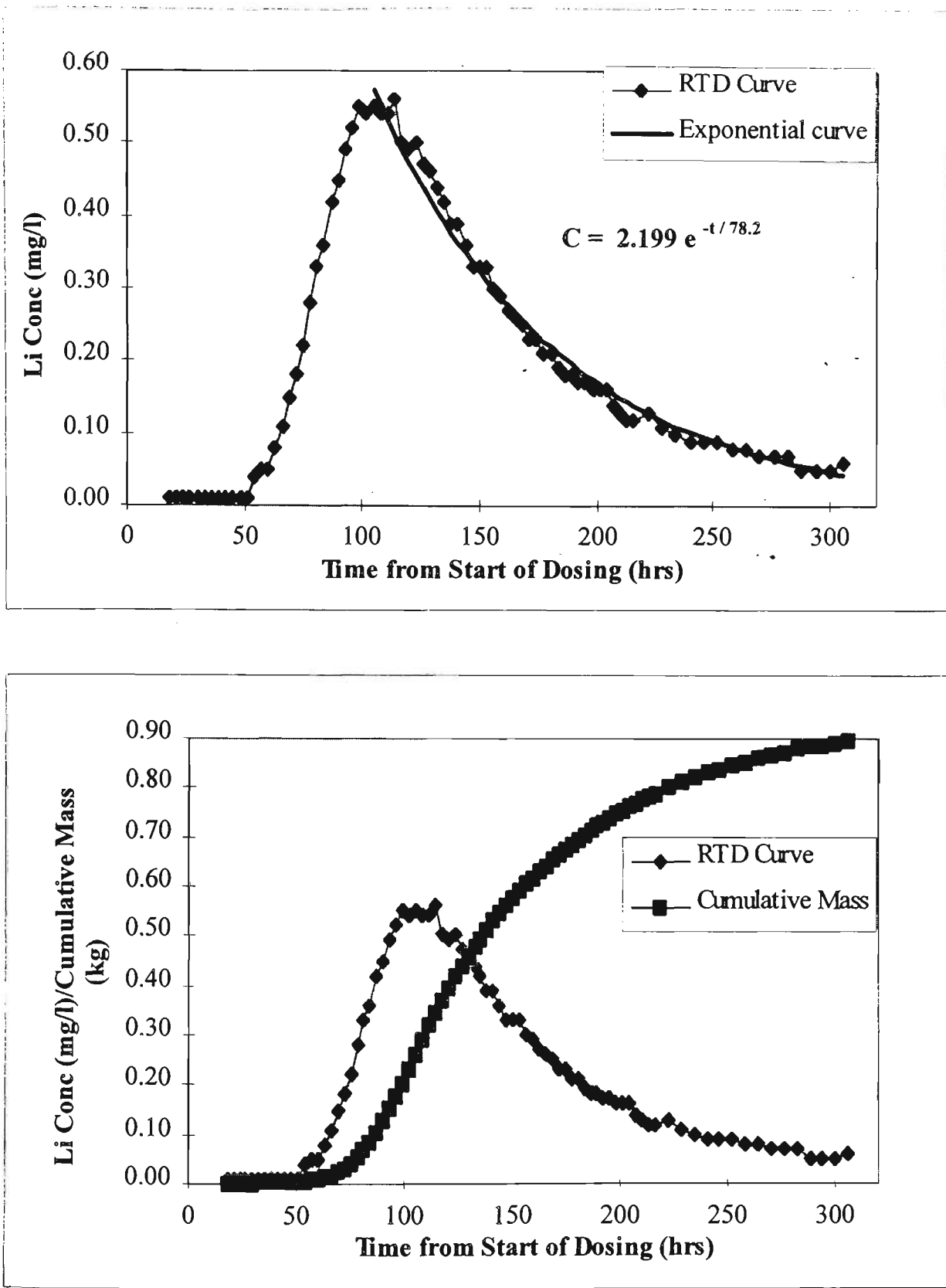
		Detention Time (days)	
Tracer Study No.	Bay No.	Centroid of Distribution Method	50% Mass Out Method
1	1	4.24	-
	3	4.08	3.60
	4	2.67	2.40
	6	2.41	2.16
2	4	4.21	3.75
	5	6.30	6.06
	6	4.08	3.52
	7	3.07	2.60

- Li mass out could not be computed due to lack of outflow data



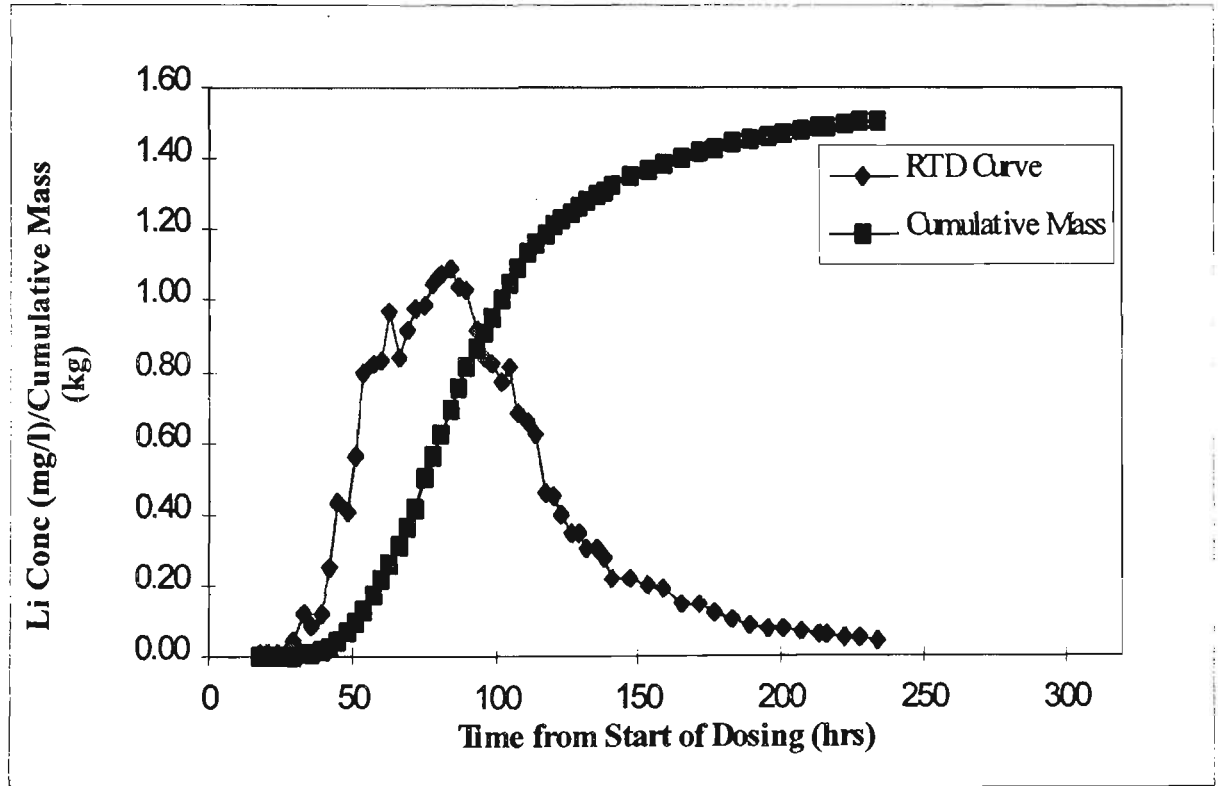
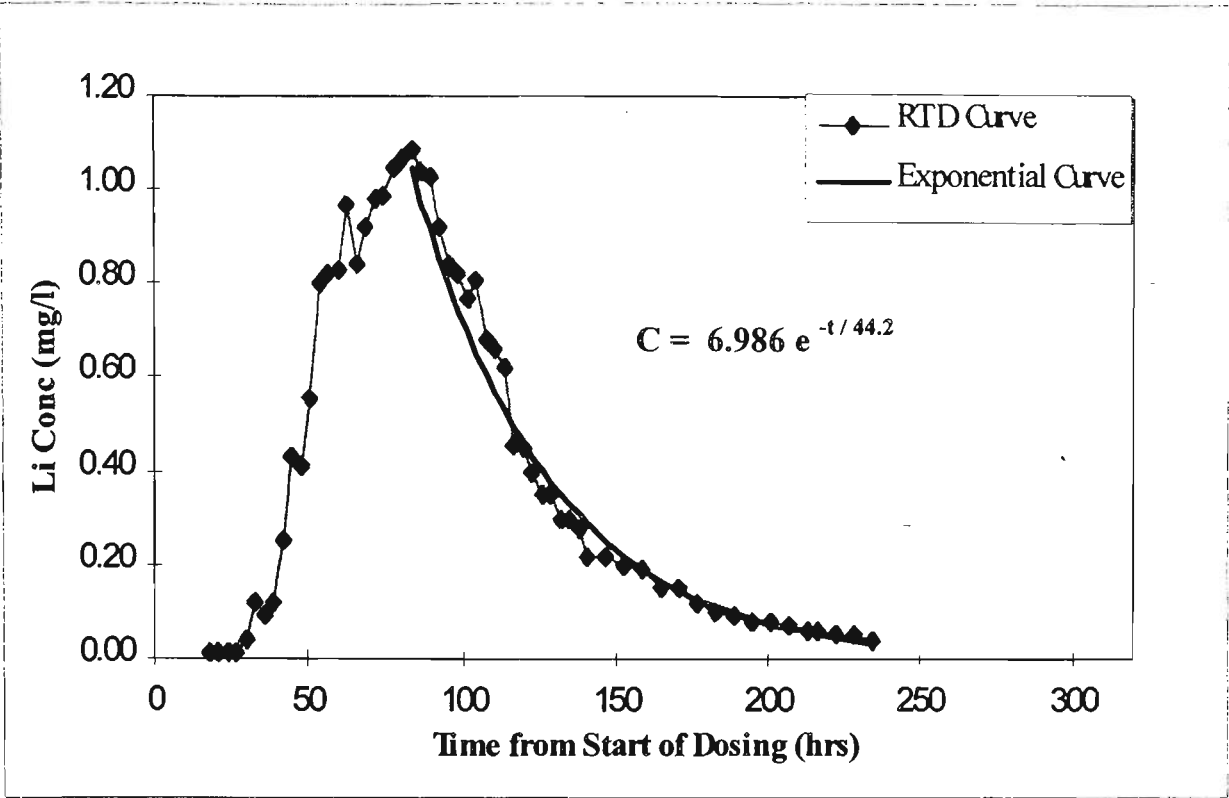
Bay 4

4.18 Figure RTD, Exponential and Cumulative Mass Curves for Tracer Study 2



Bay 5

Figure 4.18 RTD, Exponential and Cumulative Mass Curves for Tracer Study 2
(Contd..)



Bay 6

Figure 4.18 RTD, Exponential and Cumulative Mass Curves for Tracer Study 2
(Contd..)

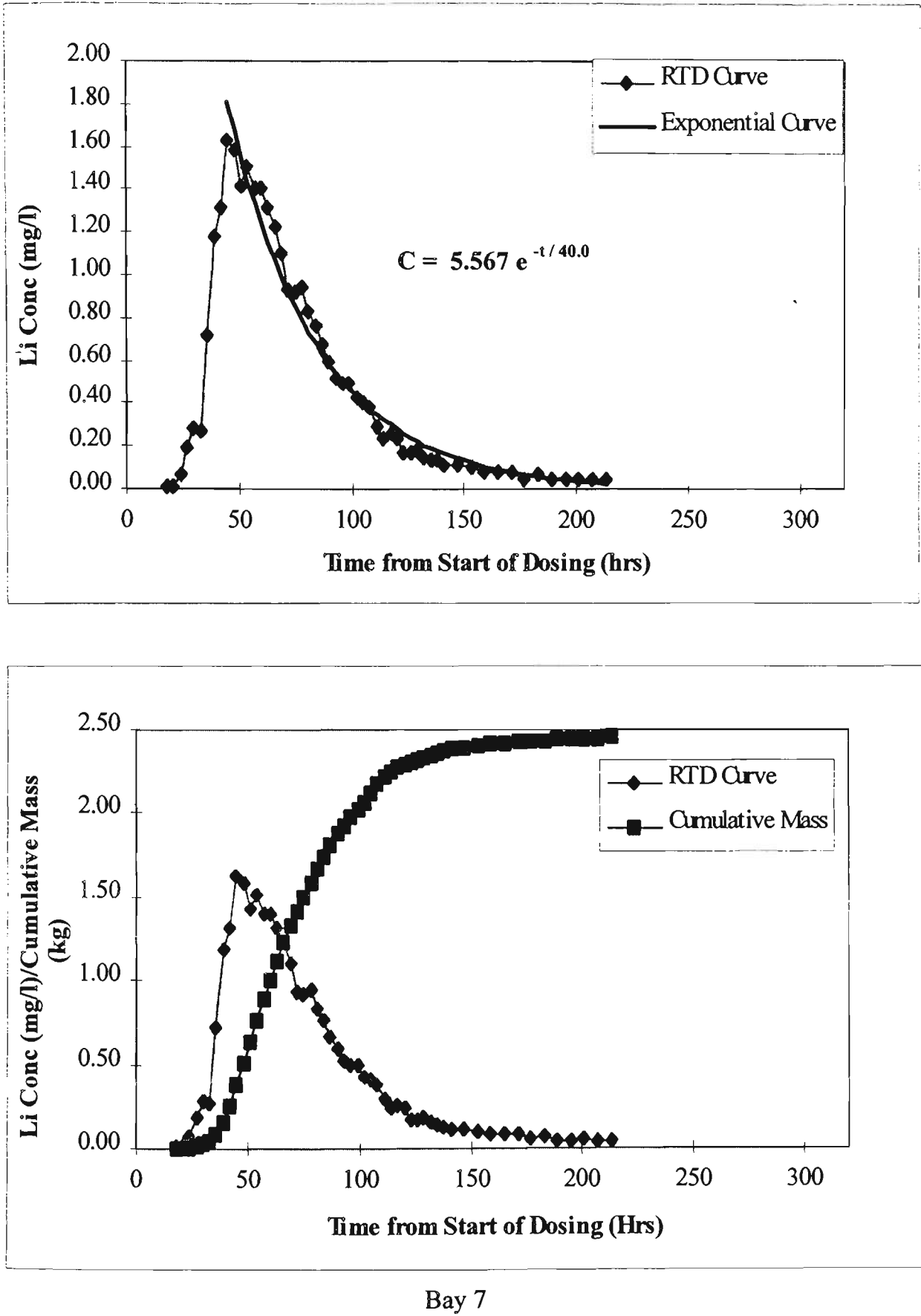


Figure 4.18 RTD, Exponential and Cumulative Mass Curves for Tracer Study 2
(Contd..)

The detention times for bays 4 and 6 were significantly higher in the second tracer study compared to those of the first study. Also, the detention times for bays 5 and 7 of the second tracer study were higher than the corresponding values expected for the loading rates of 40 and 50 mm/day obtained in the first tracer study (i.e. bays 4 and 6 of the first study respectively). These changes in detention times could be partly due to the changes in the flow losses during the two tracer studies. For example, the average inflow and outflow during the first tracer study for bay 6 were 48 and 40 mm/d respectively, while they were 48 and 37 mm/d respectively during the second tracer study. This produced average flow losses of 8 and 11 mm/d during the first and second studies respectively, resulting in longer detention time of 4.08 days in the second study in comparison to 2.41 days of the first study. However, flow losses in bay 4 do not vary significantly between the two tracer studies. The average inflow and outflow of bay 4 during the first study were 42 and 29 mm/d respectively, while they were 42 and 31 mm/d respectively during the second study. These results indicate flow losses of 13 and 11 mm/d during the first and second studies respectively, resulting in detention times of 2.67 and 4.21 days. For bay 4 slightly higher flow losses produced lower detention time, while higher flow losses produced a higher detention time for bay 6. Therefore, on the basis of flow losses alone, it is difficult to explain the difference in detention times of the two tracer studies for bays 4 and 6. However, the increase in detention times of both bays 4 and 6 (i.e. increase of 1.67 and 1.54 days respectively) are quite similar.

In general, the increase in detention times of bays 4, 5, 6 and 7 during the second study suggests that apart from the flow losses, the other factors such as the density of vegetation on the bays, may be important in determining detention time. The density of vegetation and the amount of detritus built up around the vegetations can affect the speed of the flow and the flow path. During the later stages of the second tracer study, the vegetation was fully matured (i.e. more dense), and therefore longer detention times can be expected for the second study than those of the first study.

The detention times obtained from the tracer studies were used for preliminary nutrient modelling (Chapter 6) using the first-order reaction kinetics equations. Table

4.8 shows a summary of detention times recommended from the two tracer studies. Only bays 3, 4 and 6 were used for modelling of nutrients in Chapter 6.

Table 4.8 Detention Times of the Bays

Bay No.	Detention Time (days)	Tracer Study No.
1	4.24	1
3	4.08	
4	4.21	2
5	6.30	
6	4.08	
7	3.07	

For bays 1, 3, 5 and 7, the detention times are available only from one tracer study. The detention times of the first tracer study for bays 4 and 6 were not chosen since they both suffered from high Li losses (as explained in Section 4.5.2.3). In general, lower loaded bays should have a longer detention time compared to the higher loaded bays, assuming no losses and vegetation is the same. This can be seen in Table 4.8, except in bay 5.

It should be noted that the detention time throughout the entire trial period (i.e. from May to October 1997) is not a constant parameter, although the values in Table 4.8 were used in preliminary modelling of nutrients in the bays.

4.5.2.3 **Lithium Losses from the Bays**

As mentioned in Section 4.5.1, the tracer measurements can be used to explain some of the hydraulic losses and gains within the bays. Also it was believed that some of the dosed Li was lost within the bays. Therefore, to further investigate these issues, computations of the Li losses within the bays were carried out.

A mass balance analysis was carried out to determine the Li losses which occurred within the bays during the tracer studies. The Li mass input (i.e. Li mass in) was based

bays. The input concentration of Li of bay 6 in tracer study 1 seems to be consistent with the concentration of other bays in both trials (i.e. Li concentration is higher in the tracer study 1 due to different purities of LiOH). The RTD curve and cumulative mass curve do not show any inconsistencies. Therefore, the Li losses of bay 6 during the first tracer study stands as an outlier. Apart from bay 6 of the first tracer study, bays 4, 5 and 6 show similar losses in both tracer studies. The similarity in the losses in the two studies for these bays suggests that the difference in detention time was probably due to different densities of vegetation on the bays.

According to Table 4.9, bay 5 shows relatively large amount of Li losses (i.e. 38%). According to the flow balance analysis (i.e. Table 4.4), the average flow losses of bay 5 was the highest (i.e. 12 mm/d) amongst all 7 bays throughout the entire duration of the trials. The average flow losses of bay 5 were also the highest (i.e. 12 mm/d) during the second tracer study. At the same time (i.e. during the second tracer study) the average flow losses in the adjacent bays (i.e. bays 4 and 6) were 11 and 11 mm/d respectively (Table 4.4). The average flow losses during this period are very similar for bays 4, 5 and 6. Therefore, on the basis of the flow balance analysis, it is difficult to explain the high Li losses which occurred within bay 5 during the second tracer study.

As seen from Table 4.9, bay 7 gained some Li, suggesting that it received flow from bay 6. The high Li losses and gains by bays 6 and 7 respectively, do not explain fully the modest wastewater losses from bay 6 to bay 7 observed in the flow series plots (Figure 4.6 and Figure 4.12). This may indicate that the cross flows between these two bays occurred fairly far upstream of the bays when the Li plume had not fully dispersed or mixed up and therefore, even small amounts of flow losses might correspond to large amounts of Li mass transfer.

Bay 3 shows modest Li losses (5%) during the first tracer study. This result is consistent with the modest flow losses shown in Table 4.4 during the period of first study (i.e. 7 to 20 July). Bay 3 had the least flow loss of all bays except bay 7 during this period (Table 4.4).

4.6 SUMMARY AND CONCLUSIONS

The hydraulic data required for this project consisted of inflows to the bays, outflows from the bays, and rainfall and evaporation data in bays. The water depths at the inlets and outlets were measured using depth loggers to obtain the inflows and outflows. These water depth measurements were then corrected for temperature variations using correction equations obtained by regression analysis. The corrected water depths were in turn converted to inflows and outflows using rating curves which were developed through calibration of the orifice plates and V-notch weirs at inlets and outlets respectively. Due to malfunctioning and unavailability of some data loggers, some flows were missing during the trial period. These missing flows were obtained via regression analysis.

The rainfall and evaporation data were obtained using a 0.2 mm pluviometer and a Class A evaporation pan respectively. Although temperature corrections were applied on the flow depths, a sensitivity analysis showed that its effect on the flows was insignificant. Flow balance computations based on the flows, rainfall and evaporation data were also carried out to estimate any possible water losses or gains within the trial bays. The flow balances showed some flow losses and gains within the trial bays. The magnitude of the losses and gains were not similar in every case. However, most of the losses occurred from bays 2 and 6. Flow balances showed gains in bays 1 and 7, which suggested that bays 2 and 6 had lost flow to their adjacent bays.

Monitoring of flows at the outlets after the inflows were stopped, showed water gains at bay 1 and losses at bay 2, suggesting that crossflows between these bays must have occurred. It is therefore suggested that care must be taken in the construction of the bays especially in construction of the check banks to prevent cross flows between the bays. In general, the average loss from all the bays over the entire study period was 7 mm/d. This consisted of 1 mm/d loss due to net evapotranspiration and the remaining 6 mm/d due to infiltration.

As part of the hydraulic studies of the bays, the detention times were also measured via two tracer studies. The two tracer studies determined the detention times of bays 1,

3, 4 and 6 and bays 4, 5, 6 and 7 respectively. The second tracer study was conducted to evaluate some uncertainties suspected during the first study. The results of the second tracer study showed longer detention time for bays 4 and 6. It was argued that factors apart from flow losses such as vegetation could have an effect on the detention time of the bays. Li losses computed for the bays during the tracer studies showed losses similar to those obtained from the flow balance studies.

CHAPTER 5

WATER QUALITY DATA AND ANALYSIS

5.1 INTRODUCTION

The hydraulic data collection and analysis presented in Chapter 4 mainly focused on the flows, rainfall, evapotranspiration and their balances. However, Chapter 5 presents the water quality data and analysis. In order to assess the performance of grass filtration bays in wastewater treatment, close monitoring of most water quality parameters is required. Several water quality parameters were considered and monitored in this study, throughout winter 1997 at the grass filtration trial site at WTP. An extensive sample collection program was designed to cover all essential data needed from this study.

The sample collection program consisted of twice weekly in-situ sample collection from the trial site from May to October 1997. These samples were then analysed by the Water Ecoscience laboratories, which are National Association of Testing Authorities (NATA) registered. Although the samples were collected from all 7 grass filtration trial bays at the WTP, bays 3, 4 and 7 were selected as representative bays for discussion purposes in this chapter. The reason for selecting these bays as representative bays are discussed in Section 5.3. Since the study concentrated on nutrients, mass balances of the nutrients were performed on these representative bays. The discussions on these bays mainly relate to the temporal variability of the nutrients and other water quality parameters over the period of the study. The other water quality parameters include physical, chemical and bacteriological parameters, which are presented individually throughout this chapter.

This chapter discusses and presents details of the water quality sampling program, and analysis and discussions on the temporal variability of the data obtained from the site. Furthermore, a detailed analysis of nutrients, based on both concentrations and mass balance approaches is presented in the chapter. The optimal hydraulic loading rate based on the trials is also presented.

5.2 WATER QUALITY SAMPLING AND LABORATORY ANALYSIS

The sampling program for this project was aimed at obtaining representative water quality parameters needed to evaluate the efficiency of the grass filtration bays in removing nutrients (in particular nitrogen) and other pollutants, and to compare the waste treatment performance of the bays at different hydraulic loading rates.

Sampling of the water quality parameters was undertaken for a total of 22 weeks from 13 May to 9 October 1997. Samples were collected between 7:30 and 9:30 AM on Mondays and Thursdays. These two days of the week were particularly chosen to almost evenly space out the sampling interval of twice per week. The water quality parameters analysed and sampled are shown in Table 5.1. Also, the table shows the sampling schedule for each parameter. According to Table 5.1, some parameters were sampled twice weekly and some only once a week. It can also be seen from Table 5.1 that samples of faecal coliforms, total phosphorus (TP) and ortho-phosphorus (OP-P) were collected only from certain locations. These samples were collected less frequently and only at certain locations within the bays, to minimise the costs associated with their analyses by the laboratories. This was done in consultation with Melbourne Water officers, giving due considerations to the importance of these parameters in terms of wastewater treatment and effluent discharge to Port Phillip Bay.

Table 5.1 Water Quality Parameters and Sampling Schedule

Parameter	Location				Number of samples/week	
		All bays				
	Inlet	200 m	300 m	Outlet	2 sample days/week	1 sample day/week
BOD	√	√	√	√		22
CBOD	√	√	√	√		22
Filt. CBOD	√	√	√	√		22
COD	√	√	√	√		22
NH ₃ -N	√	√	√	√	44	
TKN	√	√	√	√	44	
NO ₃ -N	√	√	√	√	44	
NO ₂ -N	√	√	√	√	44	
TSS	√	√	√	√		22
Colour (Pt-Co Units)	√			√	16	
Temperature ^a	√	√	√	√	8	
DO	√	√	√	√	44	
pH	√	√	√	√	44	
Faecal coliforms ^b	√			√	8	
Total phosphorus ^c	√	√		√		9
Ortho-phosphorus ^c	√	√		√		9
Redox Potential ^d						

^aIn-situ measurements at the common inlet, and 200 m, 300 m and outlet of bay 7 only
^bOne each for bays 3, 4, and 6
^cOne each for bays 1, 2, 4, and 6 at 200 m
^dIn-situ measurements (63 measurements) every three weeks at 100, 200 and 300 m for each bay.

As seen from Table 5.1, the water quality samples were collected from:

- the common inlet inside the trapezoidal supply channel,
- the outlet of each bay, and
- the 200 m and 300 m transverse sections of each bay.

These sampling points are shown in Figure 5.1. Single samples were collected at the inlet and the outlets. However, at each of the 200 and 300 m transverse sections, where walkways were constructed to collect the samples with minimum disturbance to Ryegrass, a composite sample was taken from five representative points (approximately equally spaced) along the section, as shown in Figure 5.1. The composite samples were taken to average out non-uniform distribution of water quality parameters along a transverse section due to lack of complete mixing and possible channelling effects on the bays.

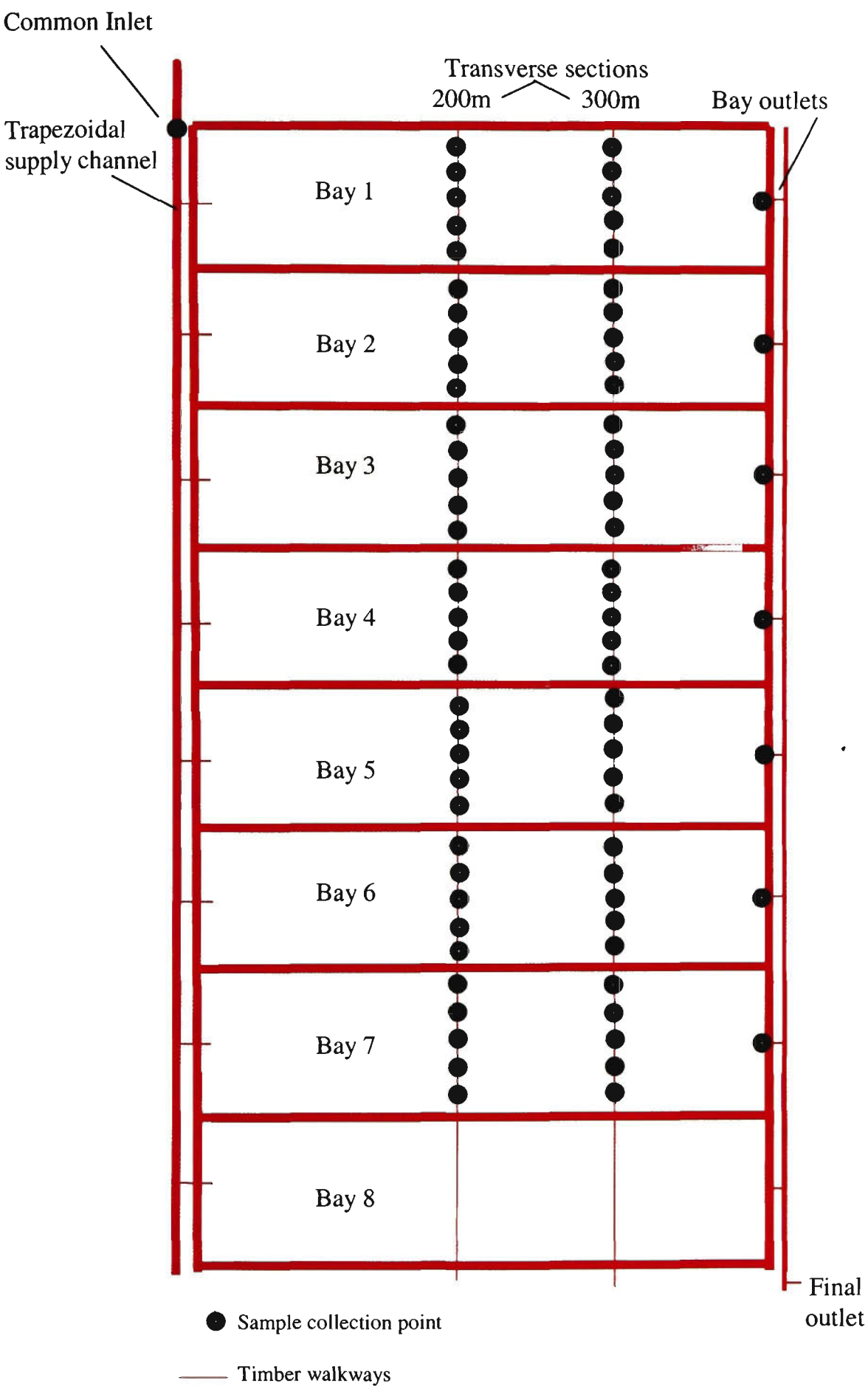


Figure 5.1 Water Quality Sample Collection Points at the Trial Site

A redox potential survey of sludge and water column was done once every three weeks at the 100, 200, and 300 m transverse sections at three equally spaced points along each of the sections for each bay. The redox potential was measured in-situ using a multipurpose pH meter, calibrated by Water Ecoscience staff. Because of this in-situ measurement, it did not incur extra cost for analysis in the laboratories. Walkways were not provided at the 100 m transverse section, which made it difficult to obtain in-situ measurements, especially on rainy days. However, the redox potential measurements were obtained at the 100 m transverse section to see the level of oxygen reduction at the upstream end of the bays.

Water Ecoscience staff were included in the team that undertook the water quality sampling as a quality assurance measure for the sampling program. Sampling procedure is discussed in detail in Section 5.2.1. Once all samples were collected, they were immediately transported to Water Ecoscience laboratories. Apart from pH, temperature, and redox potential which were measured in-situ, all other water quality parameters were measured from collected samples that were preserved for analysis in the Water Ecoscience laboratories. Results obtained from the Water Ecoscience laboratories were plotted in the form of temporal plots for all bays (Appendix A). However, these plots for the representative bays are shown again in Sections 5.4 to 5.8 with relevant discussions. On the basis of temporal trends observed in these plots, the trial period was sub-divided into two periods. These two sub-periods are explained in Section 5.2.2.

5.2.1 Sampling Procedures

The sampling procedures adopted in this project follow the guidelines given in ISO 5667 (Standards Australia, 1991) and were in accordance with requirements set by NATA and ISO 9001 (Standards Australia, 1994). These procedures were the same as those adopted by Melbourne Water in both their current and previous field monitoring studies at the WTP.

General sampling procedures recommended collection of sub-surface samples, wherever possible, to avoid any floating layers that were not representative of the volume. Prior to sample collection all containers which did not contain preservatives were rinsed with the sample.

The measured water quality parameters may be physical, chemical, microbiological and nutrients. The physical parameters included wastewater temperature, water column pH, DO, redox potential, TSS, and colour. As stated earlier, wastewater temperature, water column pH and redox potential measurements were measured in-situ using a pH meter. However, the procedure for collecting samples to be analysed for chemical and microbiological parameters is different and explained below.

Chemical parameters consisted of BOD₅, CBOD₅ and COD. The only microbiological parameter was faecal coliforms and nutrients consisted of nitrogen and phosphorus. As stated in Section 5.2, all samples obtained from the bays were composite samples. The sampling procedure for chemical and microbiological parameters, except for DO, involved filling 1 litre plastic bottles (i.e. as shown in Figure 5.2) within 2 cm of the neck with wastewater directly from the source. The bottles were then tightly sealed and kept cool until delivery to the laboratory.

However, small 300 ml glass bottles were used for obtaining samples to be analysed for DO. Figure 5.2 clearly shows both types of bottles which were used for collection of these samples. Once the glass bottle was filled with the composite sample, reagents were added to the bottle (in accordance to the Water Ecoscience sample collection procedures) to ensure the quality of the sample was not altered before its delivery to the laboratory.

The reagents added to the samples were manganous sulphate and alkali-iodide-azide. 1 ml of manganous sulphate was added to the bottom of the glass bottle and 1 ml of alkali-iodide-azide was added to the top of the bottle. The bottle was then shaken vigorously and sealed. After the addition of the two reagents and sealing the bottle, a yellow to brown precipitate was gradually formed and settled to the bottom. If air



Figure 5.2 300 ml Glass Bottle and 1 Litre Plastic Bottle used in Sampling

bubbles appeared in the bottle after it was shaken and sealed, then the procedure was repeated and a new sample was obtained.

The samples collected in the small glass bottles (i.e. for DO analysis) were also used for faecal coliforms analysis. Therefore, these bottles were sterile bottles and for that purpose the lid was to be held in hand and not put down. Also, since the inside of the bottle was sterile, any touch of fingers inside the bottles was to be avoided. Once the sample was obtained, the bottle was tightly sealed and kept cool until delivery to the laboratory.

5.2.2 Sub-Periods Within Sampling Period

Throughout the sampling period, the effluent characteristics of the trial bays showed significant temporal variability (Appendix A). It is clear from most plots in Appendix A, especially on nitrogen and phosphorus, that the trial period has two distinct sub-periods. Therefore, the trial period was subdivided into two sub-periods (i.e. Period 1 and Period 2) and the effluent data were analysed considering these two sub-periods. Most of the discussion in this chapter is based on these two sub-periods. Periods 1 and 2 are approximately equal in duration (74 and 76 days respectively). The two sub-periods and their respective durations are:

- Period 1 13 May to 24 July, and
- Period 2 25 July to 9 October.

5.3 REPRESENTATIVE BAYS

Wastewater treatment performance of grass filtration bays is affected by the occurrence of flow losses and cross flows between the bays. If significant flow losses and cross flows between the bays occur, it is difficult to interpret the effluent water quality data in terms of wastewater treatment performance. On the basis of minimising analysis problems caused by flow losses and cross flows, three bays (i.e. bays 3, 4, and 7) were chosen as representative bays for further discussion in this chapter. Nevertheless, the results from the other bays were sometimes used to complement the conclusions, especially on the range of performance efficiency for different hydraulic loading rates. The following criteria were used to select the representative bays.

- each bay represented one of the higher hydraulic loading rates of 30, 40 and 50 mm/d,
- the bays showed minimal losses through cross flows to the other bays, and
- the bays produced reliable and accurate data.

Bay 1 was not selected because it received a large amount of flow from bay 2, as discussed in Section 4, and also it did not satisfy the first criterion as listed above (i.e. it had a hydraulic loading rate of 20 mm/d). Furthermore, bay 1 does not have outlet flow data during Period 1 because it had a faulty outlet depth logger during this period.

The remaining six bays had pairs of 30, 40 and 50 mm/d hydraulic loading rates, and the representative bays were chosen on the basis of hydraulic losses which occurred within these six bays. Table 5.2 summarises the flows and hydraulic losses of all seven trial bays, which were discussed in detail in Section 4.4. The average losses for the whole period (i.e. for Periods 1 and 2) shown in the table are the same as those

shown in Table 4.4. The average losses are the difference of the average inflows and outflows for the respective periods.

Table 5.2 Inflow, Outflow and Losses of all bays for Periods 1 and 2

Bay No	Hydraulic Loading Rate (mm/d)	Average Inflow (mm/d)			Average Outflow (mm/d)			Average Losses (mm/d)		
		Period			Period			Period		
		1	2	1 & 2	1	2	1 & 2	1	2	1 & 2
1	20	-	21.0	-	-	17.0	-	-	3.0	-
2	30	29.5	30.0	30.0	16.5	18.5	17.5	13.0	11.5	12.2
3	30	27.5	29.4	28.4	22.0	25.0	24.6	5.5	4.4	5.0
4	40	38.5	42.0	40.2	26.0	33.4	29.7	12.5	8.6	10.5
5	40	40.0	41.0	40.5	30.0	27.0	28.5	10.0	14.0	12.0
6	50	45.0	49.4	47.1	43.0	39.5	41.3	2.0	9.9	6.0
7	50	46.0	51.0	48.5	43.0	52.0	47.5	3.0	-1.0	1.0

- flows not available due to lack of field data (i.e. faulty data loggers at inlet and outlet)

As can be seen from Table 5.2, bays 2 and 3 were loaded at 30 mm/d, bays 4 and 5 at 40 mm/d and bays 6 and 7 at 50 mm/d. Within these three pairs of bays, bays 3, 4 and 7 show the minimal hydraulic losses over the whole period of study respectively. Therefore, on the basis of minimal hydraulic losses, bays 3, 4 and 7 were chosen as the representative bays for further discussion.

5.4 PHYSICAL PARAMETERS

The physical parameters monitored for this trial were wastewater temperature, water column pH, DO, redox potential, TSS and colour. These parameters are indicative of different aspects of wastewater treatment using grass filtration. For instance, monitoring of temperature, pH, DO and redox potential was essential to detect the level of biological activity that occurred within the bays with respect to nitrogen transformations and losses, while TSS and colour measurements of the wastewater

were essential in identifying the level of treatment provided by the Italian Ryegrass with respect to the EPA licenced discharge levels.

5.4.1 Temperature

Wastewater temperature was measured twice a week, throughout the trial period at the common inlet and at 200 m, 300 m and outlet of bay 7. All temperature measurements were obtained between 7.30 to 9.00 AM on water quality sampling days. The temporal plot of temperature at the above locations throughout the trial period is shown in Figure 5.3. It can be observed from this plot that the influent temperature followed the expected seasonal trend with high values of about 13°C in May, gradually decreasing as winter progressed to a minimum of about 7 °C, before starting to increase from early September to values of around 14 °C by the end of September.

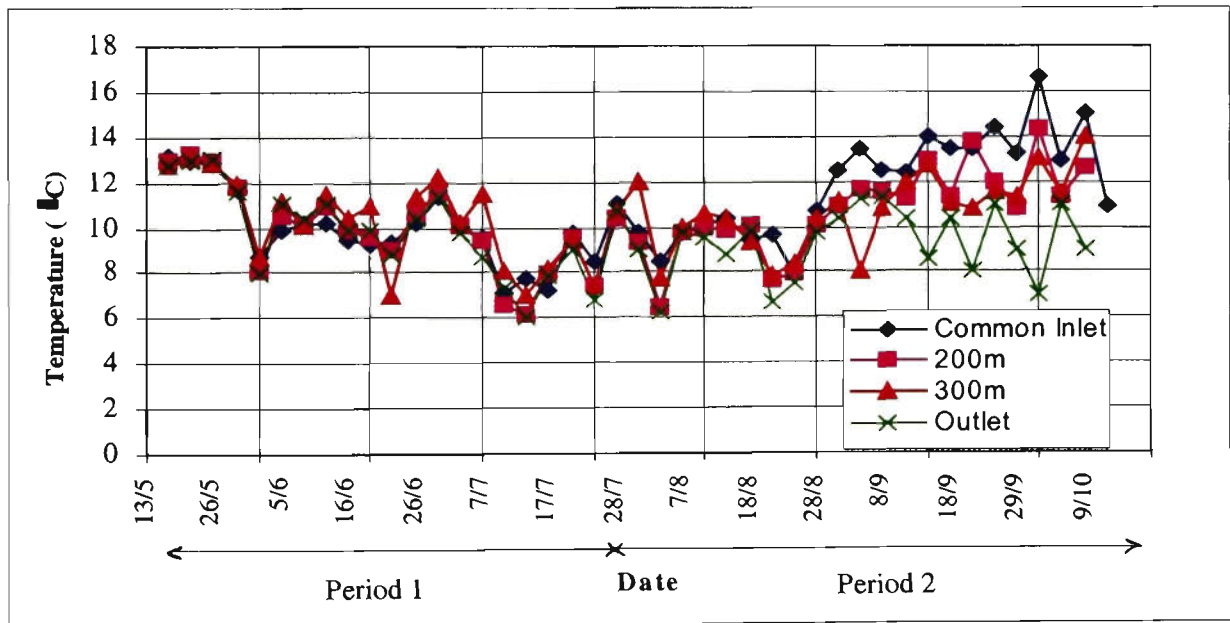


Figure 5.3 Wastewater Temperature at the Common Inlet, 200m, 300m and Outlet of Bay 7

The temperature was measured only at bay 7 as it was assumed that the temperatures in the other bays were almost the same. Figure 5.3 shows larger variation between the influent and effluent temperature towards the end of the trials, indicating that the wastewater became marginally cooler as it flowed downstream.

During the entire trial period, the influent temperature was between 7 and 17 °C, while the temperatures on the bay and at the outlet were in the range 5 to 14 °C. The average wastewater temperatures at the common inlet, 200 m, 300 m, and at the outlet were 10.9, 10.3, 10.4 and 9.9 °C respectively. These average temperatures show that the influent gets cooler as it travels through the bay.

5.4.2 pH

Water column pH was measured at the common inlet and the 200 m, 300 m points and the outlets of all bays twice weekly for the whole duration of the trials. The temporal plots of pH at the common inlet and the outlets for the representative bays are shown in Figure 5.4. However, the temporal plots of pH at the inlet and 200 m, 300 m and the outlets of all seven bays are shown in Figure A.1 of Appendix A.

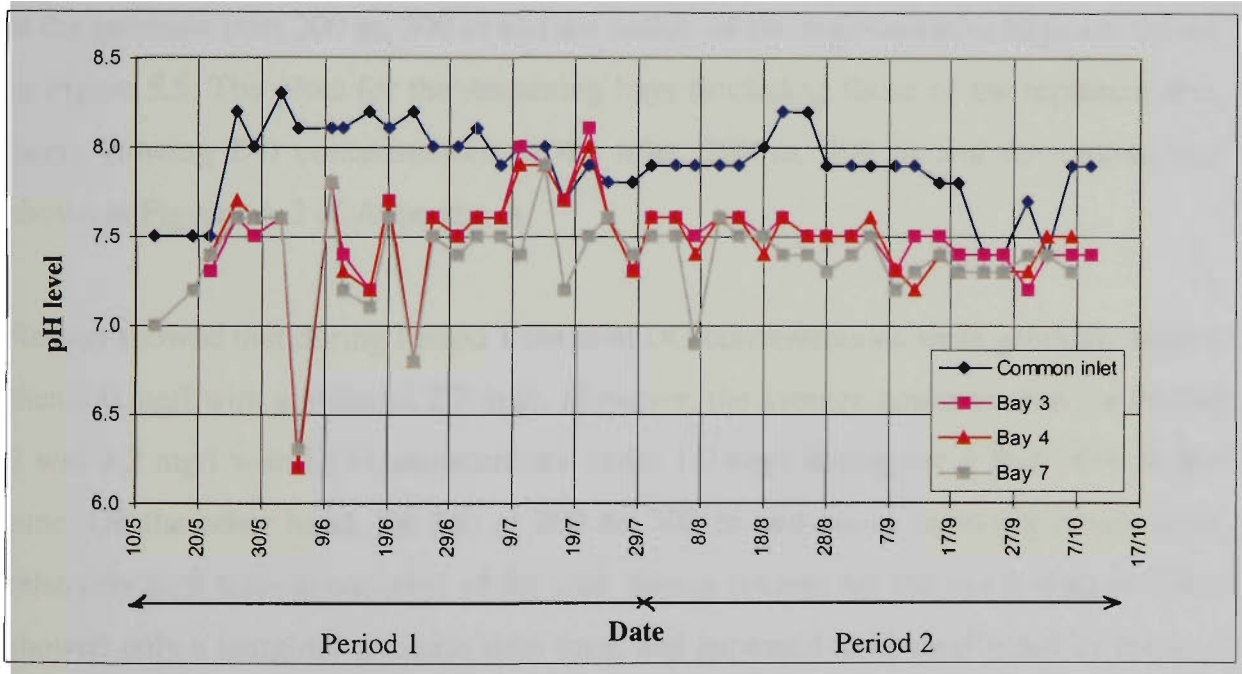


Figure 5.4 Temporal Plots of pH at Common Inlet and Outlets of Representative Bays

The inlet pH showed a steady pattern throughout the trial period with minor fluctuations at the beginning and towards the end of the trials (Figure 5.4). The pH values of the influent wastewater were in the range 7.4 to 8.3 during the trial period, indicating that the wastewaters were alkaline. However, the effluent pH values ranged between 6.2 to 7.9. There were few acidic effluent samples (i.e. with pH levels less than 7). These particular acidic samples may have been affected by presence of

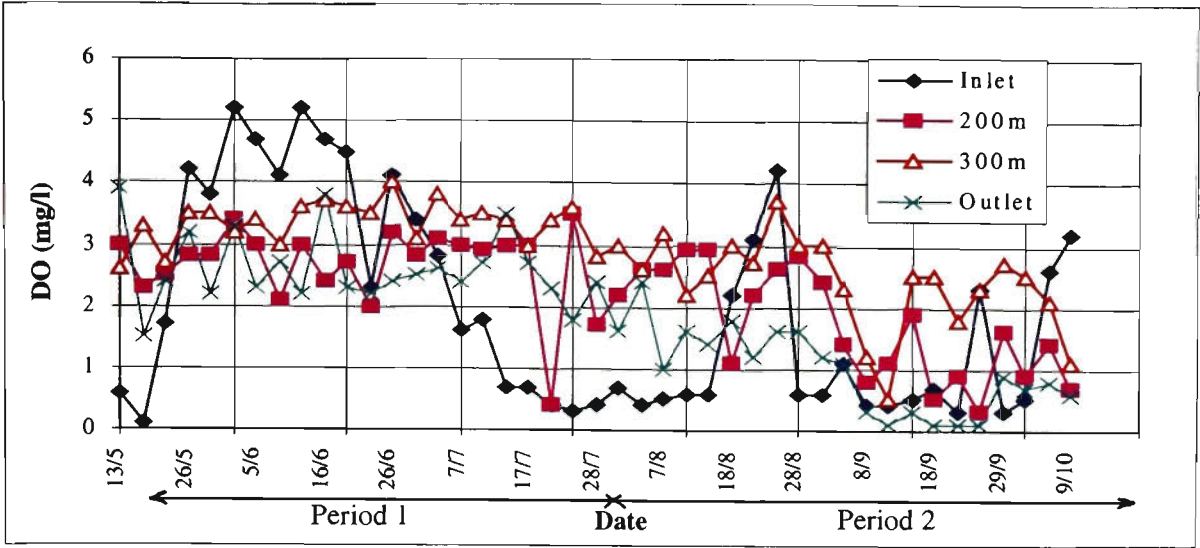
minerals within the bays. Apart from these few acidic effluent samples, Figure 5.4 shows no significant difference between the pH levels of the wastewaters during Periods 1 and 2 of the trials.

According to Figure A.1 of Appendix A, the influent pH is always higher than the pH levels of the wastewater at the 200 m, 300 m and the outlets. However, the pH levels at the 300 m mark are mostly higher than the 200 m and the outlet pH levels for all seven bays. The reason for this particular trend is not known.

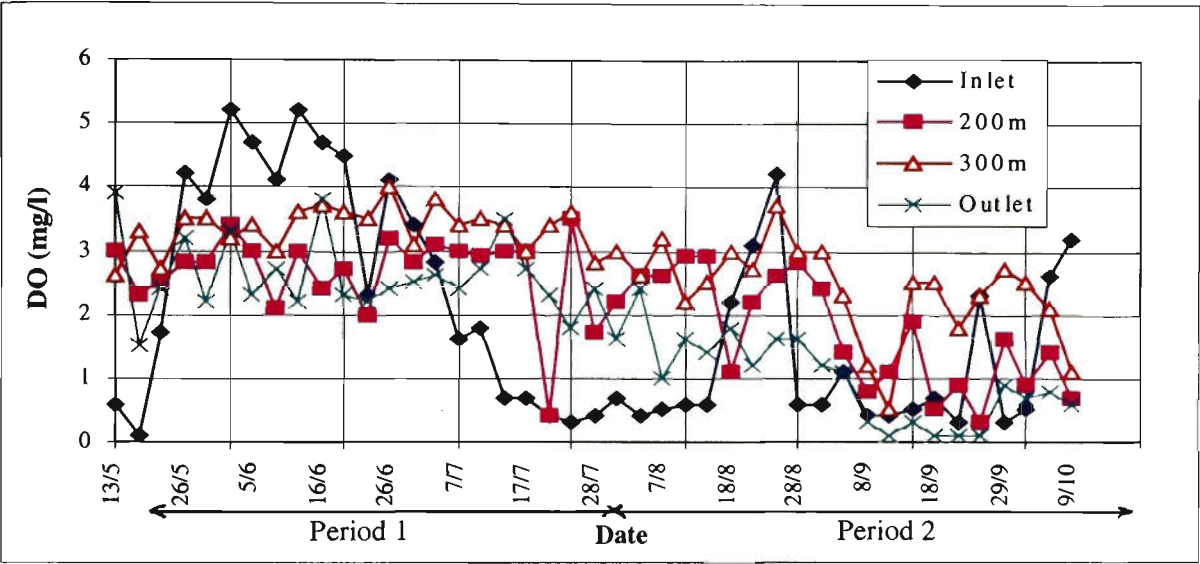
5.4.3 Dissolved Oxygen (DO)

DO measurements were conducted at the common inlet 200 m, 300 m and the outlet of all seven bays twice per week for the entire duration of the trials. The measurements were carried out between 7:30 and 9:30 AM. The temporal plots of DO at the common inlet 200 m, 300 m and the outlets of the representative bays are shown in Figure 5.5. The plots for the remaining bays (including those of the representative bays) showing DO concentrations at the inlet, 200 m, 300 m and the outlets, are shown in Figures A.2 of Appendix A.

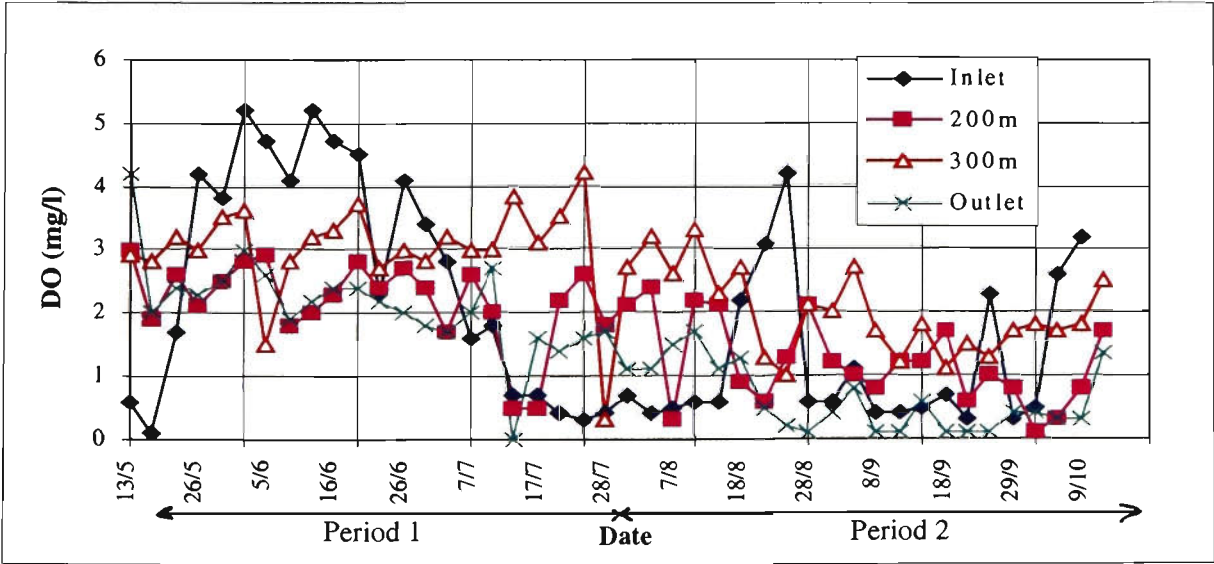
Results showed that during Period 1 the inlet DO concentrations were generally higher than 2.0 mg/l with a mean of 2.7 mg/l. However, the average concentration for Period 2 was 1.2 mg/l with DO concentrations under 1.0 mg/l during more than 60% of the time. On the other hand, the DO at 200 m, 300 m and outlet sampling points were relatively high throughout most of the trial period (except for the last 6 weeks). They showed only a marginal decrease with time, and appeared to be unaffected by the low influent DO after 7 July. However, the DO concentrations were consistently low for all bays during the last six weeks of the sampling program. Considering the low influent DO from 7 July to the end of the trials, it can be said that the DO concentrations on the filtration bays depend more on sources from the roots of Italian Ryegrass during photosynthesis and less on the DO level of the influent. However, the gradual DO decrease on the bays with time and the very low DO levels during the last six weeks of the trial is due to the ageing of the Italian Ryegrass, which reduces



(a) Bay 3



(b) Bay 4



(c) Bay 7

Figure 5.5 DO Plots for Inlet and 200m, 300m and Outlets of Representative Bays

growth and photosynthetic activity. When photosynthetic activity and grass growth have reduced, the DO input from the roots of the grass into the water column is also reduced.

There was also a general trend of initial increase in DO concentration up to the 300 m point and then a decrease in the most downstream part of the bays (Figure 5.5). For example, the average DO concentrations at the common inlet and 200 m, 300 m and outlet points of bays 3 and 4 during the entire trial period were 1.9, 2.4, 3.2, 1.8 and 1.9, 2.2, 2.9, 1.8 mg/l respectively, while the corresponding values for bay 7 were 1.9, 1.7, 2.5, 1.4 mg/l. Similar trend can be observed in the remainder of the bays as shown in Figure A.2 Appendix A. Bays 3 and 4 show this trend, where there is an increase in DO concentration up to the 300 m point, whereas bay 7 shows an immediate reduction in DO up to 200 m and then an increase. This trend is due to the net effects of oxygen addition by the grass and oxygen utilisation for oxidation of pollutants. The oxygen addition is due to the photosynthetic activity which produces DO through the roots of the grass into the watercolumn. As the wastewater travels through the bay, the produced oxygen is then utilised by the grass for oxidation of pollutants and therefore the DO levels decrease between the 300 m point and the outlet. Therefore, this analysis suggests that most of the photosynthetic activity occurs upstream of the bays (i.e. within the first 200 meters).

The DO data used for the above discussion were based on samples that were taken between 7:30 and 9:30 AM on sampling days, throughout the trials. Thus, the values neither reflect the mean daily DO concentration nor the diurnal variations. It was also believed that the DO levels varied with the time of the day. Therefore, a limited measurement program was carried out near the end of the trials to investigate the diurnal variation of DO, using a Grant YSI 3800 DO logger. This program included measurement of DO levels at 1-hour intervals for 24 hours at the common inlet and the outlet of bay 7 (see Figure 5.12). The measurements at the common inlet were repeated to see any variations in DO levels between two different days and reported as Run 1 and Run 2 as shown on Figure 5.6. Although the plant growth and photosynthetic activity had been significantly reduced (i.e. since these measurements were obtained near the end of the trials), compared with the rest of the trial period, there was a well-defined diurnal patterns seen on Figure 5.6. This figure clearly indicates that the DO levels did vary with the time

of the day. Therefore, it is suggested that any future trials should consider 24-hour DO concentration measurements, since such data are important for understanding the pollutant transformations and behaviour on the bays.

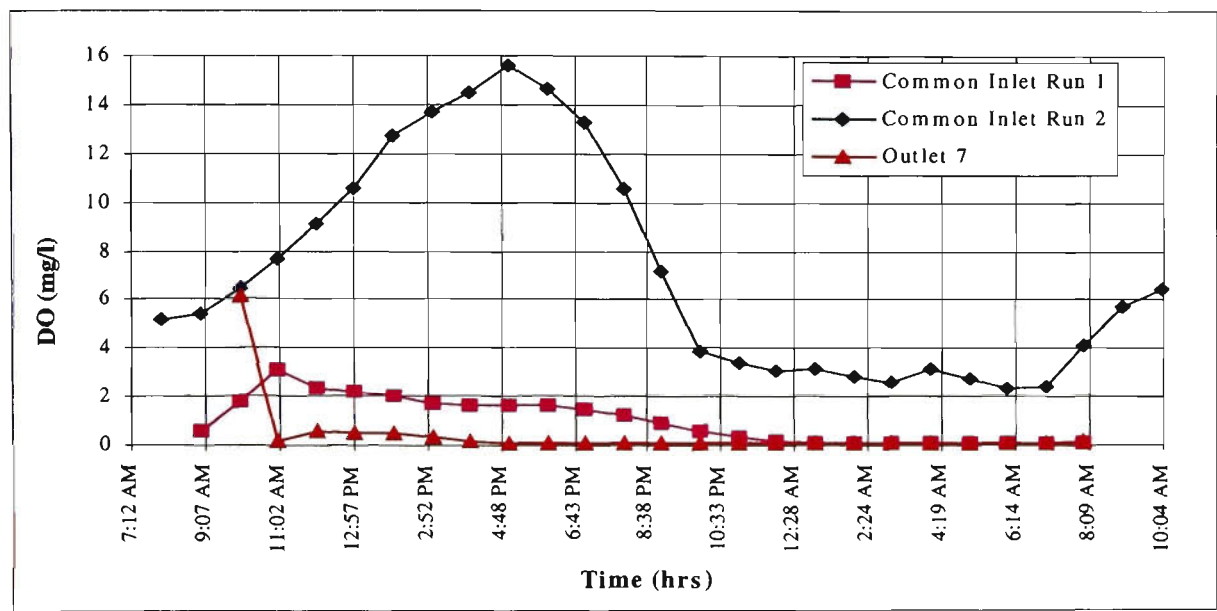
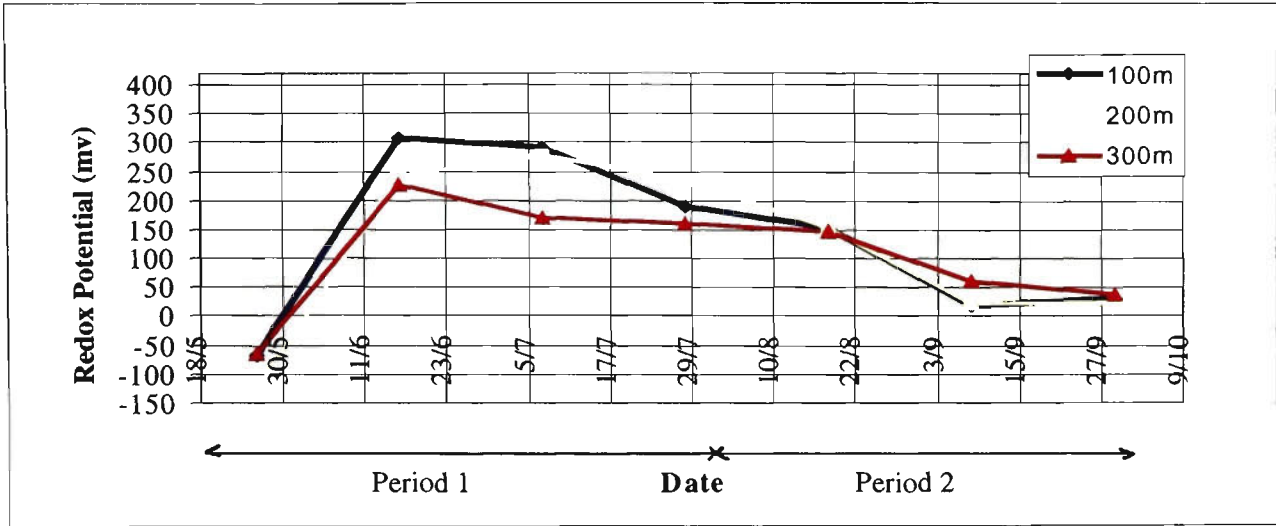


Figure 5.6 DO Diurnal Plots at Common Inlet and Outlet of Bay 7

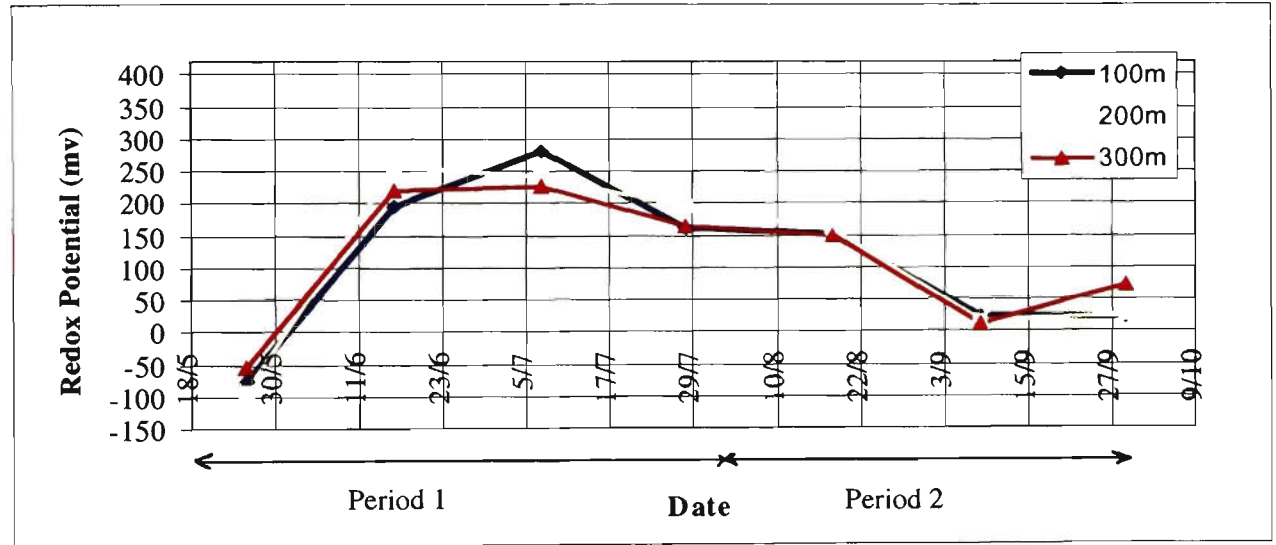
5.4.4 Redox Potential

Redox potential of the water column and sludge was measured at the 100, 200, and 300 m transverse sections of each bay, once every three weeks. Redox potential is essentially a measure of the effectiveness of the dissolved oxygen content of a waste. Therefore, apart from the 200 and 300 m transverse sections, it was also measured at the 100 m transverse section to observe the level of biological activity in the upper end of the trial bays.

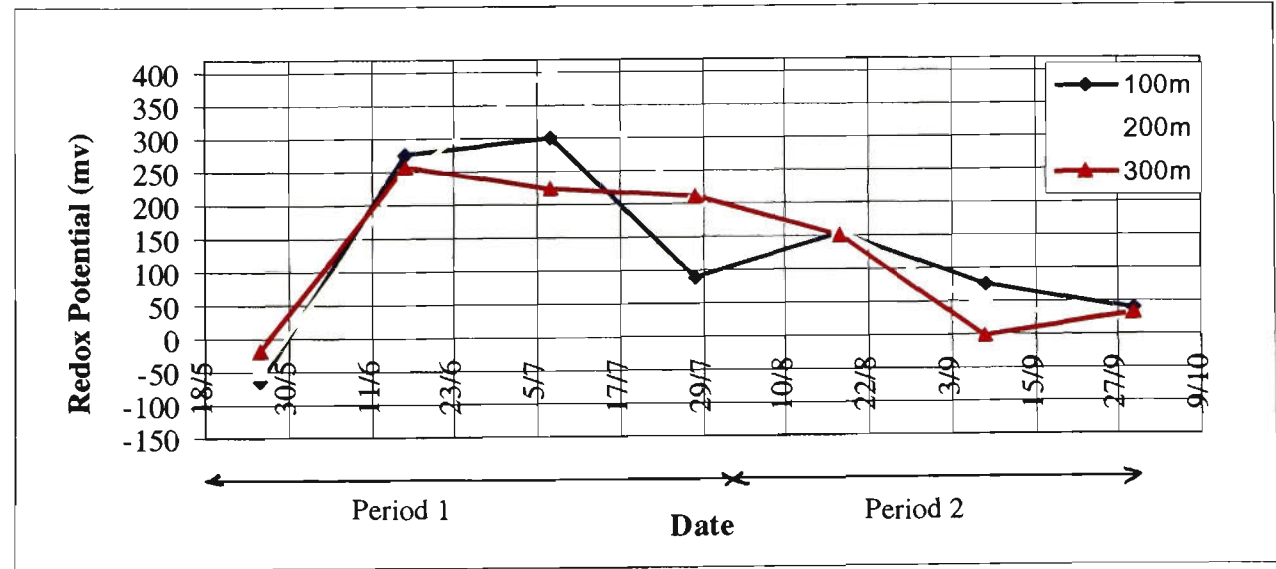
Figure 5.7 shows the temporal plots of redox potential for bays 3, 4 and 7 at the three transverse sections mentioned above. Temporal plots of redox potential for all seven bays however, can be found in Figure A.3 Appendix A. According to Figure 5.7, the redox potential for each bay showed a temporal pattern and increased from the beginning of the trials with values between 0 and -100 mV. The parameter attained a maximum value of 200-300 mV on the bays between weeks 6 and 8 (i.e. June 16 -



(a) Bay 3



(b) Bay 4



(c) Bay 7

Figure 5.7 Temporal Plots of Redox Potential for Representative Bays

July 3) of the monitoring program. After this period, the redox potential decreased gradually to values between 0 and 100 mV at the end of the grass filtration trial. Figure 5.7 does not clearly suggest a consistent spatial trend for redox potential within bays, since it changes differently at 100 m, 200 m and 300 m as the season progresses. Redox potential patterns similar to those of the three representative bays (shown in Figure 5.7) can also be seen in Figure A.4 Appendix A for all seven bays. The average redox potential over each bay is plotted in Figure 5.8. This figure does not indicate a clear relationship between redox potential and hydraulic loading rate. However, Figure 5.8 shows that apart from the first measurement in the redox potential survey during week 3 (i.e. 26/5/97) of the grass filtration trials, the average redox potentials over the representative bays were higher than 200 mV during Period 1 of the monitoring program. On the other hand, the average redox potentials over the representative bays during Period 2 were generally lower than 150 mV. As indicated in Section 2.5.1.4, liquids with redox potential values of 400 mV or greater are said to be less corrosive and have better oxidation capacity. However, liquids with values of 100 mV and less are supposed to be severely corrosive and have very little oxidation capacity. On this basis, the average redox potential values of the representative bays for Periods 1 and 2 indicate that the dissolved oxygen on the bays was more effective for oxidation of wastes during Period 1 than during Period 2.

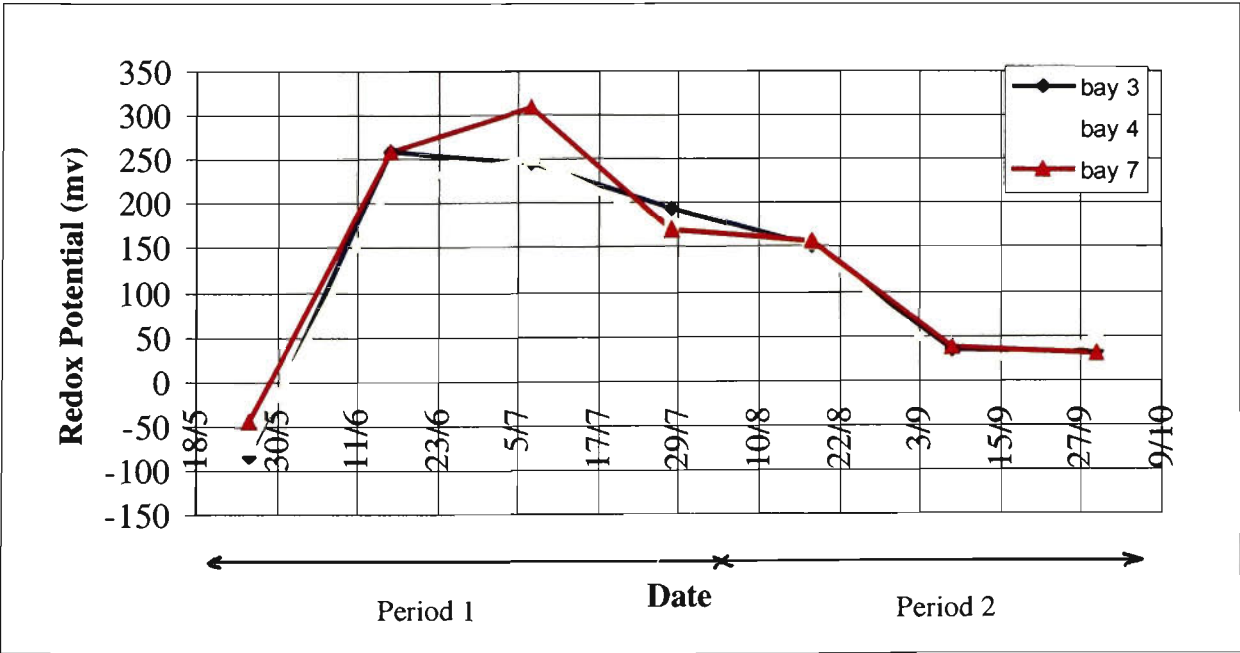


Figure 5.8 Average Redox Potential Over Representative Bays

5.4.5 Total Suspended Solids (TSS)

Total suspended solids (TSS) of the wastewater was measured at the Water Ecoscience laboratories from the field samples of the common inlet, 200m, 300m and outlet point of all seven bays obtained once a week. Figure 5.9 shows the temporal plot of TSS for the representative bays of 3, 4 and 7. This figure shows the influent TSS concentrations at the common inlet and outlet of representative bays. However, Figure A.5 of Appendix A shows more details of TSS concentrations. It shows the influent concentration and concentrations at 200 m, 300 m and the outlet of each of the 7 bays. As can be seen from Figure 5.9, the inlet TSS values stayed fairly low throughout the trial period with an average of 60 mg/l, apart from the first two weeks of sampling when TSS was often above 80 mg/l. The plot shows the variability of the influent TSS concentrations. The outlet TSS concentrations prior to August 18 were generally not higher than 10 mg/l for all three representative bays, but the values increased for some bays during the remaining trial period. Higher concentrations towards the end of the trials may be due to the accumulation of detritus materials from the start of the sampling season.

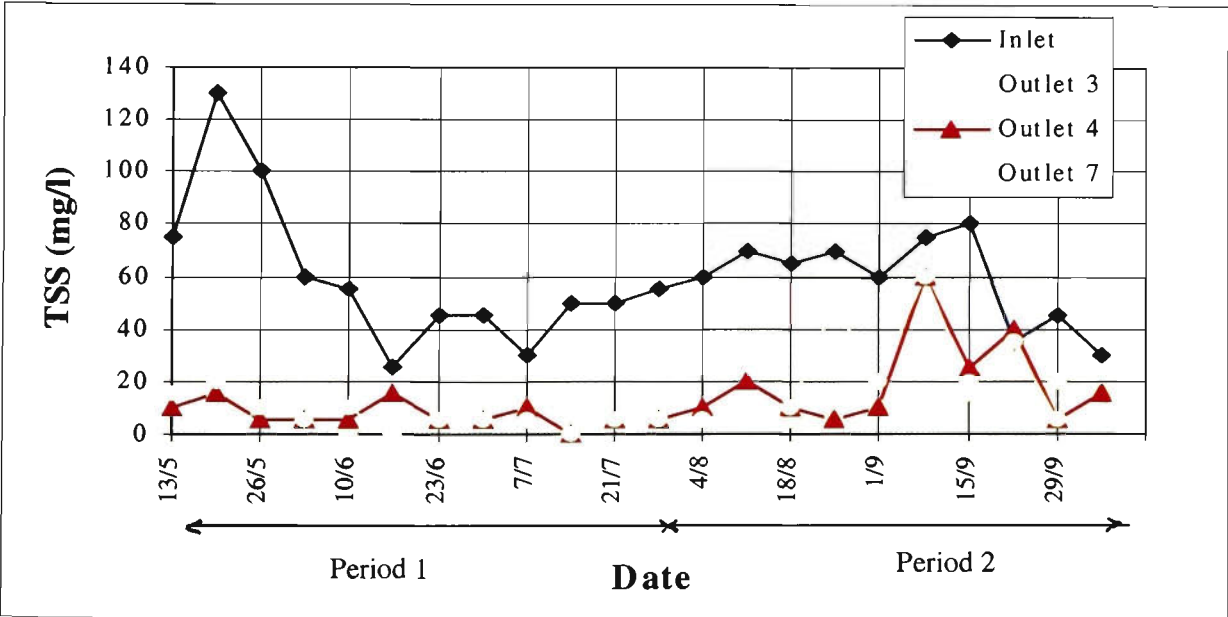


Figure 5.9 TSS Temporal Plot at Common Inlet and Outlet of Representative Bays

According to Figure 5.9, the TSS concentrations at the outlets of bays 3, 4 and 7 are quite similar during Period 1 and some portions of Period 2. This suggests that, hydraulic loading rate does not appear to contribute significantly to the performance

of the bays in reducing TSS. The average inlet TSS concentrations for Periods 1 and 2 were 60 and 59 mg/l respectively, while the average outlet TSS concentrations for all three representative bays were 8 and 13 mg/l for Periods 1 and 2 respectively.

The average outlet TSS concentration for all three representative bays for the entire trial period was 10 mg/l. Based on average inlet and outlet concentrations, the grass filtration system achieved a reduction of about 83% in TSS.

According to Figure A.5 of Appendix A, the TSS concentrations at the inlet and 200 m, 300 m and outlet points of all seven bays show a consistent pattern, with the concentrations gradually decreasing as the wastewater travels through the bays. This behaviour of the bays is normal as it is logical that the vegetation would reduce the suspended solids by acting as a filtering media while the wastewater is flowing down the bays. However, towards the end of the trials (i.e. towards the end of Period 2), the TSS concentrations at the 200 and 300 m marks are higher than the effluent TSS concentrations. This is due to the accumulation of the suspended solids which cause a build up of suspended solids within the bays from the commencement of the season.

5.4.6 Colour

Wastewater colour was measured in terms of apparent colour at the Water Ecoscience laboratories from the field samples of the common inlet and the outlets of all seven bays obtained twice per week. The temporal plots of colour at the common inlet and the outlets of representative bays are shown in Figure 5.10. However, the temporal plots for the remaining bays are shown in Figure A.6 of Appendix A, together with those of the representative bays.

Colour at the common inlet remained fairly constant at 300 Pt-Co units during the first eight weeks of the trial and improved with time to a minimum of 150 Pt-Co units in Week 17, after which it increased to 250 Pt-Co units during the rest of the trials. The average value of colour at the inlet was 256 Pt-Co units during the entire duration of the trial. However, the average colour at the inlet during Period 1 and 2 were 291

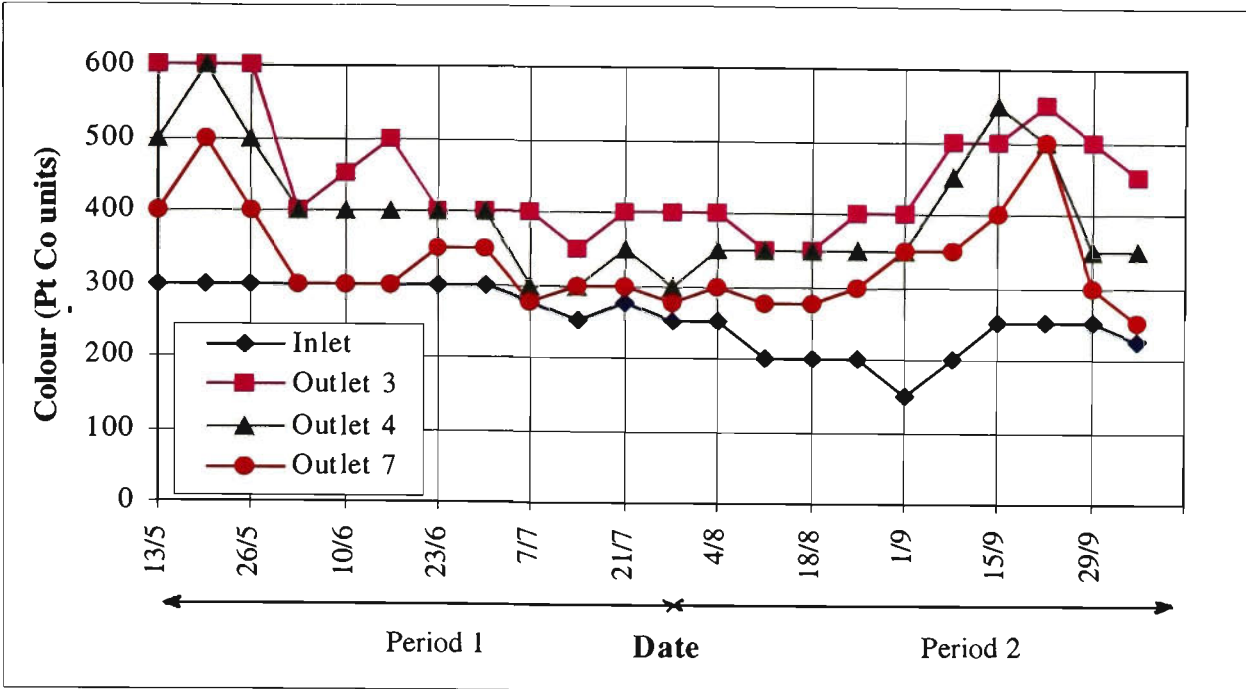


Figure 5.10 Colour Temporal Plot for Representative Bays

and 220 Pt-Co units respectively. On the other hand, the average colour measurements over the representative bays at the outlets were 407 and 383 Pt-Co units for Periods 1 and 2 respectively. Based on these averages, during Period 1 there was an increase of 28% in the effluent colour value, while during Period 2 there was an increase of 42% in the effluent colour. The higher colour at the outlet of each bay indicates that the wastewater picked up colour on the bays from organic debris such as decaying leaves and other plant tissue.

Figure 5.10 also shows that the colour of effluent was relatively higher for some bays during the first three weeks and the last five weeks of the trial than that observed during the remaining period. The same observation can be made for the remaining bays in Figure A.6 of Appendix A. Generally, effluent colour was higher than the influent colour for the WTP trial bays over the whole trial period. Therefore, if colour improvement of wastewater is required by Melbourne Water at a tertiary treatment level, then an alternative method to grass filtration shall be sought.

There is a general increase in colour deterioration when hydraulic loading rate decreases. This may be attributed to longer contact (detention) time of wastewater with decaying organic material on the bays with lower hydraulic loading rate, allowing more colour producing acids and other substances to be picked up by the wastewater.

As listed in Table 4.7, bays 3, 4 and 7 had detention times of 4.08, 4.21 and 3.07 days respectively, whereas their hydraulic loading rates were 30, 40 and 50 mm/d respectively.

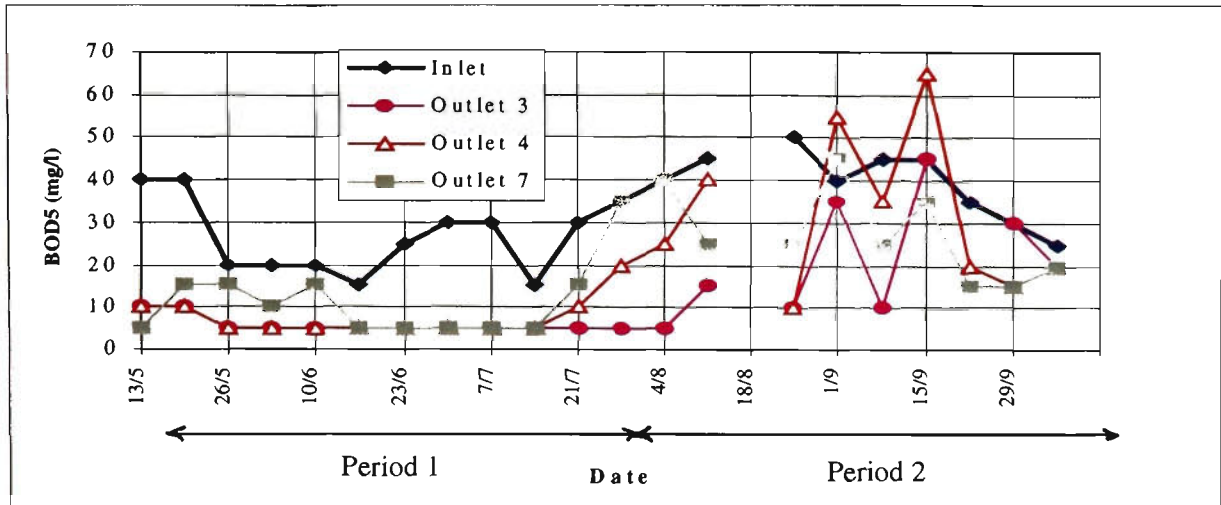
5.5 CHEMICAL PARAMETERS

Several chemical parameters were monitored during this trial, including BOD₅, CBOD₅ and COD. This section presents the data obtained from monitoring and analysis of these parameters, together with some discussion.

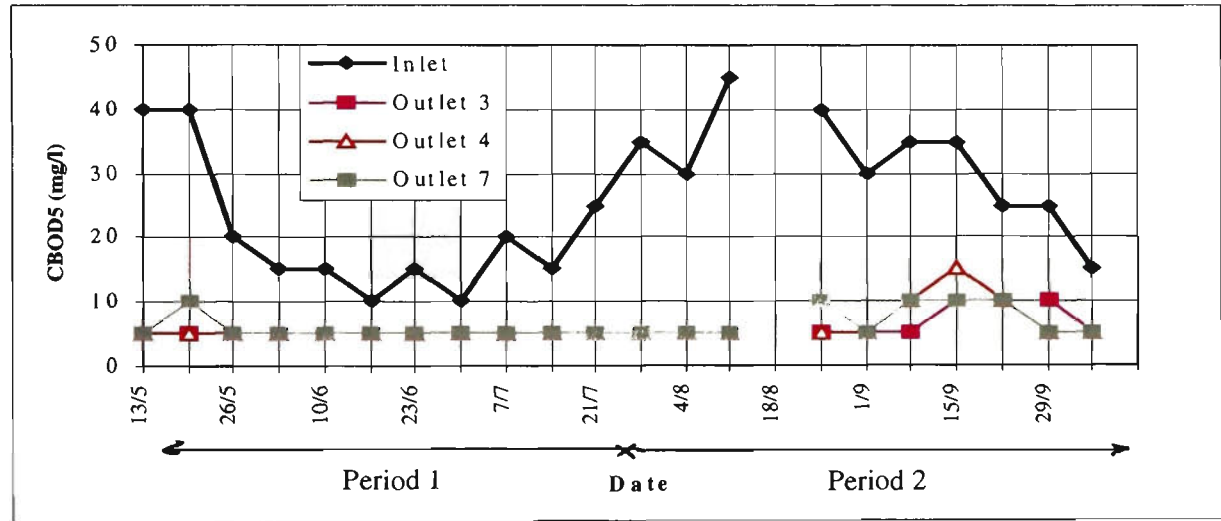
5.5.1 BOD₅ and CBOD₅

There are two main forms of biochemical oxygen demand (BOD), namely carbonaceous and nitrogenous BOD. The parameters monitored under this group include 5-day biochemical oxygen demand (BOD₅), 5-day carbonaceous BOD (CBOD₅) and filterable CBOD₅. All three parameters, (i.e. BOD₅, CBOD₅ and filterable CBOD₅) were measured once a week at the common inlet and 200 m, 300 m and outlet of all seven bays for the whole duration of the trials. The temporal plots of these parameters at the common inlet and at the outlets of representative bays are shown in Figure 5.11. The plots for the remaining bays (including those of the representative bays) are shown in Figures A.7 to A.9 of Appendix A. Figures A.7 to A.9 show the influent concentrations at the common inlet and the effluent concentrations at 200 m, 300 m and outlet of all seven bays. In Figure 5.11, a discontinuity can be observed on 18 August, on the inlet and outlet data plotted. The laboratory analysis did not produce any results on this day.

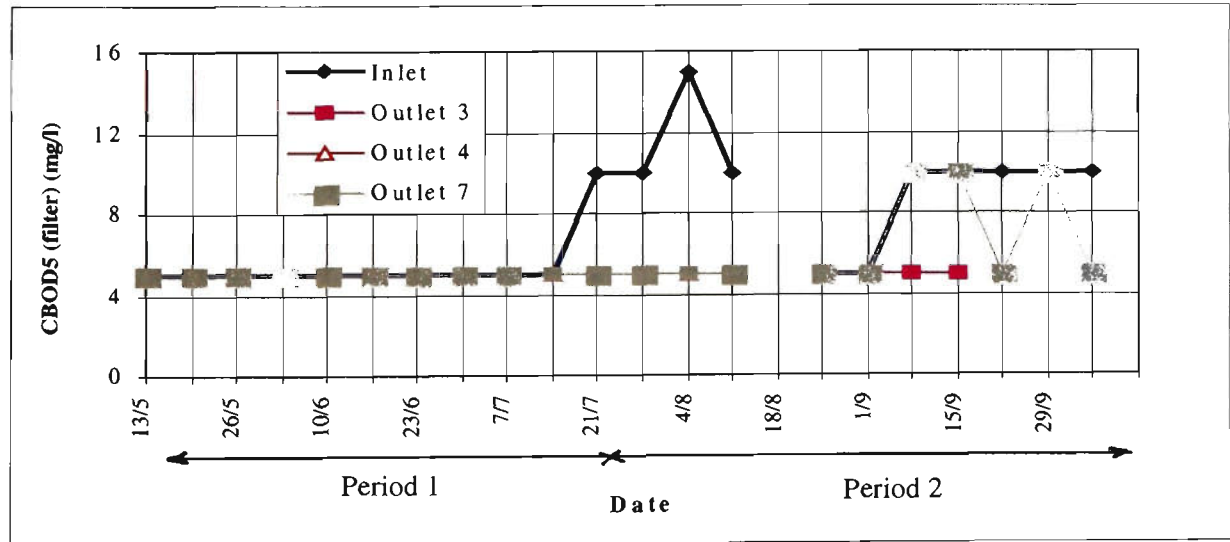
Concentration of influent BOD₅ ranged from 15 to 50 mg/l with an average of 30 mg/l, over the whole sampling period. On the other hand, CBOD₅ at the inlet had an average of 25 mg/l. This suggested that most of the oxygen demand was by carbonaceous organic matter. However, filterable CBOD₅ values never exceeded 15 mg/l with an average of 7 mg/l during the entire monitoring period. This shows that the influent contained a large proportion of the degradable carbonaceous organic material which was in the dissolved form. Generally, the concentrations of all forms



(a) BOD₅



(b) CBOD₅



(c) CBOD₅ (filt)

Figure 5.11 Temporal Plots of BOD₅, CBD₅ and CBOD₅ (Filt) at Common Inlet and Outlets of Representative Bays

of influent BOD were lower during Period 1 of the monitoring program. Although the temperature has minimal effect on the BOD and its various forms (Kadlec and Knight, 1996), it is worthwhile noting the fact that the average temperature during Period 1 was slightly cooler than during Period 2 at the common inlet. The average temperatures were 10 and 12°C at the common inlet during periods 1 and 2 respectively. Kadlec and Knight (1996) noticed that the BOD and COD removals are rarely affected by temperature in Free Water Surface (FWS) wetlands.

The concentration of BOD₅ at the bay outlets shows a marginal inverse relationship with the hydraulic loading rate, although CBOD₅ and filterable CBOD₅ concentrations do not suggest any significant relationship with the loading rate. For bays 3, 4 and 7, the average outlet BOD₅ concentrations were 10.9, 17.0, and 17.3 mg/l respectively, while both CBOD₅ and filterable CBOD₅ concentrations were not greater than 5.0 mg/l for all bays over 80% of the monitoring period. The average BOD₅ removal was between 42 and 63% across the representative bays over the whole period of trials. The average BOD₅ across representative bays at the inlet during Periods 1 and 2 was 26 and 35 mg/l respectively, while the average outlet concentrations were 7 and 23 mg/l. On the basis of these averages, BOD₅ reductions of 73% and 35% were achieved during Periods 1 and 2 of the trials respectively.

On the other hand, average CBOD₅ reductions of 75% and 79% were achieved during Periods 1 and 2 respectively across representative bays, while filterable CBOD₅ reductions of 0% and 45% were achieved during Periods 1 and 2 respectively. Although BOD₅ and CBOD₅ show higher reductions during Period 1 compared to Period 2, this is not the case for the filterable CBOD₅. This may be due to the higher influent concentrations of filterable CBOD₅ during Period 2 (i.e. see Figure 5.11).

As also mentioned earlier, Figures A.7 to A.9 Appendix A show the BOD₅, CBOD₅ and CBOD₅(Filter) temporal plots at the inlet and 200 m, 300 m and the outlets of all seven trial bays. According to these plots, the spatial variability between the inlet and the transverse sections (i.e. 200 and 300 m marks) and the outlets are such that the influent concentrations are consistently higher than the 200 and 300 mark concentrations and the effluent concentrations. This trend can be seen throughout all seven bays.

5.5.2 COD

Chemical oxygen demand (COD) is one of the chemical parameters which was measured once a week for the whole duration of the trials, at the inlet and 200 m, 300 m and outlet of all seven bays. A temporal plot of COD for inlet and outlet of representative bays is shown in Figure 5.12. However, the plots of COD for all seven bays showing concentrations at the inlet, 200 m, 300 m and the outlet of the bays are shown in Figures A.10 of Appendix A.

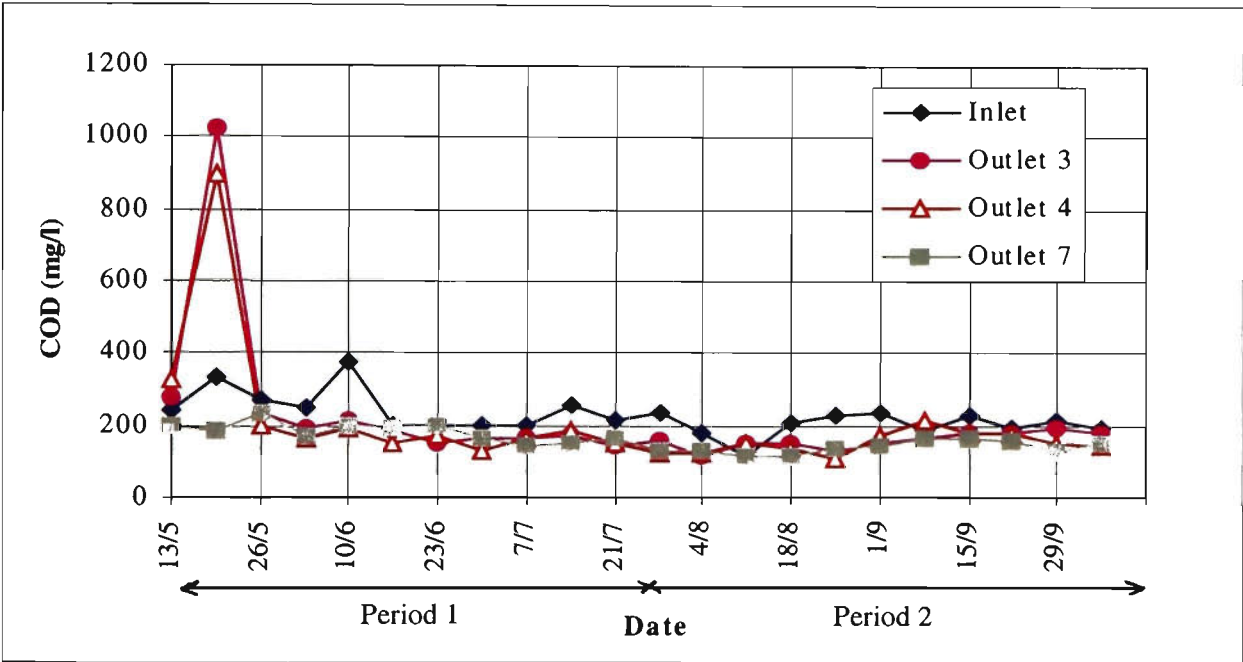


Figure 5.12 Temporal Plot of COD for Common Inlet and Outlets of Representative Bays

COD showed some variability at the common inlet and did not indicate any trend or significant difference during any period of the trial. At the inlet, COD was between 190 and 300 mg/l for 77% of the time with an average of 223 mg/l during the entire trial period. Figure 5.12 shows a COD reading in excess of 800 mg/l for outlets of bays 3 and 4. The reason for this high COD level is not known. The average COD concentration at the common inlet was 223 mg/l, while at the outlets of bays 3, 4, and 7 these were 207, 192 and 156 mg/l respectively for the whole trial period. This shows average removals of 7, 13 and 30% for bays 3, 4 and 7 respectively. Based on these removal rates, it appears that the hydraulic loading rate of the bays has an effect on the COD removals since the higher loaded bays have higher removal rates, in spite of

their lower detention times. However, as also mentioned above, bays 3 and 4 had COD concentrations in excess of 800mg/l at one occasion during Period 1 as shown in Figure 5.12. This can significantly affect the average effluent COD concentrations of bays 3 and 4 and hence their COD removal efficiency. Therefore, to examine the effect of these high concentrations on the overall removal efficiency of COD from bays 3 and 4, they were excluded from the data and the average effluent COD concentrations of bays 3 and 4 were re-calculated. This resulted in averages of 170 and 165 mg/l for effluent COD concentrations of bays 3 and 4 respectively. Based on these averages and the influent COD average over the whole period of 222 mg/l, as stated above, COD removals of 24 and 26% were obtained for bays 3 and 4 respectively. Therefore, on the basis of these averages, it can be said that the effect of hydraulic loading rate on COD removal from the grass bays is minimal.

The average COD influent concentrations for Periods 1 and 2 were 249 and 198 mg/l. The average effluent COD concentrations across the three representative bays were 226 and 147 mg/l for Periods 1 and 2 respectively. Based on these averages, average removals of 9% and 26% COD were achieved during Periods 1 and 2. This shows that the bays reduced much higher COD during Period 2 of the trials as opposed to Period 1, indicating higher oxygen demand during this period (see Figure 5.12).

According to Figure A.10 of Appendix A, the spatial variability of COD within the bays (i.e. between the inlet, 200 m, 300 m and the outlets) is quite similar to the other chemical parameters. The influent COD concentrations are consistently higher than the 200 m and 300 m and effluent COD concentrations, except for few occasions when the 200 m and the 300 m mark concentrations have exceeded those of the influent COD concentrations.

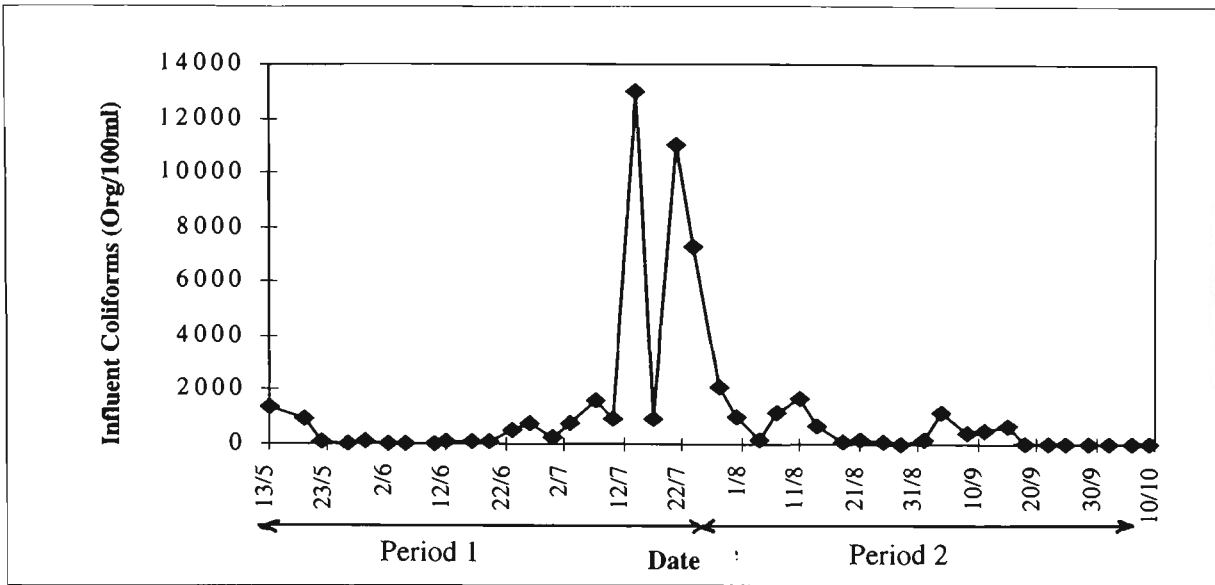
5.6 MICROBIOLOGICAL PARAMETERS

The only microbiological parameter that was monitored during the trials was faecal coliforms.

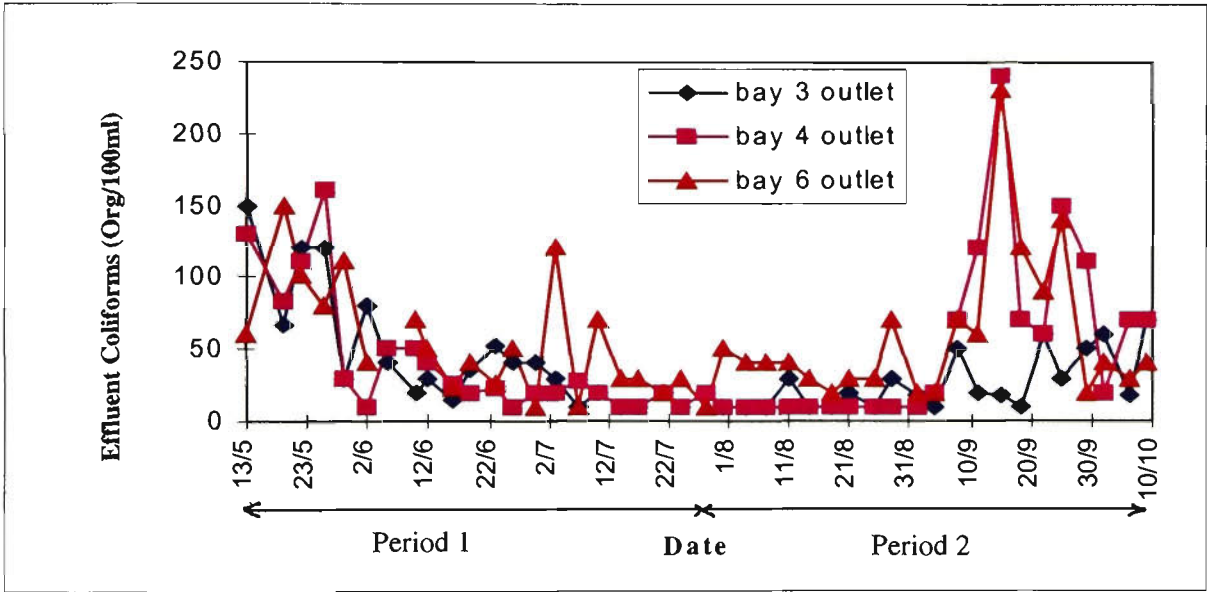
5.6.1 Faecal Coliforms

Faecal coliform concentrations were measured twice per week at the common inlet and at the outlet of bays 3, 4 and 6 only. The influent and effluent coliform concentrations are shown in Figure 5.13. As can be seen from this figure, the coliform concentrations at the common inlet did not show any well-defined trend, but had values that were higher than 1000 Org/100ml between July 7 and August 11. The parameter had a large variability with minimum and maximum concentrations of <10 and 13,000 Org/100ml respectively. The reason for this particular variability is not known, although this would be due to the effectiveness of the secondary treatment system (i.e. Western Lagoon). This effectiveness could vary according to factors such as detention time of the lagoon, sunshine etc.

Measurements of coliforms at the inlet showed an average of 1166 Org/100ml throughout the whole period, while the average outlet concentrations for the 3 bays (i.e. bays 3, 4 and 6) was 46 Org/100ml. This showed an overall reduction of 96%. Figure 5.13 also shows that more coliform reduction had taken place during Period 1 than Period 2. Analysis of coliform results showed that the average inlet concentrations of coliforms during Periods 1 and 2 were 1898 and 468 Org/100ml respectively, while the average of effluent concentrations of the three representative bays (i.e. bays 3, 4 and 6) during Periods 1 and 2 were 48 and 44 Org/100ml. On the basis of these averages, it was estimated that 97% and 90% of the influent coliforms were removed on the bays during Periods 1 and 2 respectively. Although the average influent coliform was much higher in Period 1 than Period 2, more coliform reductions were achieved during Period 1.



(a) Influent Coliform Concentrations



(b) Effluent Coliform Concentrations

Figure 5.13 Temporal Plots of Influent and Effluent Coliform Concentrations

Figure 5.13 indicates that coliform concentrations at the outlets of all monitored bays were always less than 250 Org/100ml, even during periods when input concentrations as high as 13000 Org/100ml were observed. Furthermore, it was observed that the effluent coliform concentrations were less than 50 Org/100ml throughout most of the trial period (Figure 5.13). This showed that significant reductions of bacteria had occurred through sedimentation within the trial bays and this may be due to sedimentations. The reduction of bacteria through sedimentation is supported by the work of Shimohara et al. (1985), who stated that viruses and bacteria can attach to solids, and subsequent sedimentation may remove a significant percentage of the viruses and bacteria.

5.7 NUTRIENTS

The major nutrients considered for analyses in this project were nitrogen and phosphorus. These two nutrients were particularly highlighted for attention in this study since they are the most critical pollutants, whose excessive concentrations cause eutrophication of Port Phillip Bay. One of the objectives of this trial was to investigate the efficiency of the grass filtration bays in removing nutrients, in particular nitrogen.

Nitrogen is present in wastewater in the form of nitrite ($\text{NO}_2\text{-N}$), nitrate ($\text{NO}_3\text{-N}$), ammonia ($\text{NH}_3\text{-N}$) and organic nitrogen, and is usually in the form of $\text{NH}_3\text{-N}$ or organic nitrogen in raw sewage. Nitrogen entering a grass filtration treatment system is assimilated by plants and micro-organisms, released into the atmosphere as ammonia or nitrogen gas, or filtered out by plants or soil media.

Phosphorus is present in raw wastewater in various organic and inorganic forms, both in solution and in suspension. The soluble forms include inorganic orthophosphates, condensed phosphates and some organic orthophosphates. Excessive quantities of phosphate come mainly from the large amounts of polyphosphates that are added to domestic cleaning compounds (i.e. detergents) and runoff from gardens which employ superphosphate fertilisers (Winkler, 1981).

All nitrogen and phosphorus concentrations, and their mass balance analyses are presented in the following sections.

5.7.1 Nitrogen

As mentioned previously, nitrogen in the wastewaters is present in various forms. For the purpose of this pilot study, the forms of nitrogen measured were $\text{NO}_2\text{-N}$, $\text{NO}_3\text{-N}$, $\text{NH}_3\text{-N}$ and total kjeldahl nitrogen (TKN). Hence, the oxidised nitrogen ($\text{NO}_x\text{-N}$) and total nitrogen (TN) were computed. TKN is the sum of $\text{NH}_3\text{-N}$ and organic nitrogen, while $\text{NO}_x\text{-N}$ is the sum of $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$. Hence, TN is the sum of TKN and $\text{NO}_x\text{-N}$, which effectively takes into account all forms listed above. Since TKN implicitly takes organic nitrogen into account, organic nitrogen is not separately

discussed in the analysis of the results of this trial study. The analysis of the results presented in this section is mainly based on TN and $\text{NH}_3\text{-N}$. However, to be able to evaluate whether ammonification (a process by which nitrogen is transformed into ammonium form by hydrolysis of urea and bacterial decomposition of proteins and nucleic acids) occurred within the bays, discussion on all forms of nitrogen measured for this study is also provided.

5.7.1.1 Nitrogen Concentration

As mentioned above, various forms of nitrogen were measured in this project. These are discussed separately in the following sections. However, from the computed forms of nitrogen (i.e. TN and $\text{NO}_x\text{-N}$), only TN is discussed separately since it is the most common form of nitrogen and also one the concentrations which is required by the EPA discharge licence.

5.7.1.1(a) Nitrite Nitrogen ($\text{NO}_2\text{-N}$)

The average $\text{NO}_2\text{-N}$ concentrations of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.3. According to this table, the influent wastewater contained very low levels of $\text{NO}_2\text{-N}$, which was mainly less than 0.2 mg/l on average. However, the concentration levels increased slightly at the 200m mark and decreased as wastewater moved downstream. This increase mainly occurred during Period 1. During this period, the $\text{NO}_2\text{-N}$ concentrations increased by up to 150% (i.e. at bay 4) between the inlet and the 200m mark. An increase of 100% in the $\text{NO}_2\text{-N}$ concentrations between the inlet and the outlet of bays 4 and 7 can also be seen. However, during Period 2, the concentration levels remained consistent throughout the bay. This clearly indicates that during Period 1, most of the influent nitrogen was nitrified and hence some $\text{NO}_2\text{-N}$ was produced within the bays. However, no signs of nitrification can be seen during Period 2.

Table 5.3 Average NO₂-N Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2
Inlet	0.20	0.10	0.15	0.20	0.10	0.15	0.20	0.10	0.10
200m	0.40	0.10	0.25	0.55	0.10	0.32	0.40	0.10	0.20
300m	0.25	0.15	0.20	0.30	0.10	0.20	0.30	0.10	0.20
Outlet	0.10	0.10	0.10	0.25	0.15	0.20	0.30	0.10	0.20

Figure 5.14 shows temporal plots of NO₂-N concentrations at inlet and outlet of the representative bays. According to this figure, the peak NO₂-N concentrations at outlets occurred during Period 1, in particular within the first 3 weeks of Period 1. Within the first 3 weeks, the effluent NO₂-N concentrations were higher than the influent concentrations, which suggests that the ammonia entering the bays was being nitrified and converted to NO₂-N.

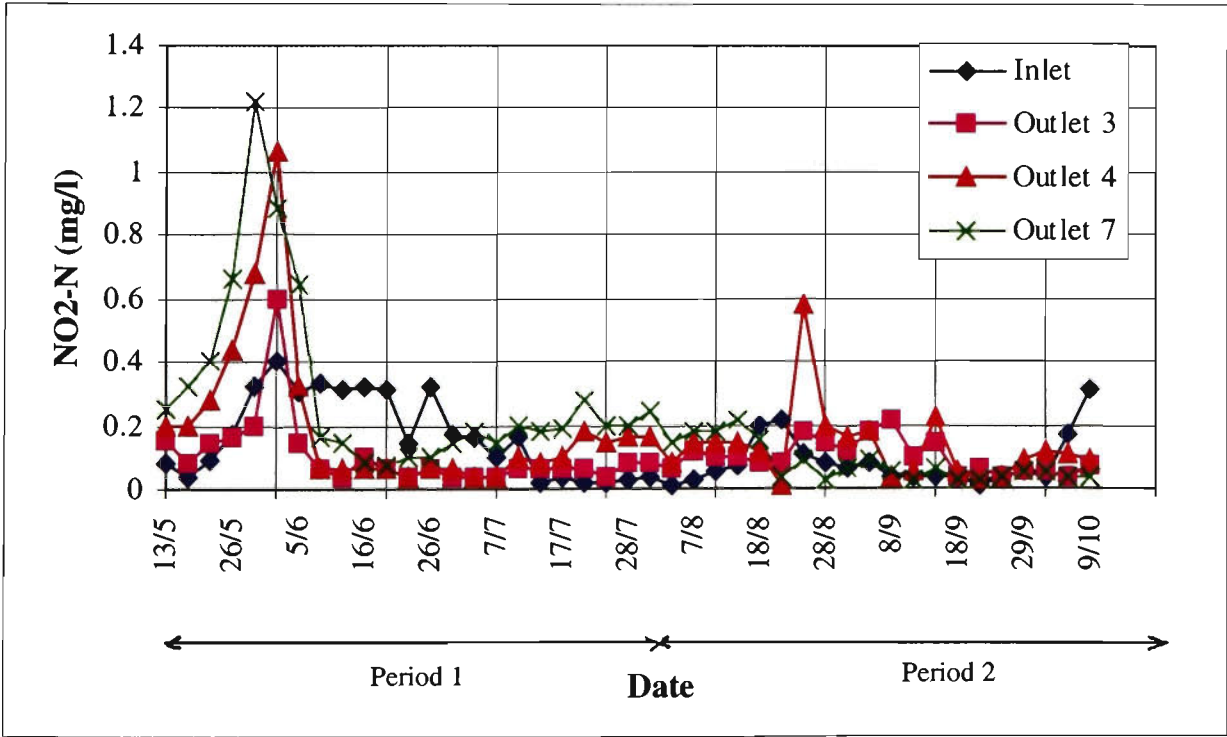


Figure 5.14 NO₂-N Temporal Plots for Representative Bays

During Period 2, however, the concentrations were variable throughout with influent NO₂-N concentrations being lower than the effluent NO₂-N concentrations for most of the time except for a couple of occasions, the reason for which is not known. Also, the

effluent $\text{NO}_2\text{-N}$ concentrations of the representative bays do not follow a particular temporal pattern and do not seem to have any relationship to hydraulic loading rate. Therefore, it is difficult to comment on the effect of hydraulic loading rates of the representative bays on this parameter.

Temporal plots of $\text{NO}_2\text{-N}$ at the inlet, 200m, 300m and the outlets of all seven bays are shown in Figure A.11 of Appendix A. According to this figure, the $\text{NO}_2\text{-N}$ concentrations are generally highest within the bays (i.e. at the 200 and 300m marks) especially at the 200m mark of all seven bays. This suggests that nitrification processes took place within the bays, especially within the first 200 meters of the bays.

5.7.1.1(b) Nitrate Nitrogen ($\text{NO}_3\text{-N}$)

The average $\text{NO}_3\text{-N}$ concentrations of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.4. The $\text{NO}_3\text{-N}$ concentrations have increased within the bays with the peak values occurring at the 300m mark except for bay 3 which has peaked at the 200m mark. The difference in concentrations of $\text{NO}_3\text{-N}$ between Periods 1 and 2 are significant, with Period 1 having higher concentrations and higher increase in concentrations within the bays than Period 2. Similar to $\text{NO}_2\text{-N}$ concentrations, these results suggest that more nitrification has taken place during Period 1 than Period 2. During the whole period, the $\text{NO}_3\text{-N}$ concentrations have increased by 300, 900 and 900% between the inlets and the outlets of bays 3, 4 and 7 respectively.

Figure 5.15 shows temporal plots of $\text{NO}_3\text{-N}$ concentrations at the inlets and the outlets of the representative bays. According to this figure, the influent $\text{NO}_3\text{-N}$ concentrations have been lower than the effluent $\text{NO}_3\text{-N}$ for the entire duration of the trials. During Period 1, the effluent $\text{NO}_3\text{-N}$ concentrations have been the lowest during the first 3 weeks. This suggests that the influent ammonia nitrified (i.e. converted to $\text{NO}_2\text{-N}$ as mentioned in Section 5.7.1.1(a)) and was denitrified or converted to nitrogen gas.

Table 5.4 Average NO₃-N Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2
Inlet	0.25	0.15	0.20	0.25	0.15	0.20	0.25	0.15	0.20
200m	4.00	1.00	2.50	4.20	0.70	2.45	2.90	0.20	1.55
300m	2.10	1.00	1.55	4.70	0.70	2.70	3.70	0.60	2.15
Outlet	0.90	0.70	0.80	3.1	1.0	2.05	3.45	0.65	2.05

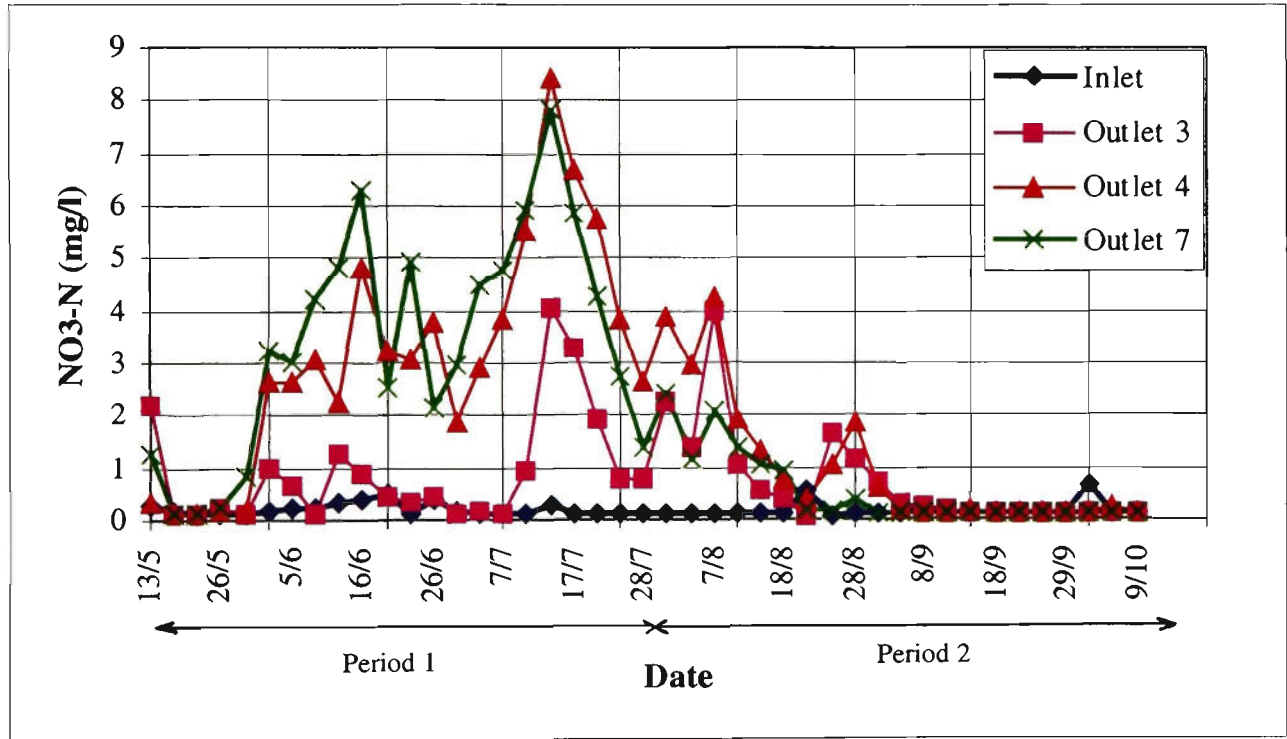


Figure 5.15 NO₃-N Temporal Plots for Representative Bays

However, during the remainder of Period 1, the effluent NO₃-N concentrations kept increasing until they reached a peak, and then gradually decreased during Period 2 to levels almost the same as the influent concentrations (and even lower than the influent concentrations at one point). This behaviour of the effluent NO₃-N concentrations suggests that during Period 2 of the trial, lower amounts of ammonia were denitrified as the influent and effluent ammonia concentrations remain higher (i.e. Figure 5.17) than the NO₃-N concentrations. Figure 5.15 also shows that the bays with the higher hydraulic loading rates (i.e. bays 4 and 7) have higher concentrations for most of the trial duration than the bay with lowest hydraulic loading rate (i.e. bay 3). This suggests

that the hydraulic loading rate and the NO₃-N removal from the bays are related to some extent.

Temporal plots of NO₃-N at the inlet, 200m, 300m and the outlets of all seven bays are also shown in Figure A.12 of Appendix A. According to this figure, the NO₃-N concentrations are the highest within the 200 and 300m marks, especially at the 200m mark of all seven bays. This suggests that nitrification processes took place within the bays, especially within the first 200 meters of the bays.

5.7.1.1(c) **Oxidised Nitrogen (NO_x-N)**

The average NO_x-N concentrations (which is the sum of NO₂-N and NO₃-N) of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.5. This table shows that the wastewater NO_x-N concentrations have increased between the inlet and the 200m and 300m marks, and then decreased at the outlets. Apart from bay 3, the NO_x-N concentrations of the other two bays have peaked at the 300m mark during both periods. The reason for this is not known.

Table 5.5 Average NO_x-N Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period	Period	Period s	Period	Period	Periods	Period	Period	Periods
	1	2	1 & 2	1	2	1 & 2	1	2	1 & 2
Inlet	0.40	0.20	0.30	0.40	0.20	0.30	0.40	0.20	0.30
200m	4.40	1.20	2.80	4.70	0.80	2.75	3.30	0.20	1.75
300m	2.40	1.10	1.75	5.00	0.80	2.90	4.10	0.70	2.40
Outlet	1.00	0.80	0.90	3.40	1.20	2.30	3.80	0.70	2.25

According to Table 5.5, once again the difference between the NO_x-N concentrations of the bays during Periods 1 and 2 are quite high, with the concentration readings being generally lower during Period 2. While the difference in the influent NO_x-N concentrations during Periods 1 and 2 is 50% (i.e. Period 2 influent concentrations are 50% lower than Period 1), the corresponding difference in the effluent concentrations are 20, 66 and 82% for bays 3, 4 and 7 respectively. Since NO_x-N is the sum of NO₂-

N and $\text{NO}_3\text{-N}$, the temporal plot of $\text{NO}_x\text{-N}$ is very similar to $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$ temporal patterns. This indicates that higher amounts of $\text{NO}_x\text{-N}$ are being generated within bays 4 and 7 during Period 1. Therefore, the results suggest that most of the nitrification processes occurred within Period 1 of the study.

Figure 5.16 shows temporal plots of $\text{NO}_x\text{-N}$ concentrations at the inlets and outlets of each of the representative bays. This figure shows that during Period 1, the effluent concentrations of $\text{NO}_x\text{-N}$ were higher for the higher hydraulic loaded bays (average of 1.0, 3.5, and 3.8 mg/l for bays 3, 4 and 7 respectively), implying that nitrification increased with the hydraulic loading rate during the first half of the trials. However, the effluent $\text{NO}_x\text{-N}$ concentrations reduced significantly to values under 2.0 mg/l during most of Period 2 for each of the representative bays (average of 0.8, 1.2 and 0.7 mg/l for bays 3, 4 and 7 respectively). These lower effluent $\text{NO}_x\text{-N}$ concentration values suggest that nitrification during Period 2 was lower than during Period 1, probably due to the lower DO levels in Period 2 (i.e. Figure 5.11).

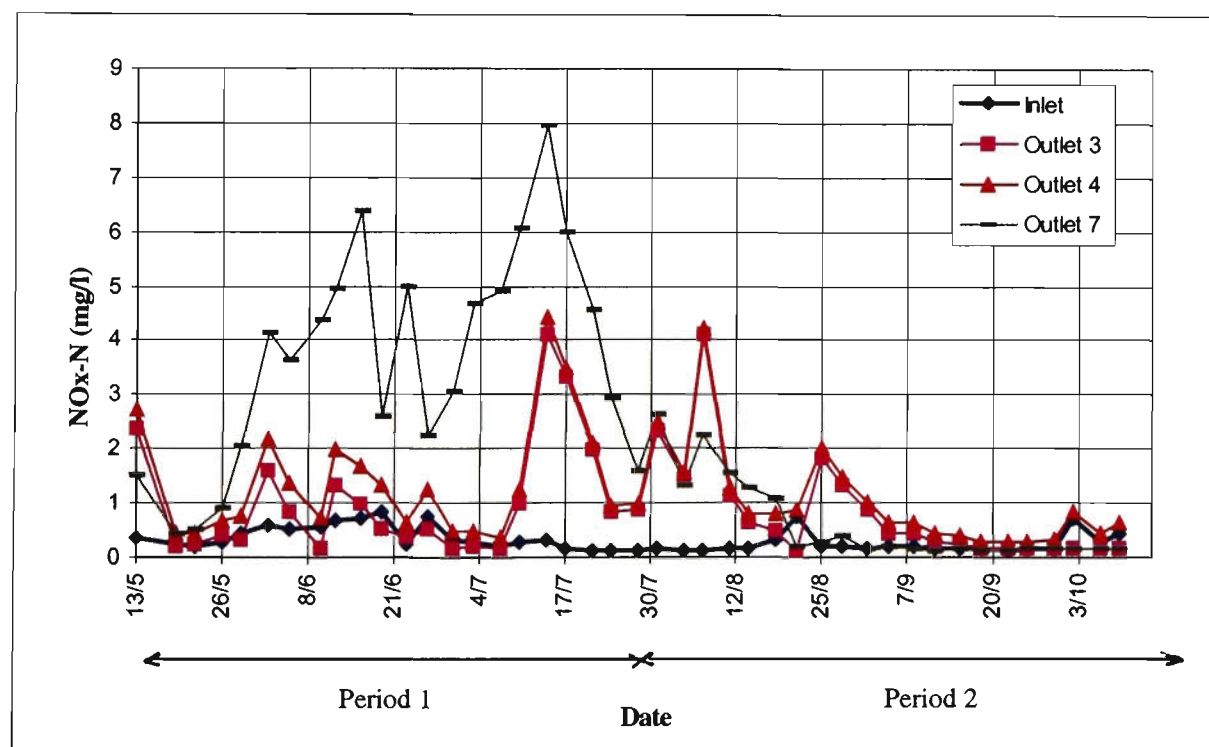


Figure 5.16 $\text{NO}_x\text{-N}$ Temporal Plots for Representative Bays

Temporal plots of $\text{NO}_x\text{-N}$ at the inlet, 200m, 300m and the outlets of all seven bays are also shown in Figure A.13 of Appendix A. $\text{NO}_x\text{-N}$ temporal patterns also show highest values at the 200 and 300m marks with the maximum concentrations at the 200 m mark.

5.7.1.1(d) Ammonia Nitrogen (NH₃-N)

The average NH₃-N concentrations of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.6. This table shows that there is a significant difference between the Period 1 and Period 2 results, with Period 1 having lower influent concentrations and hence lower concentrations at the 200m and 300m marks and the outlets of all three bays. It is believed that the higher NH₃-N concentrations during Period 2 at the 200m, 300m and the outlets of all three representative bays are due to the lower nutrient uptake capacity of the grass during the later stages of the trial and the higher influent NH₃-N concentrations. The higher influent concentrations during Period 2 can also be observed on the temporal plots of NH₃-N shown as Figure 5.17.

Table 5.6 Average NH₃-N Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2
Inlet	20.7	29.9	25.4	20.7	29.9	25.4	20.7	29.9	25.4
200m	4.6	25.4	15.3	8.7	29.1	19.1	10.1	30.9	20.7
300m	2.2	17.1	9.8	4.8	25.5	15.4	6.3	27.2	17.0
Outlet	1.6	13.2	7.5	2.9	20.4	11.8	4.7	25.3	15.3

Although influent NH₃-N concentrations increased by 44% between Period 1 Period 2, the corresponding increase in the effluent concentrations was much higher (i.e. 725%, 603% and 438% for bays 3, 4 and 7 respectively). This shows that generally the bays were more efficient in nitrogen removal during Period 1 compared to Period 2.

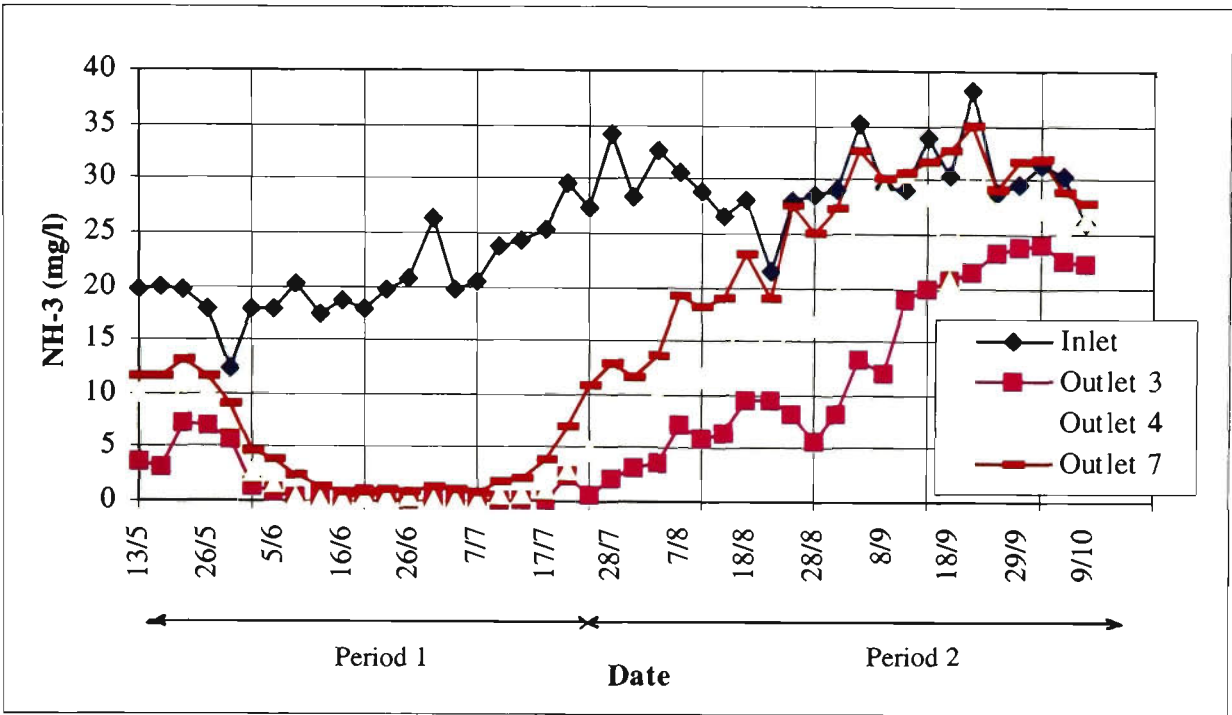


Figure 5.17 NH₃-N Temporal Plot for Representative Bays

Based on the NH₃-N concentration results shown in Table 5.6, most of the NH₃-N reductions in Period 1 had taken place within the first 200m of the bays. Bays 3, 4 and 7 showed reductions of 77, 58 and 51% respectively within the first 200m during Period 1.

This once again re-emphasises the significant difference in the efficiencies of the bays in reduction of NH₃-N in two periods. Apart from the fact that the influent concentrations were lower during Period 1, the other factor which significantly contributes to this issue is the nitrogen uptake capacity of the Ryegrass. Since the growth rate of Ryegrass is lower in Period 2 compared to Period 1, this analysis indicates that the nitrogen uptake capacity of the Ryegrass has reduced with its growth. The same bays showed NH₃-N reductions of 15, 3 and -3% within the first 200m during Period 2.

In general, the results and analysis of the three representative bays showed that the grass filtration bays were efficient in removal of NH₃-N. The representative bays (i.e. bays 3, 4 and 7, loaded at 30, 40 and 50 mm/d respectively) achieved reductions of 70, 54 and 39% NH₃-N respectively over the whole period of the study. This indicates that the lower loaded bays removed higher amounts of ammonia nitrogen, while the higher loaded bays removed lower amounts. Hence, it can be said that the ammonia

nitrogen removal efficiency is inversely related to the hydraulic loading rate of the grass filtration bays. This can also be visually observed from the temporal plot of $\text{NH}_3\text{-N}$ (shown in Figure 5.17).

The effluent $\text{NH}_3\text{-N}$ concentrations from the bays were high during the first three weeks of the trials. This was partly due to organic nitrogen sources from decaying plant material left on the bays from the grass filtration treatment of the previous season. After this period, the effluent $\text{NH}_3\text{-N}$ concentrations decreased with time to minimum values and remained low until the end of Period 1, after which they continuously increased and peaked two weeks before the end of the trials. The reason for this peak in the last two weeks is not known.

Temporal plots of $\text{NH}_3\text{-N}$ at the inlet, 200m, 300m and the outlets of all seven bays are also shown in Figure A.14 of Appendix A. According to this figure, the $\text{NH}_3\text{-N}$ concentrations at the 200m mark of all 7 bays are always higher than the 300m mark especially during Period 1. However, during Period 2, the $\text{NH}_3\text{-N}$ concentrations at the 200m, 300m and the outlets approach values of the inlet concentrations and at some instances the 200m mark concentrations even exceed the inlet concentrations. This suggests that lower ammonia uptake took place during Period 2 as also suggested in Section 5.7.1.1 (b).

5.7.1.1(e) Total Kjeldahl Nitrogen (TKN)

The average TKN concentrations of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.7. As mentioned earlier, TKN is the sum of $\text{NH}_3\text{-N}$ and organic nitrogen. Therefore, TKN concentration are always higher than $\text{NH}_3\text{-N}$ concentrations. According to the concentrations data shown in Table 5.7, the average concentrations during Period 1 are much lower than the average concentrations during Period 2 at all locations of the representative bays. Once again, this is due to the lower influent concentrations and higher nitrogen uptake capacity of Ryegrass during Period 1. The difference between the TKN influent concentrations during Period 1 and Period 2 is 33% (i.e. Period 2 influent concentration is 33% higher than Period 1). The corresponding differences in the average effluent

concentrations during Periods 1 and 2 for bays 3, 4 and 7 are 160, 200 and 185% respectively, having higher effluent concentrations.

Table 5.7 Average TKN Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2
Inlet	30.0	39.8	35.0	30.0	39.8	35.0	30.0	39.8	35.0
200m	11.5	33.7	22.9	15.1	37.1	26.3	17.3	39.7	28.7
300m	8.6	22.4	15.7	11.2	30.0	20.8	11.8	34.4	23.3
Outlet	6.6	17.2	7.5	8.4	25.2	17.0	10.7	30.5	20.8

During the whole study period, bays 3, 4 and 7 removed 78, 51 and 40% TKN. These removals are considered quite high especially for bay 3 (i.e. the bay with the lower hydraulic loading rate). Hence, it can be said that the hydraulic loading rate plays a significant role in nitrogen removal efficiency of the grass filtration bays in terms of TKN, and that the TKN removal efficiency is inversely related to the hydraulic loading rate.

Figure 5.18 shows the temporal plots of TKN concentrations at the inlets and the outlets of the representative bays. The temporal variability of TKN is very similar to that of NH₃-N (i.e. Figure 5.17), since NH₃-N is the major constituent of TKN. The TKN temporal plots show high effluent concentrations during the first 3 weeks of Period 1, which dropped to a low level and remained steady for the remainder of this period. This was then followed by a gradual increase in effluent concentrations throughout Period 2. The inverse relationship of the TKN with the hydraulic loading rate can also be seen in Figure 5.18.

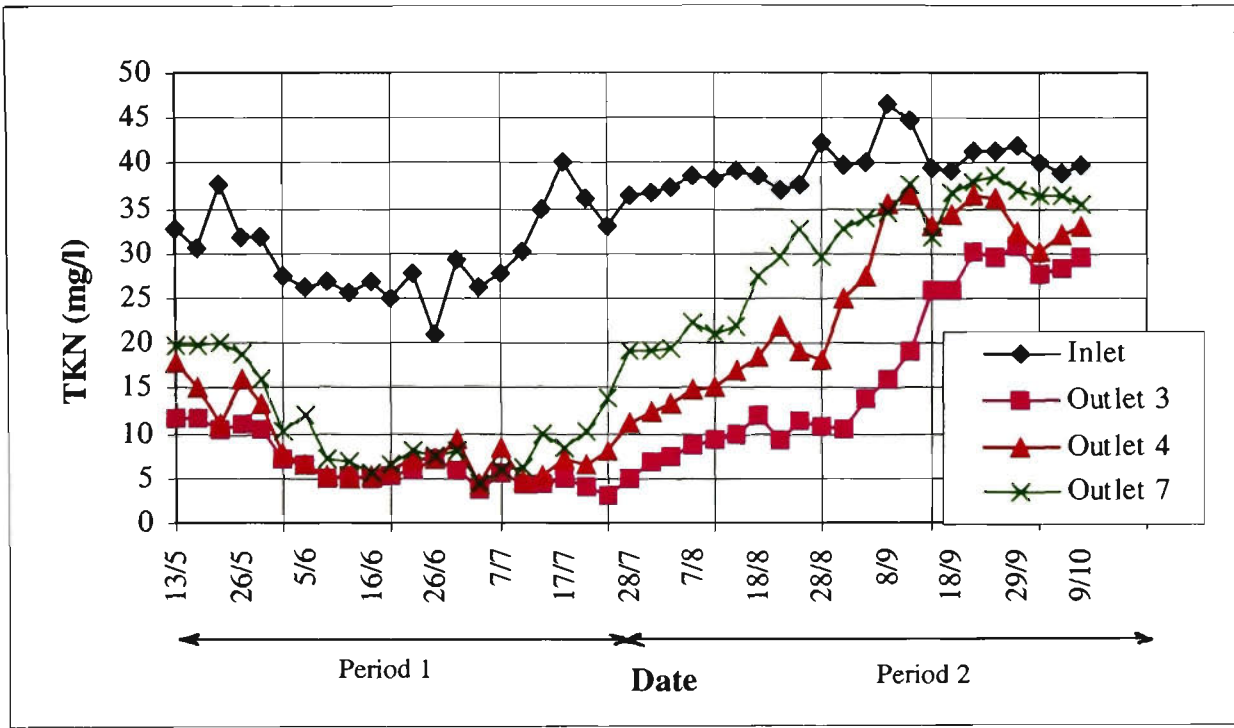


Figure 5.18 TKN Temporal Plot for Representative Bays

Temporal plots of TKN at the inlet, 200m, 300m and the outlets of all seven bays are also shown in Figure A.15 of Appendix A. TKN concentrations for all 7 bays were higher at the 200m mark than the 300m mark throughout Periods 1 and 2. Towards the end of Period 2, the TKN concentrations at the 200m mark were higher than the inlet concentrations for all 7 bays. At the same time, the TKN concentrations at the 300m mark were higher than the inlet concentrations only at the higher loaded bays (i.e. bays 6 and 7). This suggests that TKN uptake within the bays was affected by the hydraulic loading rate.

5.7.1.1(f) Total Nitrogen (TN)

The average TN concentrations of the representative bays for Period 1, Period 2 and the whole study period are shown in Table 5.8. Similar to the other forms of nitrogen, the average TN concentrations are also lower at the inlet, 200m, 300m and the outlets of all representative bays during Period 1 and higher during Period 2. During Period 1, a large proportion of the reduction in TN concentration occurred in the first 200 m of each of bays 3, 4 and 7 especially during Period 1. According to the average TN concentrations shown in Table 5.8, bays 3, 4 and 7 removed 47, 35 and 32% of the influent TN within the first 200m of the bays during Period 1. The corresponding

average reductions in the first 200m during Period 2 are 12, 5 and 0.2% for bays 3, 4 and 7 respectively.

Table 5.8 Average TN Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2	Period 1	Period 2	Periods 1 & 2
Inlet	30.3	40.0	35.3	30.3	40.0	35.3	30.3	40.0	35.3
200m	15.9	34.9	25.6	19.8	37.9	29.0	20.6	40.1	30.6
300m	11.0	23.5	17.4	16.2	30.8	23.7	15.8	35.0	25.7
Outlet	7.6	18.0	12.9	11.7	26.3	19.2	14.4	31.2	23.0

Calculations based on the concentration results of bays 3, 4 and 7 loaded at 30, 40 and 50 mm/d respectively, showed that the average TN removals for the representative bays were 63, 46 and 35% respectively over the entire sampling period. According to these reductions, it is clear that the lower loaded bays achieved higher nitrogen reductions, showing an inverse relationship between hydraulic loading rate and TN removal efficiency. This trend can also be observed from the temporal plots of TN as shown in Figure 5.19. Similar to NH₃-N concentrations, the effluent TN concentrations were generally higher during the first three weeks of the trials. Possible reasons for this are explained in Section 5.7.1.1 (d).

Temporal plots of TN at the inlet, 200m, 300m and the outlets of all seven bays are also shown in Figure A.16 of Appendix A. According to this figure, the temporal plots of all 7 bays show similar characteristics to those of all other forms of nitrogen as discussed in Section 5.7.1.1 (a) to (e). Once again, the TN concentrations at the 200 mark were higher than the concentrations at the 300m mark during Period 1, with a gradual increase in concentrations at the 200m, 300m and the outlet concentrations

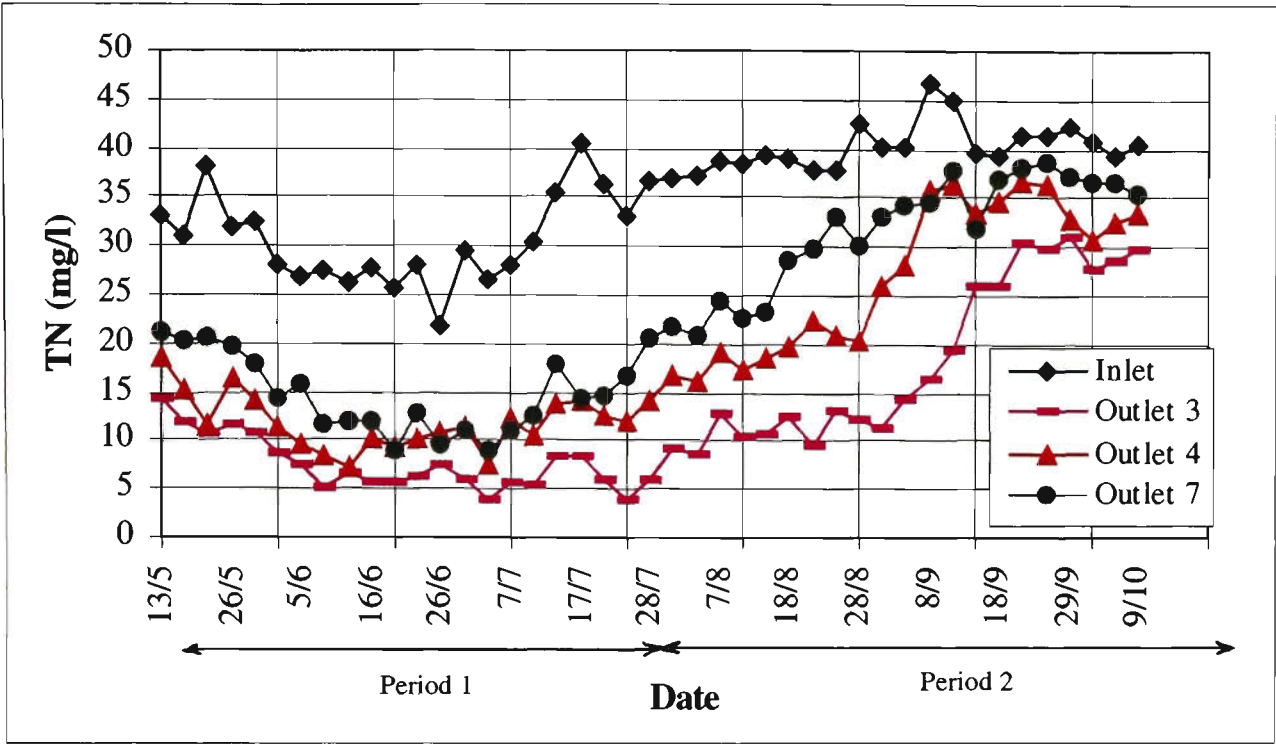


Figure 5.19 TN Temporal Plot for Representative Bays

during Period 2 as the trial progressed. In fact, the highest loaded bay (i.e. bay 7) had a small net gain of nitrogen up to the 200 m point during Period 2. However, it is observed that for specific periods, even in Period 1, when low reduction in TN concentration occurred in the first half of a particular bay, probably due to temporary overloading or some other reasons, the second half of the bays often achieved high nitrogen reductions.

5.7.1.2 Nitrogen Mass Balance

The nutrient removal efficiencies are computed using both concentration and mass balance results. If there are no losses, these efficiencies should be the same, However, since there were flow losses and the flows were unsteady, the removal efficiencies using concentration and mass loads are different.

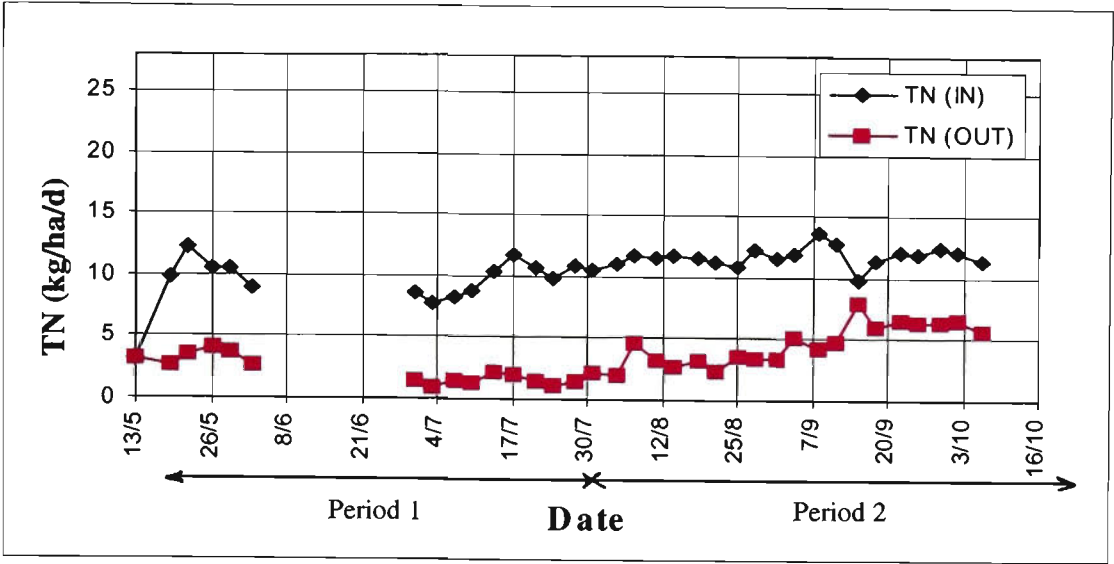
The mass balance analysis of nitrogen is only computed for TN and NH₃-N. This is due to the fact that TN implicitly takes into account all forms of nitrogen. On the other hand, NH₃-N mass balance is carried out since ammonification was a matter of concern and also to investigate the magnitude of NH₃-N loads removed from the bays, as NH₃-N is one of the most harmful forms of nitrogen.

The load of nitrogen which goes into Port Phillip Bay is more critical than the concentration. By performing the mass balance, the loads of nitrogen removed from the influent wastewaters can be determined. It should be noted that the standard unit in accordance with the Victorian Environment Protection Authority (EPA) practice for mass loadings is kg/ha/d and therefore, all nutrient masses at inlet and outlet in this study are reported in kg/ha/d. The mass loadings were computed by multiplying the concentration with flow and then dividing by the grass filtration area.

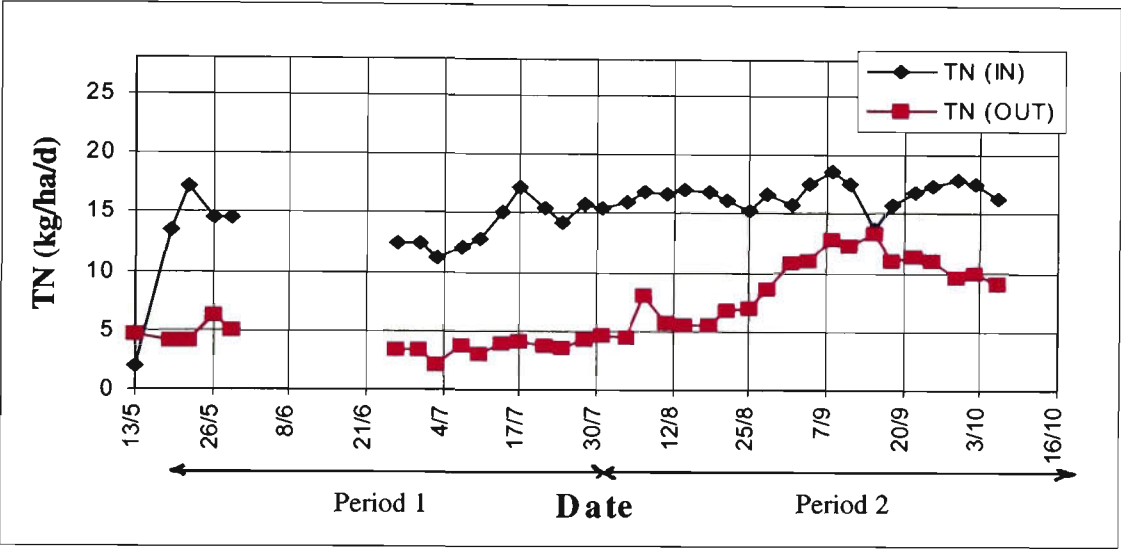
The TN and NH₃-N mass loadings of the influent and effluent for the representative bays are shown in Figures 5.20 and 5.21 respectively. The TN and NH₃-N mass balance plots for all seven bays are presented respectively in Figures A.17 and A.18 of Appendix A. In Figures 5.20 and 5.21, the mass balance results are missing for the period 4 June to 30 June, since no flow data were available during that period. Figures 5.20 and 5.21 show similar patterns and temporal trends to those as discussed under TN and NH₃-N concentrations of Section 5.7.1.1.

Figure 5.22 and Table 5.9 show the absolute removal loads and a summary of removal rates of TN and NH₃-N for the representative bays. As can be seen from Table 5.9, the removal efficiency is higher for lower loaded bays and vice versa. However, it is observed that the absolute removal rates for both TN and NH₃-N of 8.0 and 6.1 kg/ha/day respectively for the 40 mm/d loading rate (i.e. bay 4) were marginally higher than those for the 30 mm/d loading (i.e. bay 3), but significantly higher than the values of 5.9 and 4.5 kg/ha/day for the 50 mm/d loaded bay (i.e. bay 7). This is because the lower loaded bays have a higher removal efficiency with a lower discharge. On the other hand, the higher loaded bays have a lower removal efficiency with a higher discharge. Since these two parameters are multiplied to give the absolute removal of loads, bay 4 with intermediate removal efficiency and discharge had given the highest removal of loads of TN and NH₃-N.

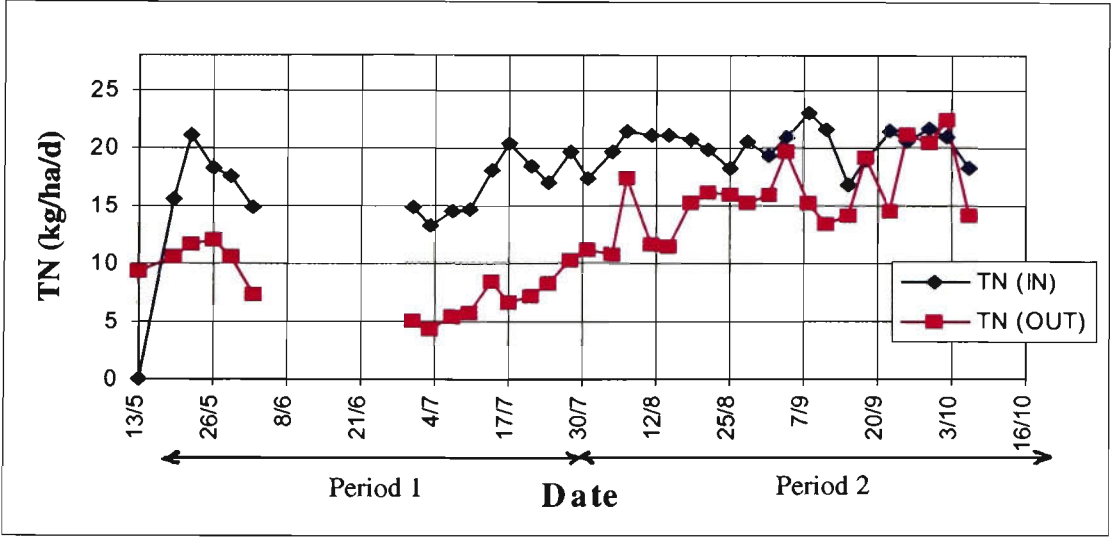
According to Table 5.9, the TN and NH₃-N removal efficiencies were higher during Period 1 than in Period 2. In both cases (i.e. for TN and NH₃-N), the differences in removal efficiencies between Periods 1 and 2 with respect to nitrogen masses seem to correlate generally with the differences in removal efficiencies



(a) Bay 3

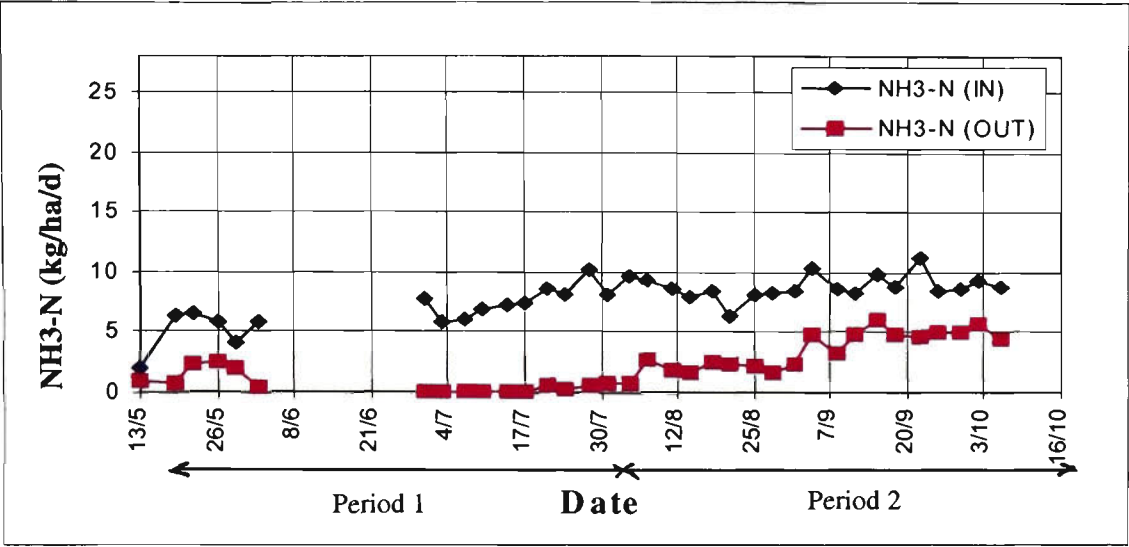


(b) Bay 4

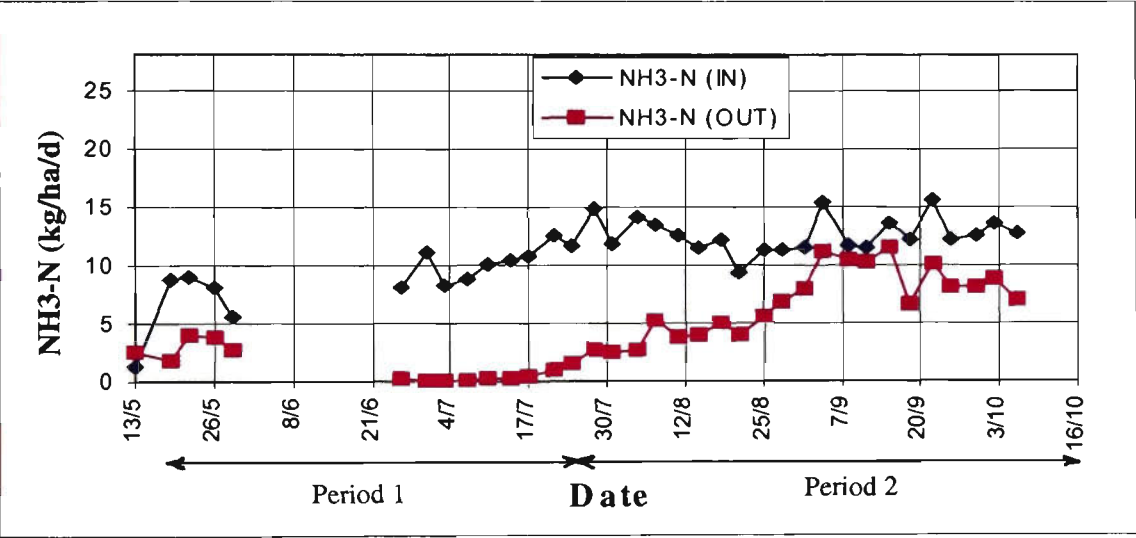


(c) Bay 7

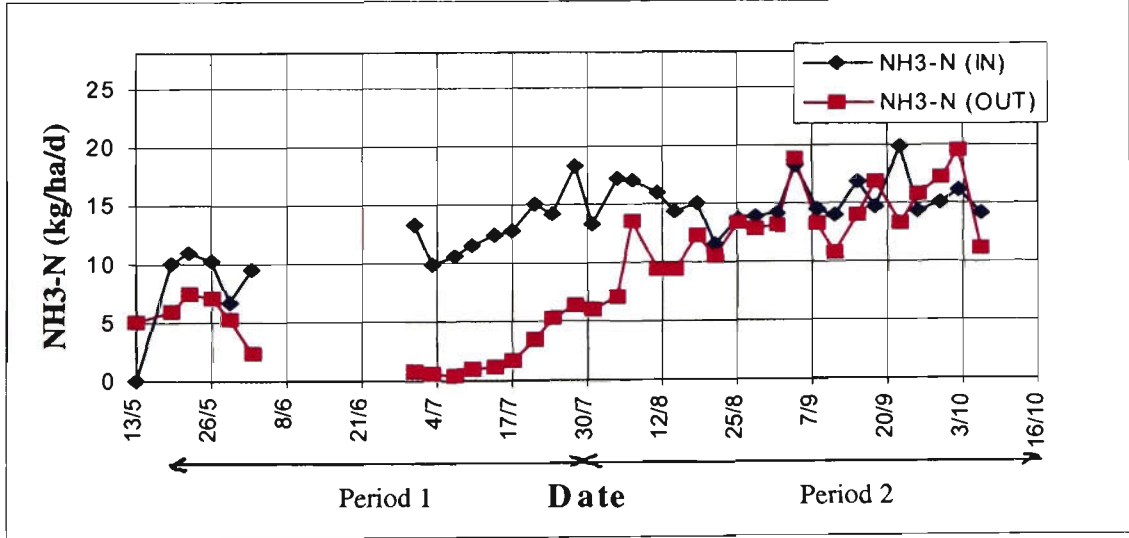
Figure 5.20 Influent and Effluent TN Mass Plots for Representative Bays



(a) Bay 3



(b) Bay 4



(c) Bay 7

Figure 5.21 Influent and Effluent $\text{NH}_3\text{-N}$ Mass Plots for Representative Bays

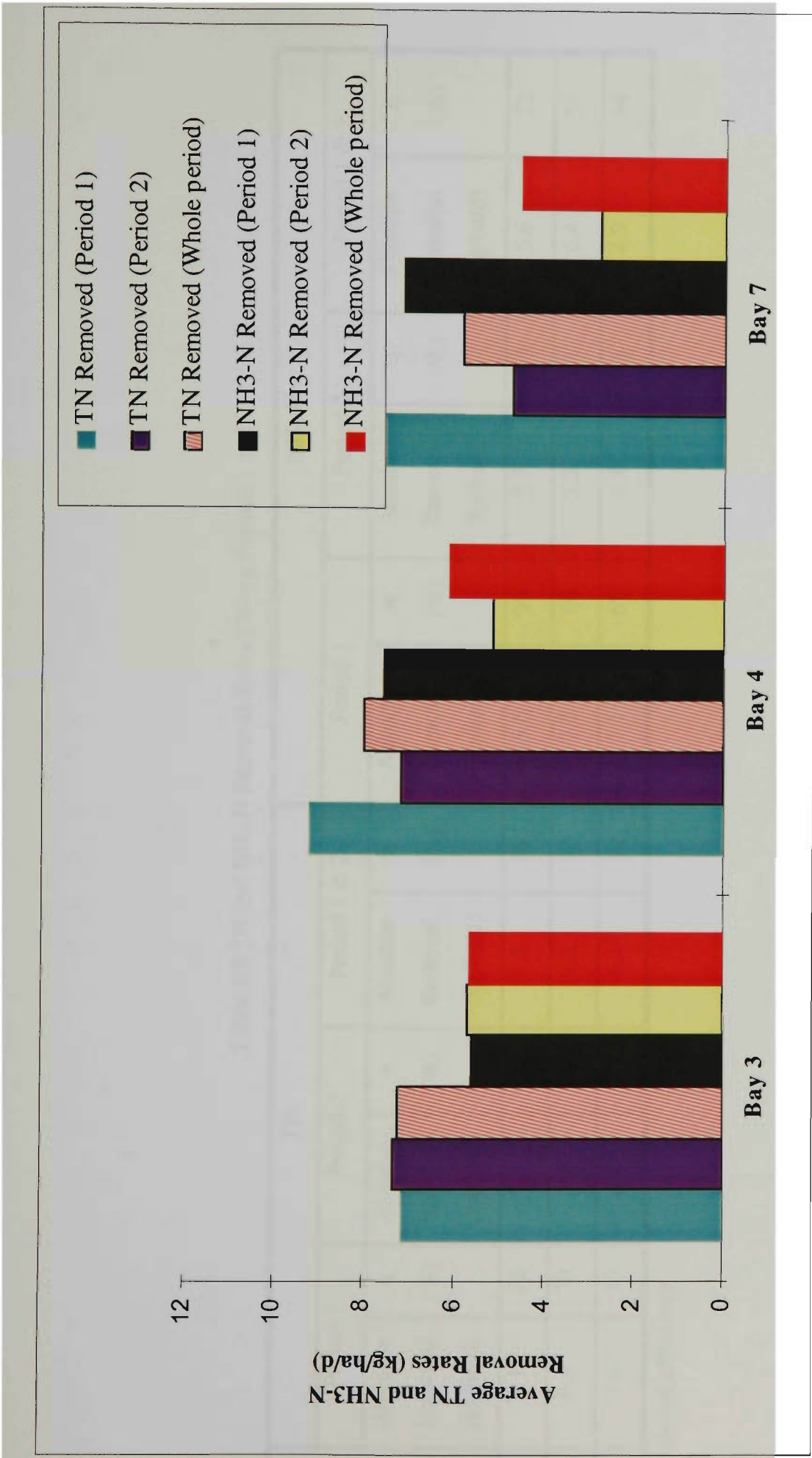


Figure 5.22 TN and NH₃-N Removal Rates for Representative Bays

Table 5.9 TN and NH₃-N Removal Rates During Periods 1 and 2

Bay	TN					NH ₃ -N				
	Period 1		Period 2		Period 1 & 2		Period 1		Period 2	
	Absolute Removal (kg/ha/d)	η (%)	Absolute Removal (kg/ha/d)	η (%)	Absolute Removal (kg/ha/d)	η (%)	Absolute Removal (kg/ha/d)	η (%)	Absolute Removal (kg/ha/d)	η (%)
3	7.1	76	7.3	63	7.2	68	5.6	90	5.7	65
4	9.3	70	7.2	44	8.2	53	7.6	86	5.2	41
7	7.6	39	4.7	23	6.1	32	7.1	68	2.8	18

η - Removal efficiency

computed with respect to nitrogen concentrations (Section 5.7.1.1). These differences are not exactly the same since the mass balance computations take into account the flows which are different for each bay.

Listed below are some factors which may have contributed to the lower percentage removal of TN and $\text{NH}_3\text{-N}$ in Period 2:

- Reduced influent DO concentration, in Period 2 (Figure 5.5),
- Reduced DO source from Italian Ryegrass due to decreased growth rate and photosynthetic activity in Period 2,
- Inhibition of nitrification due to toxic substances such as free ammonia, sulphide, peptone and heavy metals,
- Reduced uptake of nitrogen by Italian Ryegrass due to age,
- Increased influent concentration of TN and $\text{NH}_3\text{-N}$, and
- Increased nitrogen sources from decaying plants and other organic tissue on the bays.

Since denitrification (i.e. a process by which $\text{NO}_3\text{-N}$ is converted to N_2O and N_2 by denitrifying bacteria) does not change significantly during the second half of the trials (Figures 5.14 and 5.15), the first three factors listed above do not appear to be major contributors to the observed trend of higher $\text{NH}_3\text{-N}$ and TN effluent concentrations during Period 2 (see Figures 5.17 and 5.19). However, the remaining factors are likely to contribute, in different degrees, to the deterioration of nitrogen removal during Period 2. Increased influent concentration of TN and $\text{NH}_3\text{-N}$ (i.e. increased mass loading rate of TN and $\text{NH}_3\text{-N}$) during Period 2 and reduced plant uptake of nitrogen are the major contributors to the high levels of these parameters at the outlet. Also, the higher influent $\text{NH}_3\text{-N}$ concentration may be accompanied by higher concentration of free $\text{NH}_3\text{-N}$, especially during periods when the pH falls below 7.0, resulting in an inhibitory effect on nitrification.

5.7.1.3 Nitrogen Removal Mechanisms

Various processes affect the nitrogen load removed from the grass filtration bays. As stated by Sedlak (1991), they include nitrification/denitrification, plant uptake (or immobilisation), volatilisation, infiltration, cross flows and sedimentation. The denitrification closely follows the nitrification process. The above processes can be considered as nitrogen sinks that are responsible for nitrogen load removal in grass filtration bays. These processes and their relative contributions to overall nitrogen removal on the grass filtration bays are described in the following sections. These computed contributions due to various processes should be considered as approximate values due to assumptions stated in following sections. Nitrification/denitrification is presented last, since it is computed as the difference between TN removal (Table 5.9) and the sum of all other form of losses (i.e. plant uptake, volatilisation, infiltration/cross flows/sedimentation).

5.7.1.3 (a) Plant Uptake

Plant uptake is a term which refers to the movement of nutrient ions into the plant roots. The plant uptake is calculated based on estimated nitrogen content in the tissue of Italian Ryegrass. The work of Williams (1992) on Glyphosate phototoxicity trials at WTP estimated that the TN content of herbage on a typical grass filtration bay was about 110 kg/ha after approximately six months of winter grass filtration treatment. This amount of nitrogen content is supported by USEPA (1981), which reported that annual nitrogen uptake rates for the Ryegrass family were between 200 and 280 kg/ha/yr. These estimates by USEPA (1981) did not indicate the temporal trend of nitrogen uptake by Italian Ryegrass. The TN content increased with the amount of nitrogen applied, and was believed to become constant at a certain nitrogen application threshold (USEPA, 1981).

Experimental studies in Britain by Wilman (1965, 1970 and 1975) provided results on temporal nitrogen uptake by Italian Ryegrass over a period of 14 weeks when different amounts of nitrogen were applied. Results of these experimental studies showed that the nitrogen yield due to plant uptake increases quickly, peaked around week 7 - 12 of the growth period, reduced slightly and stayed fairly constant up to week 14. At a

maximum application of 196 kg/ha, the maximum nitrogen content was 134 kg/ha and the average over the 14 weeks was 103 kg/ha. The results of Williams (1992) and the range estimated by USEPA (1981) are in agreement with the work of Wilman (1975). However, the nitrogen mass loads on the trial bays at the WTP are higher than 196 kg/ha, which was reported in Wilman (1975).

On the basis of the above studies, it is assumed that around 110 kg/ha (Williams, 1992) of nitrogen might be taken up by Italian Ryegrass on the trial bays during the (approximately) 6 months trial period. However, it is estimated that this plant uptake (or immobilisation) occurred only during Period 1 and a negligible amount (i.e. 0 kg/ha) of nitrogen is immobilised in Period 2. This is because in the case of the WTP trials, Period 1 (i.e. approximately 11 weeks) coincides with the 14 week period as stated by Wilman (1975) to be the period during which the peak nitrogen yield due to plant uptake occurred on grass filtration bays. Therefore, dividing the value of 110 kg/ha by the number of days in Period 1 (i.e. 74 days), gives the average uptake of TN on the grass filtration bays as 1.5 kg/ha/day.

5.7.1.3 (b) Volatilisation

Volatilisation is the process by which some $\text{NH}_3\text{-N}$ in the water column is transformed to the gaseous form and lost to the atmosphere. Volatilisation is fairly significant on the bays, especially at high $\text{NH}_3\text{-N}$ concentrations and during periods of high water temperatures.

Generally, ammonia volatilization does not occur at pH values below 7.0, and the rate of volatilization increases as pH increases from 7.0 until values of 10.5 to 11.5 when theoretically all ammonia in the wastewater is lost. Furthermore, volatilization is expected to increase with increase in ammonia concentration. Previous laboratory tests on wastewater from grass filtration bays at WTP indicated that under wastewater conditions of 22°C and pH of 7.3, and a detention time of 3 to 6 days of the bays, about 2 to 9 mg/l of ammonia volatilised from an initial ammonia concentration of 39 mg/l (Paspaliaris, 1996). Also, work on temperature effects on ammonia removal (O'Farrell et al., 1972) indicated that the maximum ammonia volatilization efficiency drops by about 25% if temperature drops from 22.2 to 5 °C.

On the basis of the above information, average temperature and pH levels of 10 °C and 7.3 respectively (i.e. these are the average conditions during the trials), average initial ammonia concentration of 20 and 27 mg/l for Period 1 and 2 respectively and detention time of 3 to 5 days (depending on the different bays), may be estimated that 2 and 3 mg/l of ammonia have been volatilised during Periods 1 and 2, respectively, from each grass filtration bay. These values also take into consideration the dense grass covering on the bays which will reduce ammonia volatilisation. Nitrogen mass loss through volatilization is estimated for the various periods of the trials using the corresponding ammonia concentration reduction and the hydraulic loading rate.

5.7.1.3 (c) Infiltration/Cross flow/Sedimentation

Apart from the biological nitrogen reduction and transformations on the bays through plant uptake, volatilisation and nitrification/denitrification, there are also some physical processes that affect the nitrogen removal processes such as sedimentation of particulate organic nitrogen, and leaching of the dissolved inorganic forms of nitrogen together with wastewater losses/gains through infiltration and cross flows. The component of nitrogen losses through infiltration and cross flows is bay-specific and directly depends on the amount of wastewater losses from a bay.

Infiltration and cross flow losses or gains are computed for each period (i.e. Period 1 and 2) and over the whole period. These were computed using the average mass flow rate of TN and the proportion of the average volumetric flow rate on the bay that is lost or gained (see Table 4.4), excluding evapotranspiration losses and rainfall. For example, if 10% of the wastewater is lost on a given bay through infiltration and cross flow, it is assumed that the same percentage of the average mass flow rate of TN is lost on the bay due to wastewater losses. In cases where there is net wastewater gain, negative values are obtained.

5.7.1.3 (d) Nitrification/Denitrification

Nitrification is the process by which $\text{NH}_3\text{-N}$ is converted by bacterial oxidation to $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$. The $\text{NO}_2\text{-N}$ and $\text{NO}_3\text{-N}$ produced by this process can be removed by microbiological reduction to nitrogen gas, a process referred to as denitrification. One of the most important transformations on the grass bays is denitrification of $\text{NO}_3\text{-N}$ to nitrogen gas under low DO conditions and in the presence of denitrifying bacteria (Water Environment Federation, 1992). The nitrogen loss through nitrification/denitrification is similar for all bays during Period 1, although slightly higher for the lowest loaded bay and lower for the other two during Period 2.

Possible regions where nitrification/denitrification is most likely to occur within a grass filtration system are the water column and the root-water interface (Mosey, 1985). Mosey (1985) stated that nitrification is likely to occur when redox potential is in excess of 300 mV within water columns. According to the average redox potential plots (i.e. Figure 5.8), the redox potential in excess of 300 mV was observed between 100 and 200 m points on bay 7 and values close to 300 mV at 100 m points along bays 3 and 4 during late June to early July of the trials. These suggest that nitrification was taking place at the water column within the first 200m of the bays.

The root-water interface is another region where nitrification can occur. This is purely due to supply of oxygen by the roots and the large number of nutrients attached to the roots as slime. Some of the oxygen leaking from the roots is used by the nitrifying bacteria attached to the slimes on the root surface. Usually, a drop in DO level occurs due to nitrifying bacteria consuming oxygen during the conversion of $\text{NH}_3\text{-N}$ to $\text{NO}_2\text{-N}$ and then to $\text{NO}_3\text{-N}$. As can be seen from Figure 5.5, the DO levels within the representative bays fluctuate. Therefore, it is believed that at those points when the DO levels were low, nitrification had occurred by the release of DO through the root zone interface. However, at the points when the DO levels were high it is not known whether nitrification/denitrification had occurred or not.

5.7.1.3(e) Removal Contributions from Nitrogen Sinks

As stated in Section 5.7.1.3, the processes related to nitrogen removal mechanisms are considered as nitrogen sinks. The nitrogen sinks due to different nitrogen transformation and removal mechanisms on the bays are computed based on the values in Section 5.7.1.3 (a) to (d) and are presented in Table 5.10. This information is important for proper design and operation of full-scale grass filtration systems to achieve high removal efficiency.

Table 5.10 Estimated Absolute Removal Rate from Nitrogen Sinks (kg/ha/d)

Removal	Bay 3			Bay 4			Bay 7		
Mechanism	Period 1	Period 2	Period 1 & 2	Period 1	Period 2	Period 1 & 2	Period 1	Period 2	Period 1 & 2
Plant uptake (Immobilisation)	1.50	0	0.75	1.50	0	0.75	1.50	0	0.75
Volatilisation	0.60	0.90	0.75	0.80	1.20	1.00	1.20	1.70	1.45
Infiltration/ Cross flow/ Sedimentation	0.89	1.21	1.04	3.00	2.44	2.88	0.21	-0.87	-0.06
Nitrification/ Denitrification	4.12	5.19	4.62	4.01	3.56	3.58	4.90	4.07	3.97
Total from Table 5.9	7.10	7.30	7.20	9.30	7.20	8.20	7.60	4.70	6.15

As can be seen from Table 5.10, there are some differences in the nitrogen load removals between Periods 1 and 2. Considering the nitrogen removals during Period 1, apart from nitrification/denitrification, the plant uptake is the second most important mechanism which has reduced significant amounts of nitrogen loads. During Period 2, however, the plant uptake is zero. Although this is the case with respect to the computations of plant uptake as explained in Section 5.7.1.3(a), some (relatively negligible) amount of plant uptake would have taken place in the bays during Period 2.

Table 5.10 shows that most of the nitrogen load from the wastewater has been removed through nitrification/denitrification. For bay 3, the total load of nitrogen removed (from this bay) was 7.20 kg/ha/d. The plant uptake was responsible for 10% of the TN removed, while volatilisation, sedimentation/cross flows/infiltration and nitrification/denitrification were responsible for 11, 14 and 65% of the TN removal

respectively. These proportions by different removal mechanisms were similar for bay 7. However, bay 4 showed more than 34% TN removal through infiltration/cross flows/sedimentation mechanisms.

Table 5.10 also shows some negative values for infiltration/cross flow/sedimentation removal mechanism for bay 7 during Period 2. Since the infiltration/cross flow/sedimentation proportions were computed on the basis of volumetric flows, the negative values correspond to flow gains within that bay. Therefore, in the context of nitrogen load removal, the negative values indicate that the cross flows from the adjacent bays caused some nitrogen load deposit into bay 7. Hence, the total nitrogen load removed from bay 7 was lower (i.e. 4.70 kg/ha/d).

In general, considering the proportions of nitrogen load removed from the three representative bays, it can be said that nitrification/denitrification was the most significant nitrogen removal mechanism on the grass filtration trial bays of the WTP. Therefore, the major nitrogen sink is nitrification/denitrification. However, plant uptake, volatilization and sinks due to infiltration and cross flows contribute to smaller removal and transformation proportions. The infiltration/cross flows/sedimentation contribution varies from bay to bay.

5.7.2 Phosphorus

As stated earlier in Section 5.7, phosphorus in the wastewater is present in various forms. For the purpose of this project, the forms of phosphorus measured and analysed included orthophosphate-phosphorus (OP-P) and total phosphorus (TP). Similar to the discussion on nitrogen (i.e. Section 5.7.1), the discussion on phosphorus is based on the representative bays (i.e. bays 3, 4 and 7).

5.7.2.1 Phosphorus Concentration

5.7.2.1(a) Orthophosphate Phosphorus (OP-P)

In this project, OP-P concentrations were measured at the common inlet, the outlets of all seven grass filtration trial bays, and at the 200m transverse section of bays 1, 2, 4 and 6. These bays were selected for these measurements before it was known that bays 3, 4 and 7 would be the representative bays. Therefore, no OP-P data is available at the 300 m points along the trial bays. Table 5.11 shows the average OP-P concentrations at the common inlet, 200m mark (only for bay 4) and the outlet of the representative bays for Periods 1 and 2, and the entire duration of the project.

Table 5.11 Average OP-P Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period 1	Period 2	Period 1 & 2	Period 1	Period 2	Period 1 & 2	Period 1	Period 2	Period 1 & 2
Inlet	3.70	4.40	4.05	3.70	4.40	4.05	3.70	4.40	4.05
200m	N/A	N/A	N/A	3.60	10.9	7.25	N/A	N/A	N/A
Outlet	1.80	11.4	6.60	3.00	12.6	7.80	4.50	11.3	7.90

On average, 1.9, 0.7 and -0.8 mg/l OP-P was removed in bays 3, 4 and 7 respectively during Period 1. However, the same bays produced 7.0, 8.2 and 6.9 mg/l OP-P during Period 2 of the trials. Thus, bays 4 and 7 achieved the lowest reduction of OP-P concentration during Period 1 and bays 3 and 4 produced the highest amount of OP-P during Period 2. Based on the average OP-P concentration results, bays 3, 4 and 7 produced 53, 83 and 95% OP-P over the entire duration of the trials respectively.

Figure 5.23 shows the temporal plot of OP-P for the representative bays. This figure suggests that generally all 3 bays did not perform well in removal of OP-P. The OP-P removal was only significant during the first 8 weeks of the trials. From week 8 onwards (i.e. around 30 June) the OP-P production started and gradually increased. During the later stages of the trials (i.e. during Period 2), the grass filtration bays mainly showed production of phosphorus.

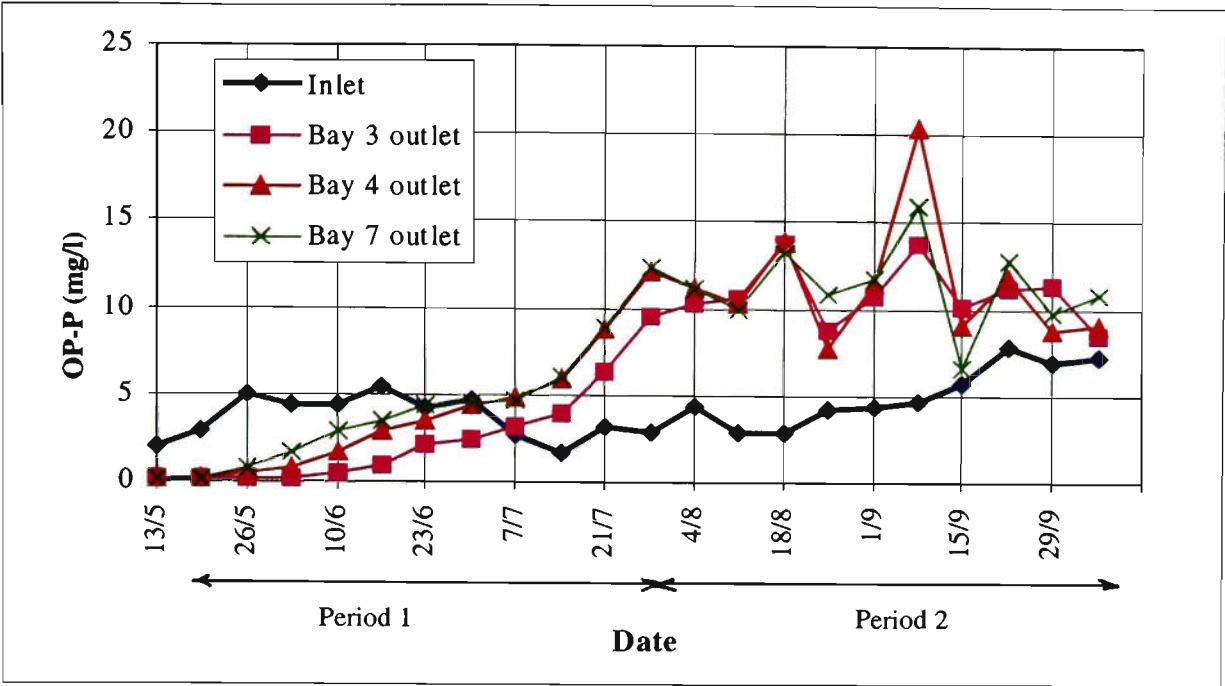


Figure 5.23 OP-P Temporal Plot for the Representative Bays

OP-P and TP plots for all seven trial bays are shown in Figures A.19 and A.20 of Appendix A. According to these plots the OP-P concentrations at the 200 m mark (i.e. for bays 1, 2, 4 and 6) are higher than the outlet concentrations during the entire duration of the trials. However, they are higher than the inlet concentrations mainly during the second half of Period 1 and the whole duration of Period 2. This suggests that some OP-P reduction occurred during the first half of Period 1, however, for the remainder of the trial’s duration, mainly OP-P production occurred.

5.7.2.1(b) Total Phosphorus (TP)

TP was measured once a week at the inlet and the outlets of all seven bays and at the 200m mark of bays 1, 2, 4 and 6. Table 5.12 shows the average TP concentrations at the common inlet, 200 m mark (only for bay 4) and the outlet of the representative bays for Periods 1 and 2, and the entire duration of the trials.-

As can be seen from Table 5.12, on average, during Period 1, 6.6, 5.3 and 3.7 mg/l TP was removed in bays 3, 4 and 7 respectively. Therefore, these bays achieved good reduction of TP during Period 1. However, the outlet concentrations were higher than

Table 5.12 Average TP Concentrations (mg/l) for Representative Bays

	Bay 3			Bay 4			Bay 7		
	Period	Period	Period	Period	Period	Period	Period	Period	Period
	1	2	1 & 2	1	2	1 & 2	1	2	1 & 2
Inlet	8.80	9.20	9.00	8.80	9.20	9.00	8.80	9.20	9.00
200m	N/A	N/A	N/A	4.50	14.0	9.25	N/A	N/A	N/A
Outlet	2.20	11.90	7.10	3.50	13.0	8.25	5.10	13.8	9.45

the inlet concentrations during Period 2 for all bays, suggesting that the bays produced TP during this period. During this period, bays 3, 4 and 7 produced 1.4, 2.4 and 4.6 mg/l TP respectively. It is not unusual for grass filtration bays to produce TP, especially when the trials progress towards the end (Bartlett, 1972). Most vegetation and grasses of different kinds tend to deposit small amounts of phosphorus through their root zones. Also, the phosphorus deposits which have remained in the soil matrix from previous seasons, may contribute to this phosphorus production on the grass filtration bays. Based on the average TP concentration results of Table 5.12, bays 3, 4 and 7 removed 21, 8 and -2% of the influent TP over the entire duration of the trials respectively.

Figure 5.24 shows the temporal plot of TP for the representative bays. According to this figure, the temporal trends between the three representative bays are similar with minor variations throughout the sampling period. This figure clearly indicates that bay 3 was performing better than the other two higher loaded bays during Period 1. However, the TP content at the three bay outlets gradually increased to a point where the bays did not achieve any significant reduction, and the TP outlet concentrations were in fact generally higher than the influent levels during Period 2. It is believed that most of the phosphorus removal during the early part of the season goes to plant growth. The TP temporal plots for all seven bays are shown in Figure A.20 Appendix.A. This figure shows similar trends as the OP-P concentrations at the inlet, 200 m mark of bays 1, 2, 4 and 6 and the outlets. The TP concentrations at the 200 m mark of bays 1, 2, 4 and 6 are higher than the outlet concentrations for the whole trial period. However, they are higher than the inlet concentrations from the second half of Period 1 to the end of the trials. This once again suggests that some TP removal had

taken place during the first half of Period 1, since the inlet concentrations are higher than those at the 200 m mark and the outlet concentrations.

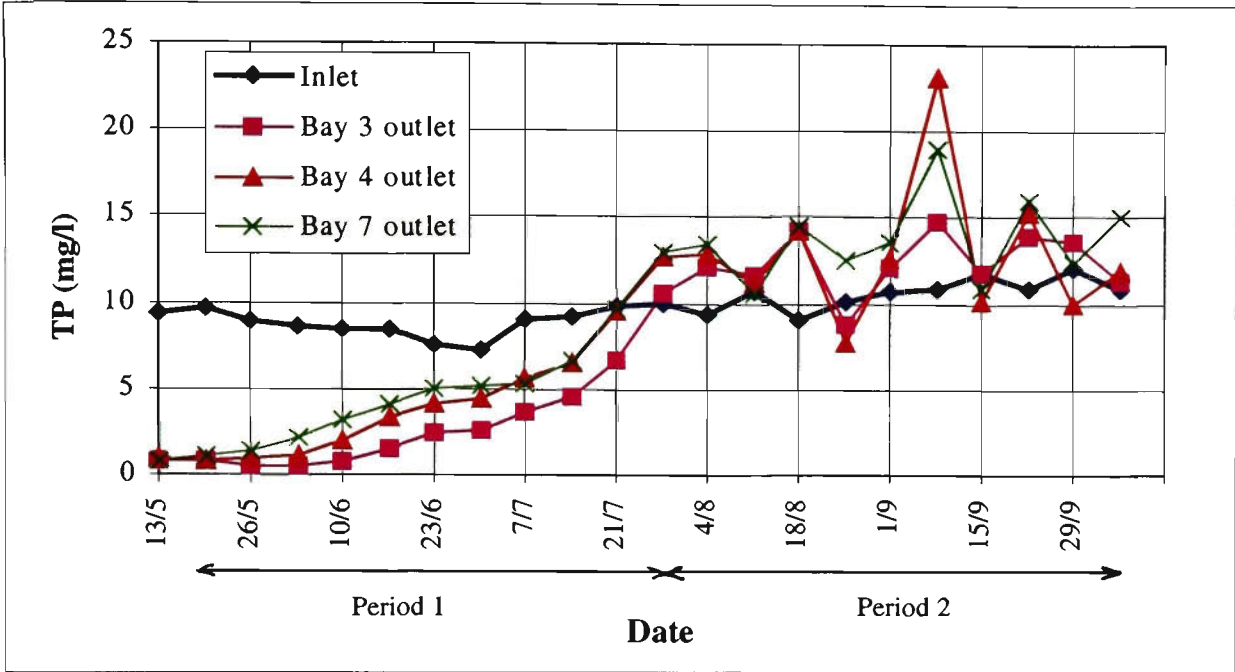


Figure 5.24 TP Temporal Plot for Representative Bays

5.7.2.1(c) Effect of Hydraulic Loading Rate on Phosphorus Removal

The hydraulic loading rate of the bays is an important factor in phosphorus removal, as is the case with nitrogen removal. As stated earlier, the evaluation of OP-P and TP removals from the 3 representative bays showed that bay 3 was the best performing bay in OP-P and TP removal. This was due to two significant factors, its lower hydraulic loading rate (i.e. 30 mm/d) and its higher detention time compared to those of other two bays (see Table 4.7).

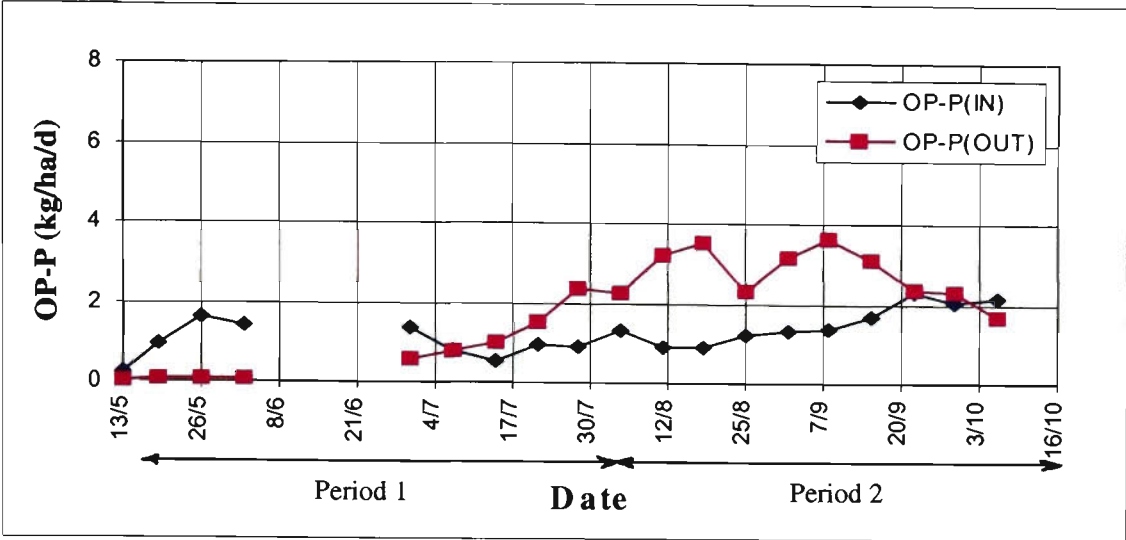
As stated earlier in Section 4.5, these two factors are inter-related with higher hydraulic loading rate producing lower detention time and vice-versa. During Period 1, when both OP-P and TP removals took place, an inverse relationship between the hydraulic loading rate and the phosphorus removal was also observed. This is also supported by the work of Paspaliaris (1996), who studied the phosphorus removal capacity of a grass filtration system and found that phosphorus removal was inversely related to the hydraulic loading rate. He studied several loading rates, which ranged from 0.05 MI/ha/d to 0.20 MI/ha/d (i.e. 5 to 20 mm/d respectively). As part of the

study of Paspaliaris (1996) it was concluded that phosphorus removal can be effective at loadings of 0.1 ML/ha/d to 0.17 ML/ha/d (i.e. 10 to 17 mm/d) using the grass filtration system. In this trial study, the closest hydraulic loading to the range commented on by Paspaliaris (1996) was the 20 mm/d bay (i.e. bay 1). According to Figure A.19 Appendix A, the performance of bay 1 in removal of phosphorus was not much different to that of bay 3 in terms of its temporal variability. For instance, OP-P and TP removal took place during Period 1, while during Period 2 both OP-P and TP effluent concentrations exceeded the influent concentrations suggesting OP-P and TP production during that period. However, the maximum effluent OP-P and TP concentrations of bay 1 were 14 and 16mg/l respectively, while bay 3 (the best performing bay within the representative bays in phosphorus removal), had maximum effluent OP-P and TP concentrations of 14 and 15 mg/l respectively. Therefore, the evaluation of bay 1 and bay 3 performance in removal of phosphorus suggest that although the phosphorus removal is inversely related to the hydraulic loading rate, the difference in performance is not significant between hydraulic loading rate of 20mm/d (i.e. bay 1) and 30mm/d (i.e. bay 3).

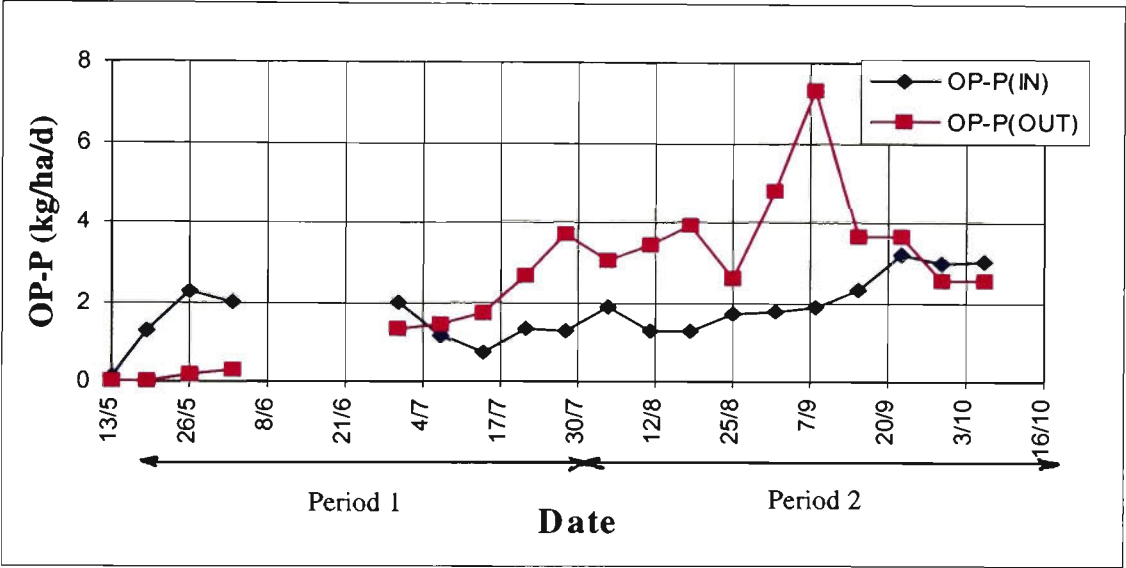
5.7.2.2 Phosphorus Mass Balance

The mass balance analysis of phosphorus is carried out for both OP-P and TP in a similar manner to that for $\text{NH}_3\text{-N}$ and TN in Section 5.7.1.2. The OP-P and TP mass loadings of the influent and effluent for the representative bays are shown in Figure 5.25 and Figure 5.26 respectively. The OP-P and TP mass balance plots for all seven grass filtration bays are shown in Figures A.21 and A.22 respectively in Appendix A. In Figures 5.25 and 5.26, the results are missing between 4 June to 30 June due to lack of flow data at the inlet and the outlets during that period. The absolute removal loads and a summary of removal rates of OP-P and TP are shown in Figure 5.27 and Table 5.13 respectively.

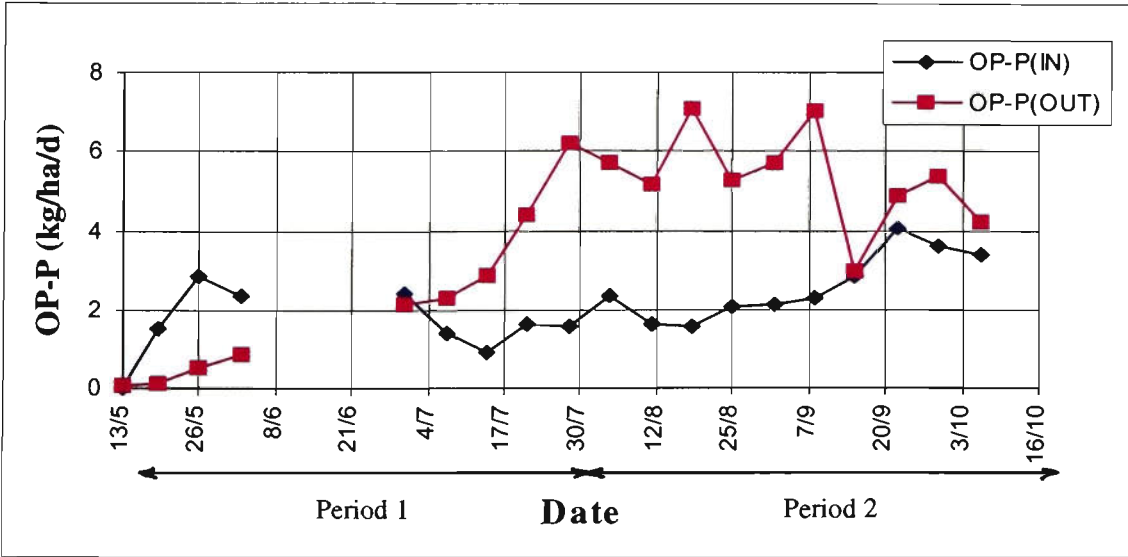
According to Figure 5.25, bays 4 and 7 produced the most amount of OP-P for the entire sampling period and this is also evident from the concentration plots (i.e. Figure



(a) Bay 3

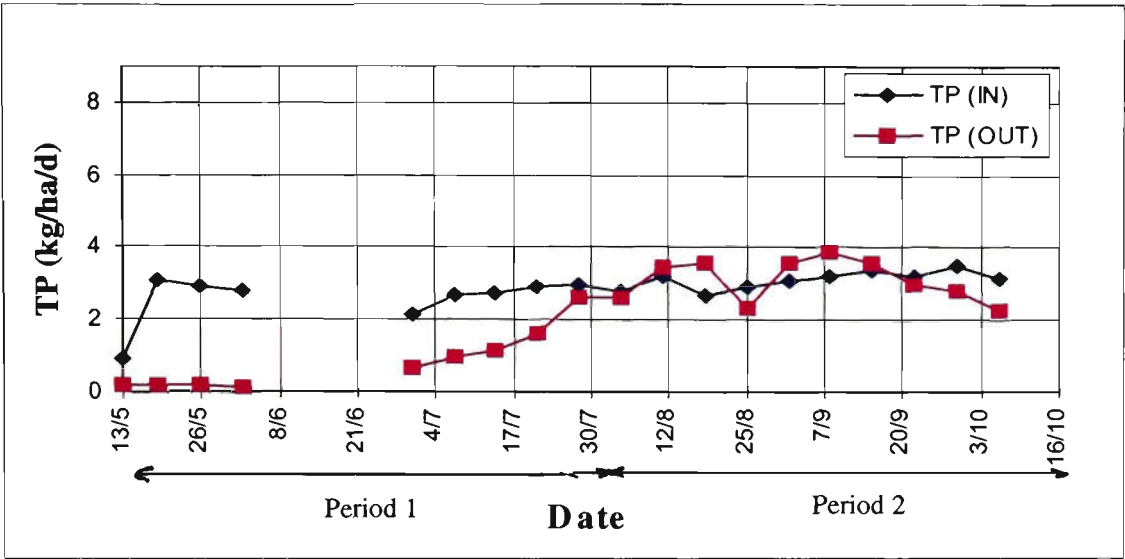


(b) Bay 4

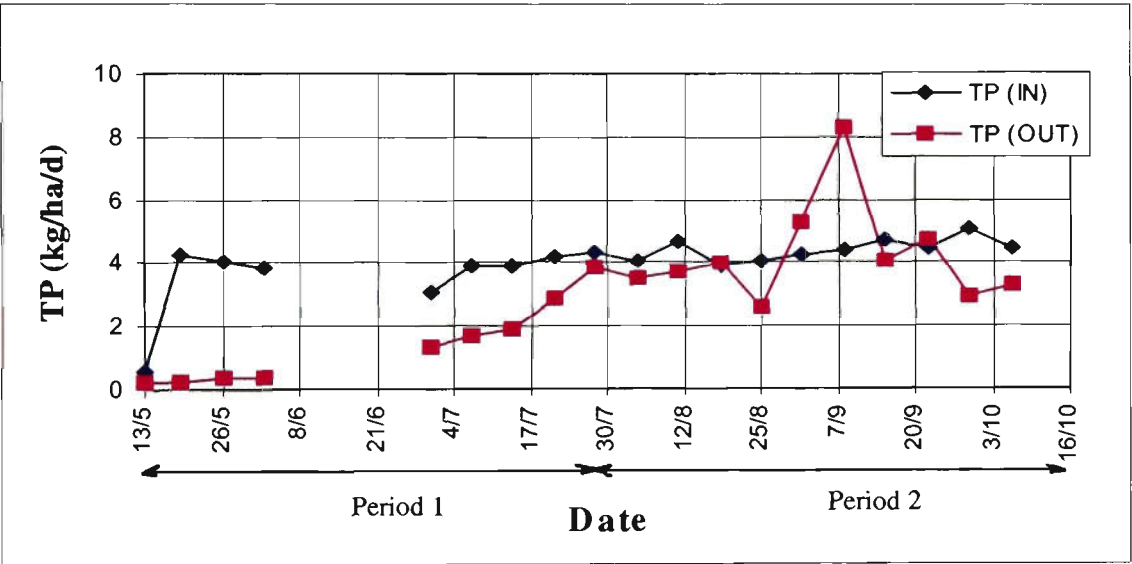


(c) Bay 7

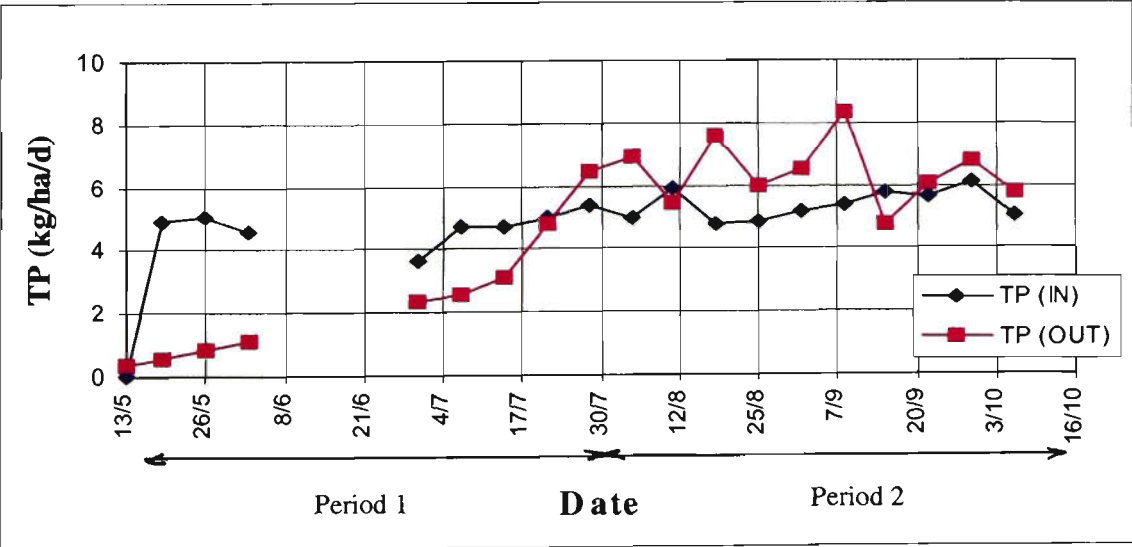
Figure 5.25 Influent and Effluent OP-P Mass Plots for Representative Bays



(a) Bay 3



(b) Bay 4



(c) Bay 7

Figure 5.26 Influent and Effluent TP Mass Plots for Representative Bays

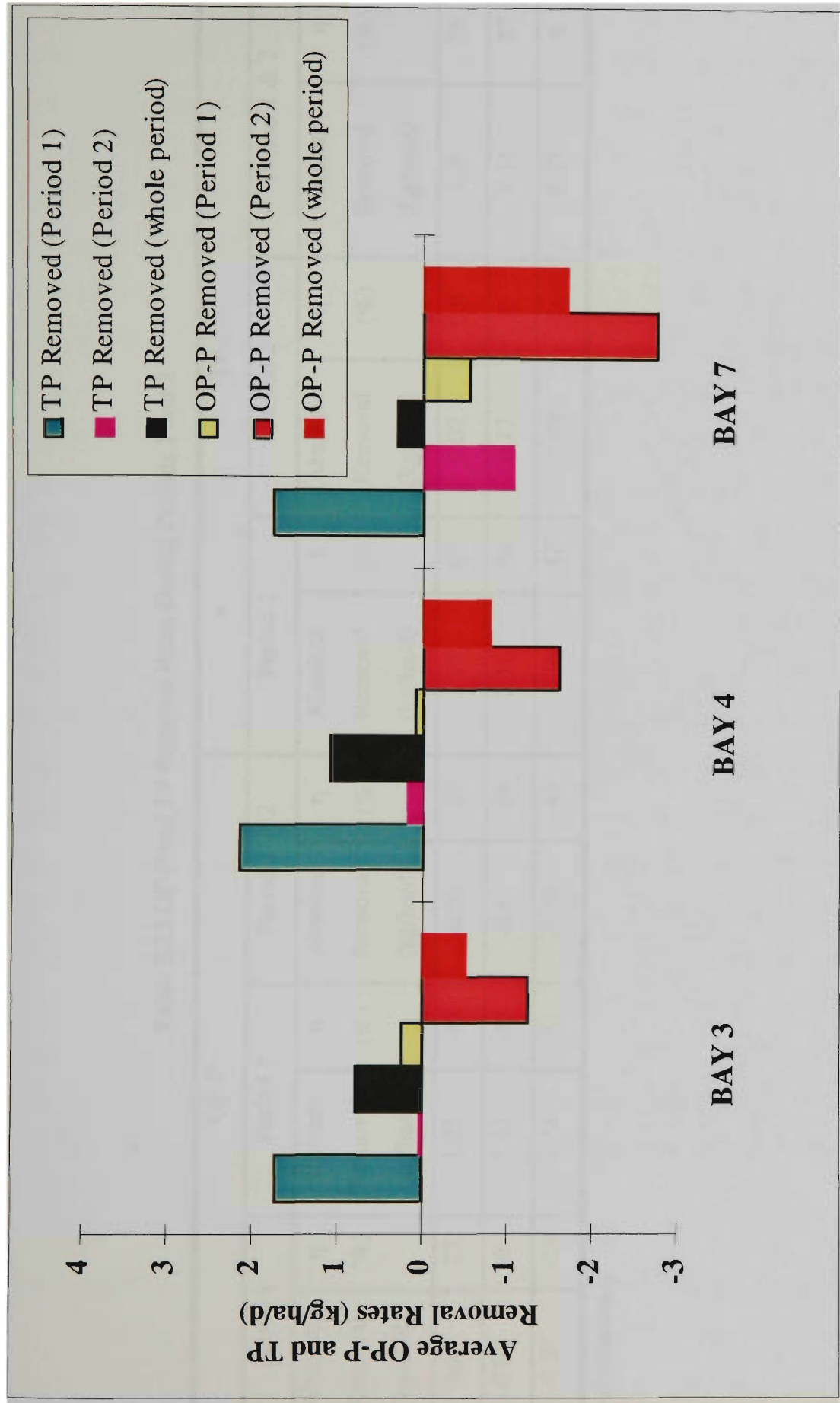


Figure 5.27 TP and OP-P Removal Rates for Representative Bays

Table 5.13 OP-P and TP Removal Rates During Periods 1 and 2

Bay	OP-P						TP					
	Period 1			Period 2			Period 1 & 2			Period 1		
	Absolute Removal (kg/ha/d)	η (%)		Absolute Removal (kg/ha/d)	η (%)		Absolute Removal (kg/ha/d)	η (%)		Absolute Removal (kg/ha/d)	η (%)	
3	0.24	25		-1.23	-45		-0.50	-33		1.72	67	
4	0.09	6		-1.62	-43		-0.81	-34		2.16	60	
7	-0.54	-25		-2.74	-51		-1.70	-45		1.76	42	

η - Removal efficiency

5.24). During Period 1 of the trials, all three representative bays achieved some modest reduction in OP-P. According to Table 5.13, bays 3, 4 and 7 achieved reductions of 25, 6 and -25% in OP-P load during Period 1 respectively, while the same bays produced 45, 43 and 51% during Period 2.

Table 5.13 also shows that the highest absolute removal of OP-P was achieved by bay 3, while the highest absolute removal of TP was achieved by bay 4. Bay 3 achieved absolute removals of 0.24, -1.23 and -0.50 kg/ha/d of OP-P during Period 1, Period 2 and Periods 1 and 2 respectively. Similarly, bay 4 achieved absolute removals of 2.16, 0.17 and 1.11 kg/ha/d of OP-P during Period 1, Period 2 and Periods 1 and 2 respectively. Since the only difference between these two bays is their hydraulic loading rates (i.e. 30 and 40 mm/d for bays 3 and 4 respectively), this suggests that OP-P removal is more sensitive to the hydraulic loading rate than TP removal, because the most OP-P removal occurred at the bay with lower hydraulic loading rate.

As stated earlier, the phosphorus load removal efficiency was lower in Period 2 compared to Period 1 of the trials. It is believed that a combination of very little (or no) phosphorus uptake by plants and wash-off of dead micro-organisms and plant tissue during Period 2 were the causes of phosphorus production on the grass bays. On the other hand, a factor which may explain some of the phosphorus removal during Period 1 is the rapid uptake capacity of the vegetation while in a growth stage. As discussed earlier, this is also supported by Bartlett (1972) who stated that most of the phosphorus removal during the early season goes to plant growth. After the plants have stopped their growth (i.e. matured), their phosphorus uptake dramatically reduces. This reduction of phosphorus uptake due to the ageing of the vegetation on the bays may explain the inefficiency of phosphorus removal during Period 2 of the trials.

Detention time of the grass filtration bays may also affect the phosphorus removal. The longer the detention time of the bays, the longer is the wastewater contact with the soil matrix and the vegetation on the bays. This allows the soil matrix to absorb some of the phosphorus and also enables the vegetation to uptake some of the phosphorus available within the water column. In this trial study, the 3 representative

bays (i.e. bays 3, 4 and 7) were found to have detention times of 4.08, 4.21 and 3.07 days respectively (Section 4.5.2.2(d)). Hence, analysis based on phosphorus concentration and mass balances showed that bay 3 was the best performing bay in removal of phosphorus amongst the 3 representative bays. As also stated earlier (Section 5.7.2.1(c)), this is due to the hydraulic loading rate and detention time of the bay being inter-related, with higher hydraulic loading producing lower detention time and vice-versa.

Another factor affecting the phosphorus removal on grass filtration bays (which was also discussed in Section 5.7.2.1(c)), is the hydraulic loading rate. Similar to the phosphorus removal trends observed under the concentration results, the results based on the mass balance also show that during Period 1 the higher loaded bays (i.e. bays 4 and 7) removed larger amounts of TP. However, Table 5.13 shows that over the entire duration of the trials, bays 3, 4 and 7 achieved reductions of 28, 27 and 6% in TP. This clearly shows that the higher loaded bay (i.e. bay 7 loaded at 50 mm/d) removed the lowest TP load over the entire duration of the trials compared to lower loaded bay (i.e. bay 3 loaded at 30 mm/d) which removed the highest TP load. This suggests that the detention time and phosphorus removal capacity of the grass filtration bays are inversely related to the hydraulic loading rate.

5.8 IMPLICATIONS OF TRIAL RESULTS FOR MELBOURNE WATER'S EPA LICENCE

In recent years, concerns have been raised regarding both eutrophication in Port Phillip Bay due to high nitrogen loading from the WTP, and odour problems affecting communities near the treatment plant. Increased waste loads discharged to Port Phillip Bay due to population increase and a demand by the public for higher environmental standards have warranted stricter wastewater discharge licence conditions for the WTP. These new licence conditions were derived by the Environmental Protection Authority (EPA) of Victoria in consultation with Melbourne Water based on National Water Quality Management Strategy (NHMRC, 1996) as well as the existing State Environmental Protection Policies and Guidelines (EPA, 1997).

A summary of the new licence conditions for critical water quality constituents relevant to WTP are given in Table 5.14. The licence conditions are given in terms of minimum, maximum, 50% and 90% non-exceedance percentiles for most water quality constituents except coliforms which is expressed as the geometric mean of 5 samples. The licence conditions in Table 5.14 are based on concentration, except for total nitrogen which is also expressed as an annual licence in terms of the load. The performance level of the representative bays of 3, 4 and 7 with respect to these critical water quality constituents are computed using data of the trials in winter and are given in Table 5.14. The comparison between the licence conditions and the performance shows the compliance. The representative bays consider all hydraulic loading rates of the trials.

The following sections evaluate and discuss the compliance level of each of these critical water quality constituents. In some cases, reference is made to comparisons between the concentrations of influent to the trials and to the proposed prototype system, and ways of improving the effluent quality.

5.8.1 BOD

Figure 5.28 shows the cumulative non-exceedance probability distribution of effluent BOD concentrations for bays 3, 4, and 7. From this figure, the BOD performance indicators can be read for the representative bays. These values are given in Table 5.14. As can be seen from Table 5.14, the effluent BOD is within EPA licence requirements. However, to further improve wastewater effluent quality, especially for the 40 and 50 mm/d loadings, BOD removal during Period 2 of the trials should be improved, as can be seen from Figure 5.11. Since influent BOD was higher during Period 2 of the trials, an improved compliance level may be achieved by improving influent quality through secondary treatment to the same level of BOD as in Period 1.

Table 5.14 Summary of new EPA Licence

Parameter ¹	EPA Licence				Bay 3 (30 mm/d) ²				Bay 4 (40 mm/d) ²				Bay 7 (50 mm/d) ²			
	min	50%	90%	max	min	50%	90%	max	min	50%	90%	max	min	50%	90%	max
BOD	-	20	40	-	<5	5	29	45	<5	10	40	65	<5	15	35	45
TSS	-	30	60	-	<5	5	15	20	<5	10	25	60	<5	5	33	60
TP	-	10	15	-	0.4	8	14	15	0.7	9	14	23	0.8	10	15	19
Apparent Colour (Pt-Co)	-	200	400		350	400	595	600	300	375	500	600	250	300	400	500
NH ₃ -N	-	10	-	30	0.1	5	22	24	0.1	10	27	32	0.7	13	32	35
TN	-	-	40	-	3.8	11	28	31	7	16	34	36	9	21	37	39
Faecal Coliforms (Org/100ml) ³	G.Mean of 5 samples ≤ 1000				10	20	70	150	10	20	120	240	10	40	120	230
TN Load (t/yr)	3500				875 [25%] ⁴				1350 [39%] ⁴				1860 [53%] ⁴			

¹ Unless otherwise specified, all units expressed in mg/l,

² Hydraulic loading rate,

³ E-coli values of bay 7 are based on measurements of Bay 6 which has a hydraulic loading rate of 50 mm/d,

⁴Percentage of EPA annual licence allowance,

The figures in **Bold Print** represent the instances where monitored parameters have exceeded the EPA discharge limits, All figures > 1 have been rounded off to the nearest whole number.

Legend:

- min - The minimum value within the range of data
- max - The maximum value within the range of data
- 50% - The 50th percentile of the data
- 90% - The 90th percentile of the data

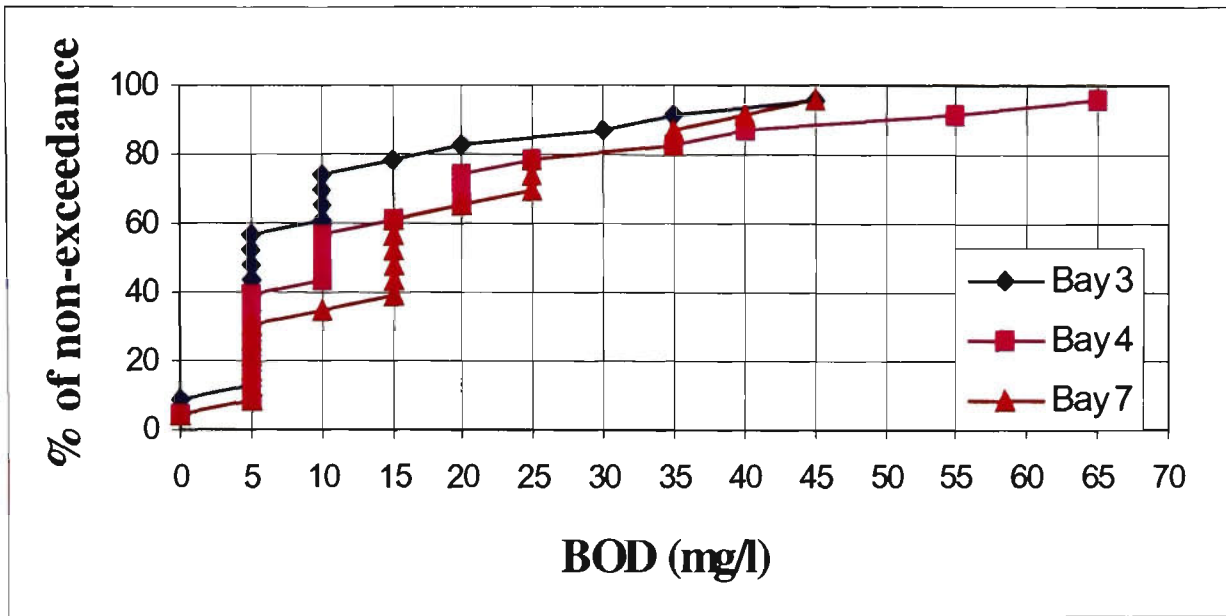


Figure 5.28 Cumulative Non-Exceedance Probability Distribution of Effluent BOD Concentrations for Bays 3, 4, and 7

5.8.2 TSS

The non-exceedance probability plots in Figure 5.29 and the statistics of effluent concentrations of TSS (see Table 5.14) clearly indicate that the licence requirements for this parameter are met for each of the performance bays. The median (i.e. 50th percentile) and 90th percentile limits in the licence are significantly higher than the corresponding values achieved during the trials.

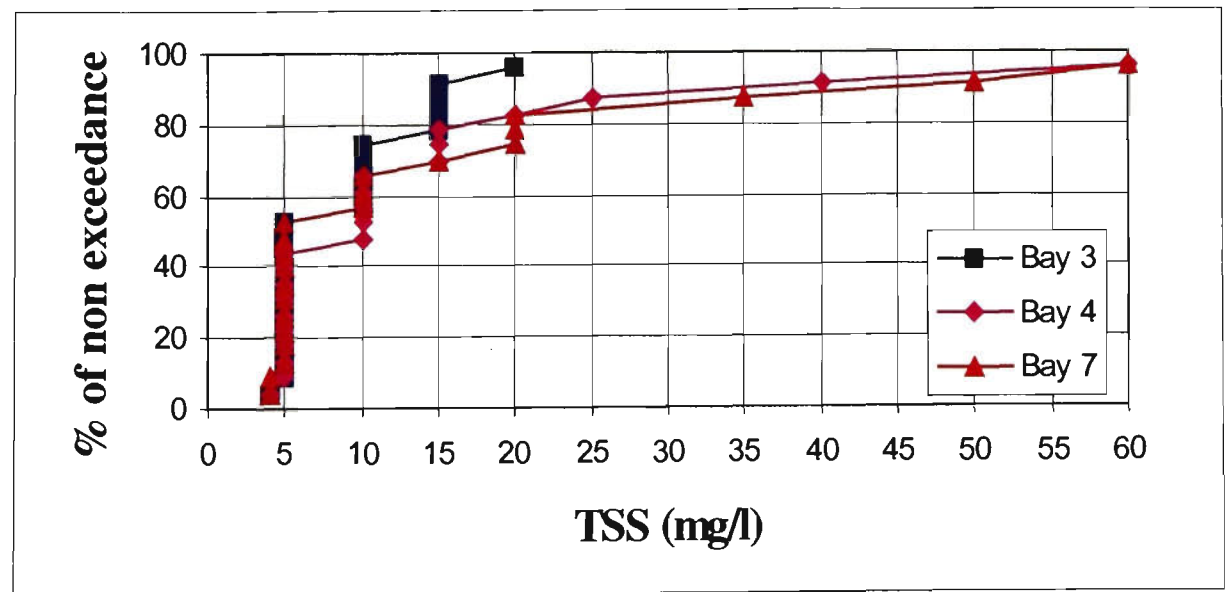


Figure 5.29 Cumulative Non-Exceedance Probability Distribution of Effluent TSS Concentrations for Bays 3, 4, and 7

5.8.3 TP

Figure 5.30 and Table 5.14 show that TP concentration limits of the licence can be achieved at all the hydraulic loading rates used for the trials. Although TP concentrations of the effluent from the trial bays were not significantly lower than the required standards, the compliance level is expected to be very high since the phosphorus concentration of the influent for the proposed full-scale system is generally lower than that of the trials. For example, the average, minimum, median, 90th percentile, and maximum phosphorus concentration of the influent to the proposed full-scale system are 8.5, 2.3, 8.4, 10.0 and 12.9 mg/l respectively, while the corresponding values at the trial bays are 9.7, 7.3, 9.4, 10.9, and 12.0 mg/l (i.e. Section 5.7.2.1).

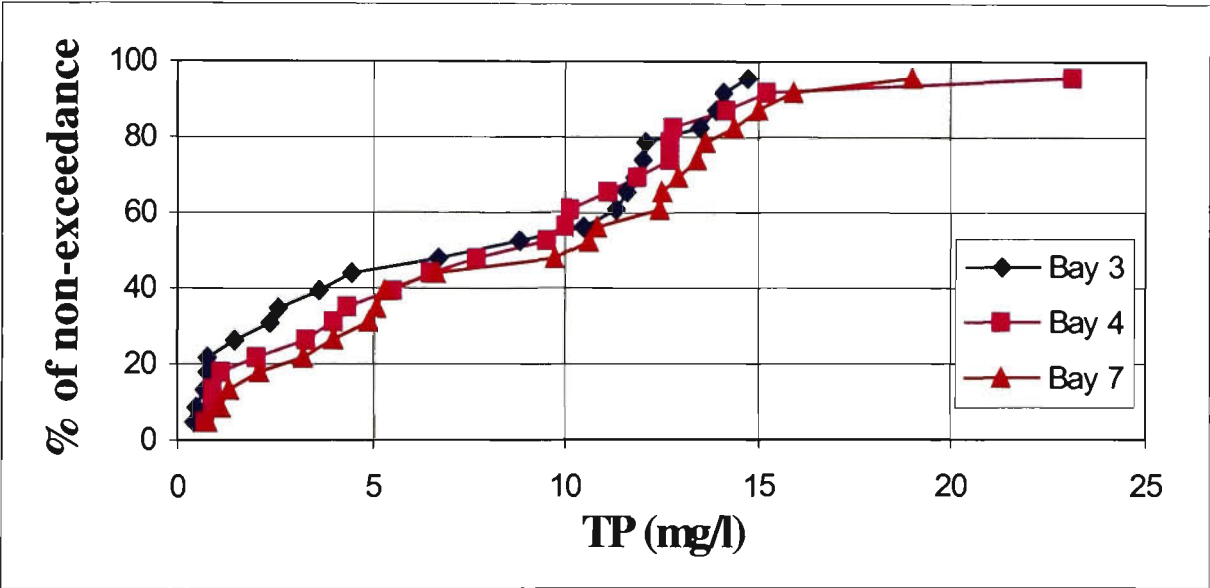


Figure 5.30 Cumulative Non-Exceedance Probability Distribution of Effluent TP Concentrations for Bays 3, 4, and 7

5.8.4 Colour

The results of the trials suggest that apparentt colour standards of Melbourne Water’s EPA licence are violated for each of the loading rates of the trials (see Figure 5.31 and Table 5.14). Based on these figures, colour is the only parameter measured in this study which has consistently violated the EPA discharge licence limits. While only the 50th percentile limit is exceeded by the 50 mm/d loaded bay, both the median and90th percentile values are violated by the bays loaded at the lower rates. Since the poor

effluent colour mainly occurred during the first three and the last five weeks of the trials (i.e. Figure 5.10), any additional treatment or innovative operation of the grass filtration system for colour improvement during these periods will improve the colour.

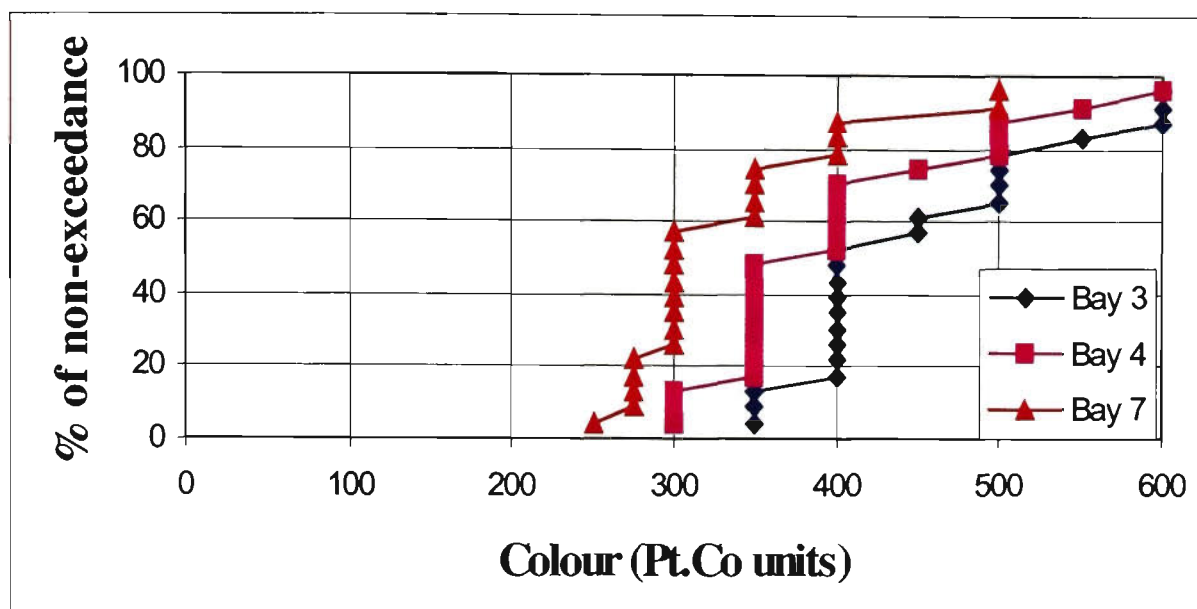


Figure 5.31 Cumulative Non-Exceedance Probability Distribution of Effluent Colour Concentrations for Bays 3, 4, and 7

5.8.5 $\text{NH}_3\text{-N}$

Figure 5.32 and Table 5.14 show that $\text{NH}_3\text{-N}$ concentration limits of the licence can be achieved at the 30 mm/d hydraulic loading rate, but cannot quite be met for the higher loaded bays. The 40 mm/d loaded bay violates only the maximum concentration standard by 2 mg/l, while the 50 mm/d loaded bay exceeds both the median and maximum concentration limits by 3 and 5 mg/l respectively (see Table 5.14).

The poor $\text{NH}_3\text{-N}$ removal performance of the bays occurred during the second half of the trial period. Therefore, if appropriate operational steps can be taken to prevent poor performance of the proposed full-scale grass filtration operations during the second half of the winter period, the $\text{NH}_3\text{-N}$ concentration standard can be achieved,

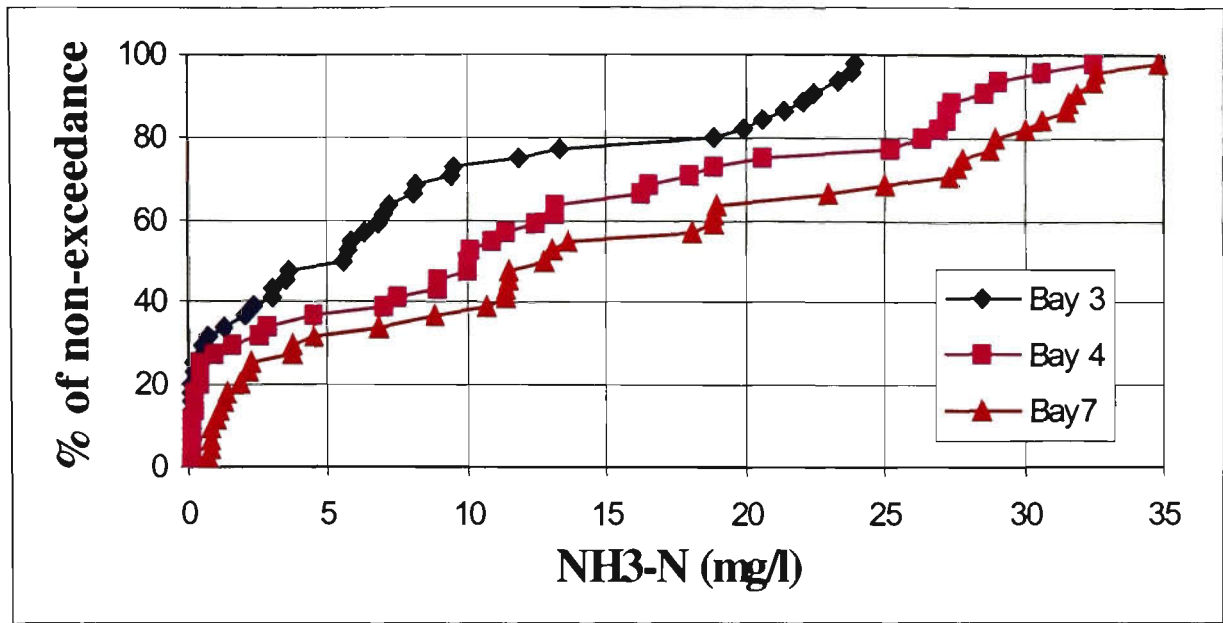


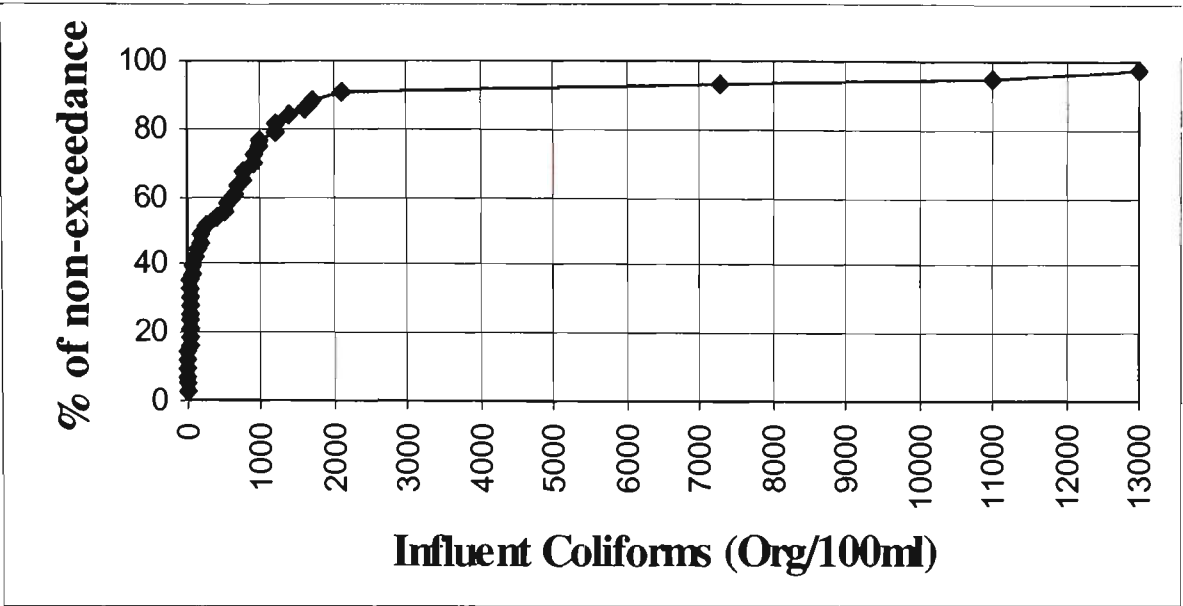
Figure 5.32 Cumulative Non-Exceedance Probability Distribution of Effluent NH₃-N Concentrations for Bays 3, 4, and 7

even at the 50 mm/d loading rate. Such steps may include slightly reducing the hydraulic loading rate and increasing the land area used for grass filtration treatment, and ensuring higher DO concentrations of influent during the second half of the winter period.

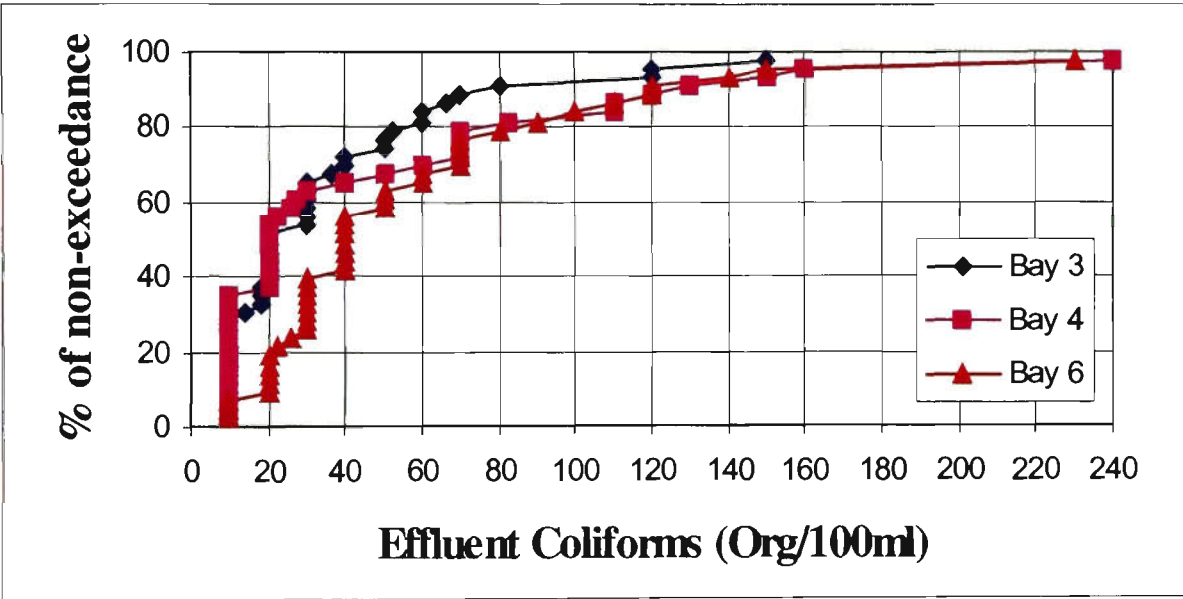
5.8.6 Faecal Coliforms

The percentage of non-exceedance Coliform concentrations at the inlet and outlet are shown in Figure 5.33. According to these plots, 50% of the influent coliforms sample had concentrations less than 200 Org/100ml. However, 50% of the effluent coliform samples had concentrations less than 20 Org/100ml in bays 3 and 4, while bay 6 had less than 40 mg/l.

Results obtained from the trials indicate that this bacteriological parameter standard will be met at all loading rates used for the trials since no effluent samples recorded concentrations above 250 Org/100ml, even when influent concentrations were as high as 13,000 Org/100ml.



(a) Influent Coliform Concentrations



(b) Effluent Coliform Concentrations

Figure 5.33 Non-Exceedance Distribution of Influent and Effluent Coliforms Concentrations

5.8.7 Annual Nitrogen Load

The EPA licence after the year 2001 limits the total annual load of TN to Port Phillip Bay at 3500 tonnes. Therefore, it is necessary to compute the total TN load during winter discharged to Port Phillip Bay at different hydraulic loading rates corresponding to typical volume of wastewater treated at the WTP. This will provide information on the TN load discharged to Port Phillip Bay at different hydraulic

loading rates, if grass filtration is practiced as a tertiary treatment during winter to treat all wastewater at WTP.

Using an average flow rate of 500 ML/day (i.e. typical volume of wastewater treated at the WTP per day), a grass filtration period of 150 days (i.e. duration of the trials) and the average TN export rates of 3.5, 7.2, and 12.4 kg/ha/day for bays 3, 4, and 7, respectively (Figure 5.19), the corresponding TN loads discharged are estimated as 875, 1350, and 1860 tonnes.

Sample calculation for bay 3 is given below:

Bay flow (hydraulic loading rate of 30 mm/d and surface area of 14,510 m ²)	=	$\frac{30 \times 14,510}{1000}$	m ³ /d
	=	435,500	l/d
TN load discharged out of the bay	=	3.5 x 1.4516	kg/d
	=	5.08	kg/d

Therefore, 5.08 kg/d TN is discharged for a flow of 435,500 l/d.

Total TN load that will be discharged from WTP, if grass filtration bays were operated with 30 mm/d	=	5.08 x 500x10 ⁶	kg
	=	5.83	tonnes/d

Grass filtration bays were operated for 150 days.

Therefore, TN load that will be discharged during winter (i.e. during trials)	=	5.83 x 150	tonnes
	=	875	tonnes

This load is equivalent to about 25% of the annual licence. These calculations assume that there is sufficient land area available for grass filtration. The above discharge rates represent 25% (for the 30 mm/d loading), 39% (for the 40 mm/d loading), and

53% (for the 50 mm/d loading) of the annual TN load of 3500 tonnes allowed by the EPA licence after the year 2001 (see Table 5.14). These discharge rates are based on 150 days which represent 41% of the year. In summer, generally land filtration is practised at WTP. With land filtration, most nutrients are absorbed by the soil. Therefore, TN load discharged to Port Phillip Bay is relatively low in summer. Hence, it can be assumed that the licence requirement can be met even at the highest loading rate of 50 mm/d based on the results of the trials.

5.9 OPTIMAL HYDRAULIC LOADING RATE

One of the major aims of this project was to determine the optimal hydraulic loading rate for the grass filtration trial bays of the WTP. Analysis of the data based on nutrients showed that the hydraulic loading rate of the bays can have a significant effect on the nutrient removal capacity. Generally, it was observed that the bays with higher hydraulic loading removed lower amounts of nutrients, while the bays with lower hydraulic loading removed the highest amounts of nutrients from the wastewater. However, with lower hydraulic loading rates (although the nutrient removal efficiency is higher), the amount of wastewater treated is less, requiring larger grass filtration areas.

Therefore, to reach a good balance where maximum nutrients can be removed at the highest possible hydraulic loading, an optimal hydraulic loading was required. The range of hydraulic loadings from which an optimal hydraulic loading rate could be derived in this study was from 30 to 50 mm/d at the intervals of 10 mm/d (i.e. 30, 40 and 50 mm/d). The optimal hydraulic loading rate certainly cannot be the highest loading rate (i.e. 50 mm/d), since the nutrient removal efficiency at this loading rate was found to be low, although the treated flow was the highest. On the other hand, the bay with the lowest hydraulic loading rate (i.e. 30 mm/d) achieved the best nutrient removal. However, at this hydraulic loading, the treated flow still remains low. Therefore, the optimal hydraulic loading which produces an optimal balance between both high flow and high nutrient removal efficiency is somewhere between these two extreme cases. In this case, it is the hydraulic loading rate of 40 mm/d, since it is the only hydraulic loading rate that was considered in this trial between 30 and 50 mm/d.

Although the measurements were conducted on bay 1 which had a hydraulic loading rate of 20 mm/d, it was not considered in detailed analysis, since the absolute nutrient loads in kg/ha/d removed through this bay were relatively small.

Since the study was mainly concentrated on removal of nutrients, the results produced in Tables 5.9 and 5.13 were investigated for the performance of bay 4 (with hydraulic loading rate of 40 mm/d) on absolute nutrient removal. As can be seen from these tables, the absolute removal of TN and NH₃-N is highest in bay 4, while OP-P and TP removals were also fairly high (although not the highest).

At the optimal hydraulic loading rate of 40 mm/d, only parameter that consistently violated the EPA licence was colour (Table 5.14). NH₃-N was violated only at the maximum level by 2 mg/l. The winter TN load discharged to Port Phillip Bay based on daily flow at WTP was 39% of annual licence, at the hydraulic loading rate of 40 mm/d. Based on the concentrations, the TN removal efficiencies of the 40 mm/d bay (i.e. bay 4) were 61 and 34% during Periods 1 and 2 of the trials respectively, while the TP removal efficiencies were 60 and -26% during Periods 1 and 2 respectively. The TN removal efficiencies based on mass balances for bay 4 were, 70 and 44% during Periods 1 and 2 respectively and the TP removal efficiencies were 60 and 4% during Periods 1 and 2 respectively. As can be seen from these figures, nutrient removal is high during Period 1 of the trials compared to Period 2. However, the total TN load discharged to Port Phillip Bay based on daily flow at WTP is within limits of EPA licence.

5.10 CONCLUSIONS

Physical, bacteriological and chemical water quality parameters including nutrients were monitored during this project. Water quality samples were taken at regular time intervals at the inlet and outlet of bays, and in some cases within the bays. The samples were scientifically analysed at the Water Ecoscience laboratories in Melbourne, Australia. Some water quality parameters were measured in-situ using a multifunction pH meter. The data were analysed for their temporal trends, and mass balances were carried out to compute the removal efficiency of nutrients.

The results were analysed in detail and reported for three representative bays (i.e. bays 3, 4 and 7) which had hydraulic loading rates of 30, 40 and 50 mm/d respectively. Two periods (i.e. first and second half of the trials) each with almost the same durations were seen to produce distinctively different removal efficiencies.

The grass filtration bays were more efficient in removal of BOD₅, CBOD₅ and CBOD(Filt) during Period 1 of the trials compared to the Period 2. This was mainly due to the lower influent concentrations of these parameters during Period 1. However, this was not the case for DO. The low influent DO did not have much effect on the effluent DO concentrations. Generally, the variations in the DO levels within the bays were due to photosynthetic activity of the grass through the root zones. The variations in the DO concentrations within the bays also affected the nitrogen removals. At low DO levels, the nitrogen removals occurred through nitrification and transformation of nitrogen to gases.

Evaluation of bay performances in TN removal suggest that the TN removal capacity of the bays is inversely related to the hydraulic loading rate. Within the bays, nitrification/denitrification was identified as the major nitrogen removal mechanism. This is because the detailed analyses of all 3 representative bays showed that more than 60% of the total nitrogen was removed by nitrification/denitrification as opposed to plant uptake, volatilisation and infiltration/cross flows/sedimentation. Some of the nitrification occurred within the bays by the release of DO through the root zone interface. However, it is believed that some nitrification can still occur at even low DO conditions.

Phosphorus did not show a consistent temporal pattern in terms of removal. Removal of phosphorus only occurred during the early stages of the trials, while the remainder of the trial period showed phosphorus production. Most of the phosphorus production was probably due wash-off of dead micro-organisms and decomposition of organic matter. The lack of reduction of phosphorus, however, was due to inability of plants to uptake phosphorus. It was found that the removal efficiency of phosphorus via Italian Ryegrass decreased as the season progressed due to ageing of the vegetation. It was also found that phosphorus removal was inversely related to hydraulic loading and was significantly affected by the detention time of the bay.

Therefore, the lower hydraulic loading gives a longer duration, which produces higher removal efficiencies. This is also the case with nitrogen removal. The longer detention of the bay provides longer contact time between the wastewater and the soil matrix, as well as the wastewater and the vegetation, which enables some of the phosphorus to be absorbed by the soil matrix and some taken up by the vegetation. Due to the ageing of the vegetation its phosphorus uptake capacity is lowered, leaving the soil as the next available medium to uptake the phosphorus from the wastewater. Furthermore, if the site has been used for wastewater irrigation for many years, then phosphorus deposits from the previous seasons may interfere with this causing a deposit of phosphorus in the wastewater.

A comparison of the trial results with the allowable EPA discharge licence limits showed that among the water quality parameters measured, effluent colour was the only parameter which did not comply with the discharge limits consistently. It was also found that in some cases the effluent from the bays with lower hydraulic loading rates was higher in colour than the influent. This was mainly due to the longer detention time of the bay, which allowed the wastewater to pick up more colour producing acids. The effluent concentrations of other parameters, including those of nitrogen and phosphorus except for $\text{NH}_3\text{-N}$ at higher hydraulic loading rates were found to be in compliance with the EPA discharge licence limits.

Based on the results of this trial study, and especially those of the representative bays, the hydraulic loading rate of 40 mm/d was chosen as the optimal hydraulic loading rate for the bays. At this hydraulic loading rate, high nutrient removal efficiencies could be achieved while discharging higher flows of wastewater. It was also found that at this hydraulic loading rate the absolute removal of nitrogen is higher. Comparison of nutrient removal during Period 1 and 2 shows that the removal rate is significantly higher in Period 1 compared to Period 2. Although the TN removal in Period 2 of the trial was lower, the total TN load discharged to Port Phillip Bay based on daily flow at WTP is within limits of EPA licence.

CHAPTER 6

MODELLING OF NUTRIENT REMOVAL

6.1 INTRODUCTION

In many engineering applications, models are used to investigate or predict the behaviour of a prototype system under various inputs. They are used where tests cannot be performed on the prototype mainly due to cost considerations. The major types of models currently used are physical and mathematical. Mathematical modelling basically involves the development of one or more descriptive equations based on a theoretical analysis of a problem, calibration and validation of these equations using field data obtained from surveys or other sources, and then use of these equations as a tool to predict the behaviour of a system under various design inputs.

Mathematical modelling has been used for many applications in hydraulic and environmental engineering, including several relating to wastewater treatment. These include models of unit processes involved in various forms of treatment including sludge management and effluent polishing. Generally, in wastewater treatment even on a small scale, the proper management of processes involved is very difficult. There are a number of quality criteria to be considered and in most cases the level of each criterion is the result of complex interactions. Furthermore, the situation is made more difficult when experimental approaches are used to forecast the quality of effluent wastewater. Therefore, mathematical models have become popular in predicting effluent quality of wastewaters (James, 1984).

The use of mathematical models can be helpful in predicting the nutrient concentrations of either the influent or the effluent of the grass filtration process. Grass filtration, as used in this study, is a polishing process in which further reductions of nutrient concentrations are obtained, and can be considered as a tertiary treatment process (details given in Section 2.3.1.2). The influent concentrations are usually known as they are effectively the same as the effluent concentration levels of the secondary treatment process. If a particular concentration level of the nutrients in the effluent is required (i.e. by EPA licence), a simple mathematical model can be used to predict the actual concentration which can then be compared with the required concentration. The model could also be used in a manner where the influent concentrations of nutrients can be determined from effluent concentrations specified by the authorities. Therefore, once the influent concentrations are obtained corresponding to the desired effluent concentrations, the required improvements to the secondary processes (i.e. lagooning processes) upstream can be studied.

There are two types of models which can be applied to any grass filtration process. These are models derived from first-order plug flow kinetics and simple regression based models. The first-order plug flow kinetic models are generally applicable to any process which involves nutrient reduction or removal (Reed et al., 1995). The parameters of the models are derived for specific sites using their data. Similarly, the simple regression models are based on regressions of influent and effluent concentrations and again developed for a site, using data at the site. Therefore, both of these models are only applicable to sites with the same or similar conditions to the study site on which the model constants are based. The literature in relation to both types of models is reviewed in Section 2.8, showing attributes and deficiencies of each model.

This chapter deals mainly with the derivation and validation of mathematical models (both reaction kinetics and regression models) for nutrient removal capabilities of the trial grass filtration bays at the WTP, with a particular focus on nitrogen and phosphorus, since these were identified as the major nutrients causing eutrophication of Port Phillip Bay (CSIRO, 1994). The data obtained from the trial site at the WTP are used to derive the site-specific constants of these mathematical models. Since the purpose of this chapter is to demonstrate the applicability of these two types of models

for WTP trial bays, only TN and TP were considered. Similar models can be developed for other forms of nitrogen and phosphorus, which involve decay (or removal).

6.2 MODELLING OF WTP TRIAL BAYS

6.2.1 Assumptions and Theories Used for Modelling

As explained in Sections 5.3, bays 3, 4 and 7 were considered as the representative bays, since they were the best performing bays in terms of reduced losses and representative of different discharges. Models are presented in this chapter only for the representative bays. The data from bay 3 was used to determine the parameters of the model since it was the best performing bay and suffered from minimal hydraulic losses (see Table 4.4). These model parameters were used for bays 4 and 7, to demonstrate the validity of the model. Due to the distinct differences in performance of the bays during Periods 1 and 2 in removal of TN and TP (see Section 5.2.2) development of separate models was attempted for each period. However, it was found that both types of models produced low correlation coefficients. In case of Period 2, the correlation coefficients related to first order kinetics models were as low as 0.05 and 0.006 for TN and TP respectively. The corresponding values for simple regression models were 0.005 and 0.0001. Therefore, based on low correlation coefficients, these models do not have adequate predictive capacity.

The outlet temperature values were only available for bay 7. Therefore, it was assumed that the effluent temperatures at the outlets of other bays (i.e. bays 3 and 4) were the same as bay 7 effluent temperature. These temperatures were used in the first-order reaction kinetics models, since the decay coefficients depend on temperature.

6.2.2 First-Order Reaction Kinetics Model

6.2.2.1 TN Removal Model

Bay 3 had a detention time of 4.08 days (see Table 4.8). Therefore, the observed effluent data were lagged by 4.08 days using linear interpolation techniques to give accurate match of correspondent effluent to influent. Throughout the trial study at the WTP, the TN data consisted of 43 measurements (i.e. twice weekly measurements for 22 weeks with the exception of week 1 which only had one sampling day). However, since only Period 1 was considered for nutrient modelling, this period consisted of 20 data points, which were used in modelling.

The standard first-order reaction kinetics model as given in Equation (6.1) is used.

$$C_e / C_o = \exp (-k_T t) \quad (6.1)$$

where C_e = effluent nutrient concentration, mg/l

C_o = influent nutrient concentration, mg/l

t = hydraulic residence time, d

k_T = temperature-dependent first-order reaction rate constant, d^{-1}

Re-arranging the above gives:

$$k_T = \ln (C_o / C_e) / t \quad (6.2)$$

The value of first-order reaction rate constant k_T is related to the wastewater temperature via the following formula:

$$k_T = k_{20} \theta^{(T-20)} \quad (6.3)$$

where k_{20} = 20°C first-order reaction rate constant, d^{-1}

θ = constant

T = wastewater temperature, °C

The parameters k_{20} and θ are generally constants for a particular quality of wastewater under a particular treatment. As can be seen from Equation (6.1) to (6.3), the model parameters are k_{20} and θ . To obtain k_{20} and θ , the logarithmic base of both sides of Equation (6.3) must be obtained. This is shown below:

$$\log k_T = \log k_{20} + T \log \theta - 20 \log \theta \tag{6.4}$$

Re-arranging Equation (6.4) gives:

$$\log k_T = T \log \theta + (\log k_{20} - 20 \log \theta) \tag{6.5}$$

For each pair of observed influent and corresponding 4.08-day lagged effluent concentrations of TN on bay 3, k_T was computed using Equation (6.2), and their logarithmic values were regressed against the average outlet temperature values. The outlet temperature was considered as a representative temperature for the bay. However, for each influent and corresponding effluent, the outlet temperature was computed as the value corresponding to the day which shows average conditions of the influent/effluent. The regression plot of TN of bay 3 for Period 1 is shown in Figure 6.1. As seen from Equation (6.5), the graph of $\log k_T$ versus T should be a straight line, although there is substantial scatter in Figure 6.1.

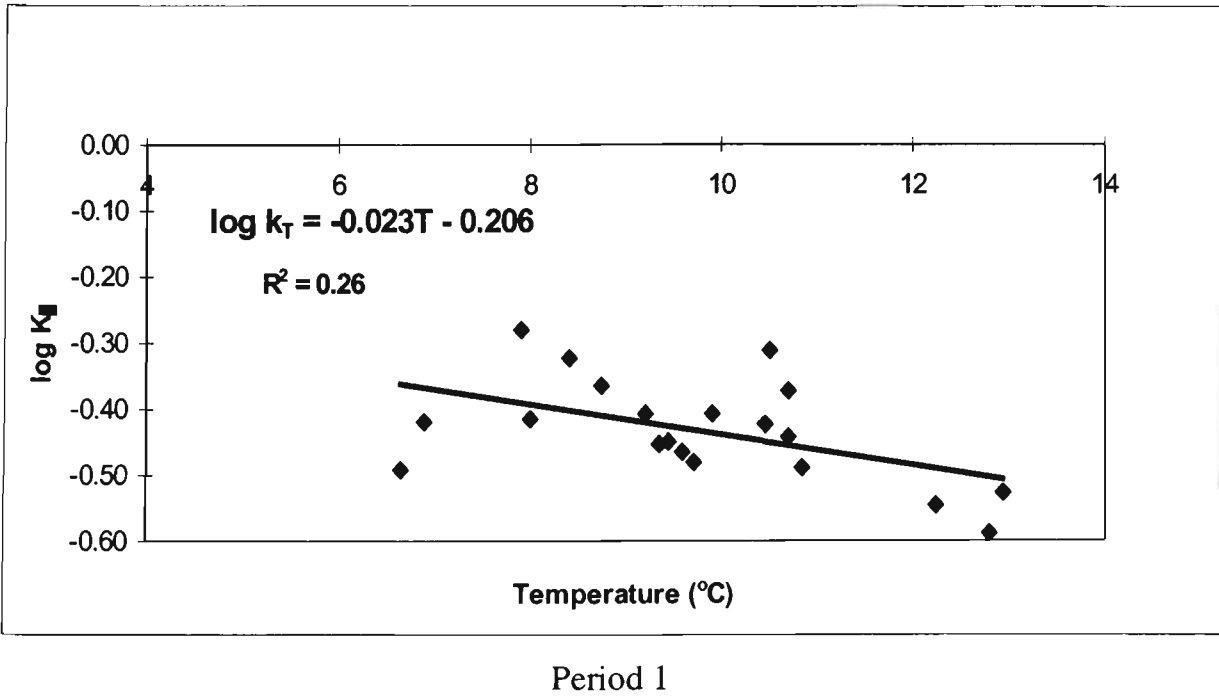


Figure 6.1 Regression Plots for First-Order Reaction Kinetics Model for TN

The line of best fit equations from the regression analysis (i.e. Figure 6.1) are as follows:

$$\log k_T = -0.023 T - 0.206$$

Period 1

(6.6)

Comparing Equation (6.6) with Equation (6.5), gives:

$$\log k_{20} - 20 \log \theta = -0.206$$

Period 1

(6.7)

Noting that $\log \theta = -0.023$, and then solving the above Equation gives θ and k_{20} values as shown in Table 6.1.

Table 6.1 θ and k_{20} Values for TN Model

	Period 1
θ	0.948
k_{20}	0.213

Therefore, the first-order reaction kinetics model for TN removal of bay 3 can be written as follows:

$$C_e / C_o = \exp (-k_T t)$$

(6.8)

where $k_T = 0.213 (0.948)^{(T-20)}$

Period 1

(6.9)

As can be seen from Figure 6.1, there is a fair amount of scatter in estimating k_T and a low correlation coefficient of 0.26. Therefore, it was necessary to study how Equation (6.9) performs as a predictive model. Equation (6.9) was used to compute k_T and then effluent concentrations for observed influent concentrations and outlet temperatures for bay 3. This was then compared with the observed effluent TN concentrations from the bay. The plot of calculated and observed TN effluent concentrations for bay 3 is shown in Figure 6.2.

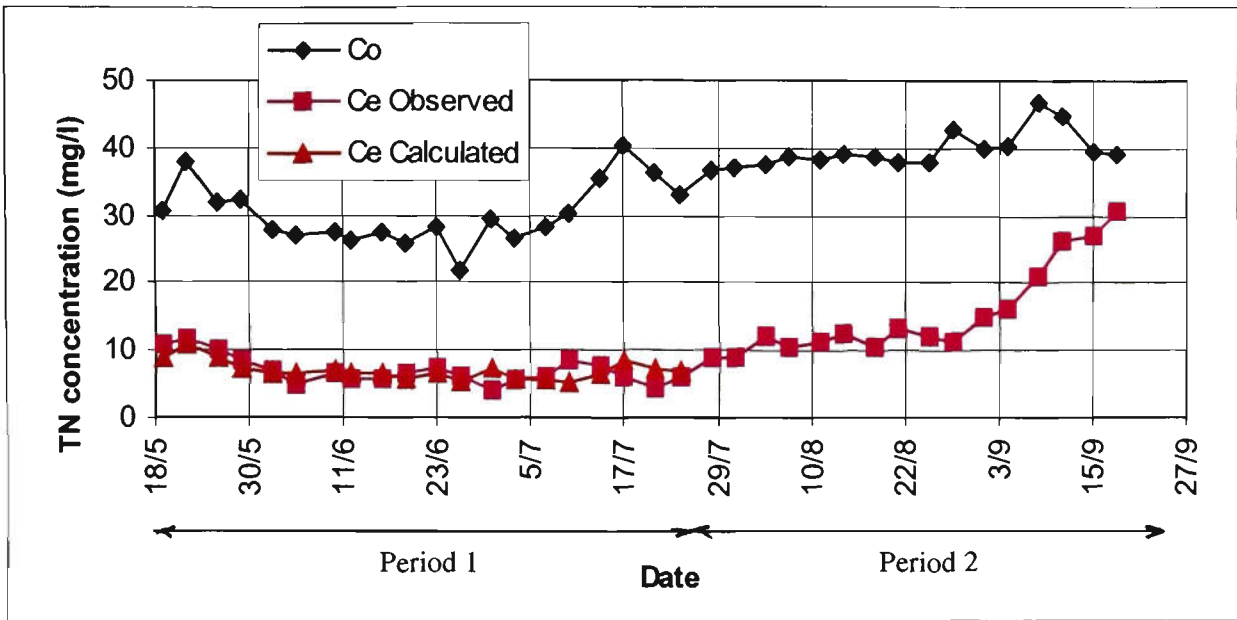


Figure 6.2 Observed and Calculated TN Effluent Concentrations (Using First-order Reaction Kinetics Model for Bay 3)

According to Figure 6.2, the observed and calculated values of the TN effluent are very closely matched. Since there was a fair amount of scatter in estimating k_T (Figure 6.1) with a low correlation coefficient, a good match might not necessarily be expected between observed and computed effluent concentrations. The calculated TN effluent concentrations are slightly less than the observed concentrations. The average under-prediction is 20%.

Once the TN removal model for bay 3 was developed (i.e. Equations 6.8 and 6.9), these θ and k_{20} values were used to compute the TN effluent concentrations of bays 4 and 7, to validate the model for the other bays of WTP. Appropriate detention times, as given in Table 4.8 were used in Equation (6.8), and also to get matching effluent concentrations for comparison with those computed. The plots of observed and calculated TN effluent concentrations for bays 4 and 7 are presented as Figures 6.3 and 6.4 respectively.

As it is evident from Figures 6.3 and 6.4, the calculated and observed values of effluent TN concentrations follow a similar pattern with slight spread between these two values during Period 1. This spread (or error) is not a result of error build up. Since each effluent value is computed based on its corresponding influent value, the build up of error in the results is not likely to occur.

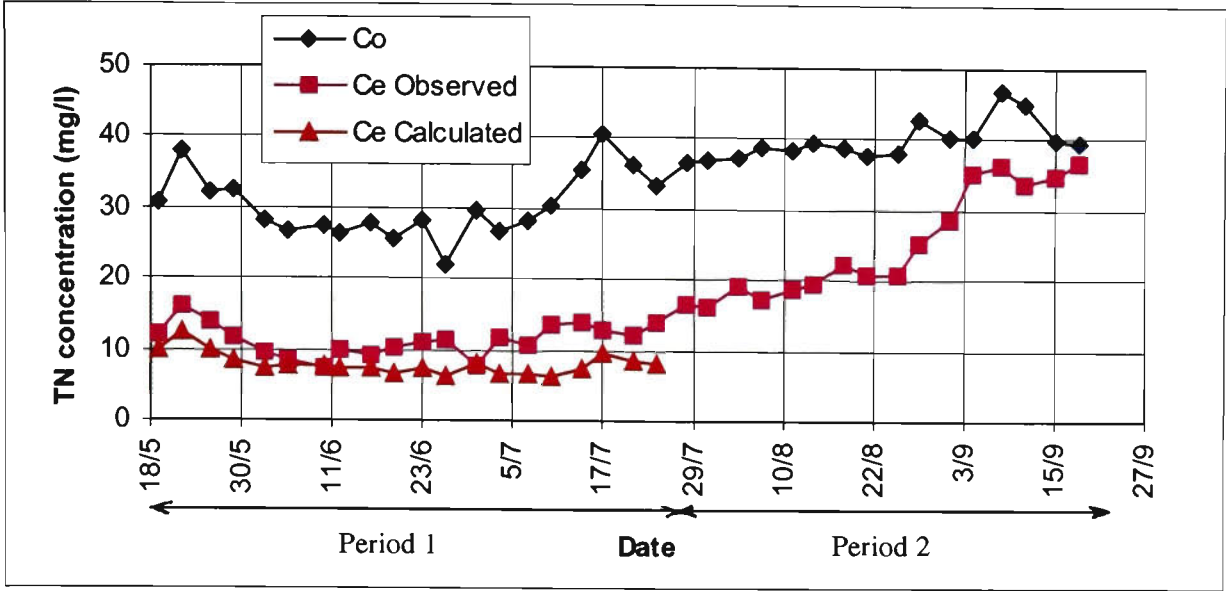


Figure 6.3 Observed and Calculated TN Effluent Concentrations (Using First-order Reaction Kinetics Model) for Bay 4

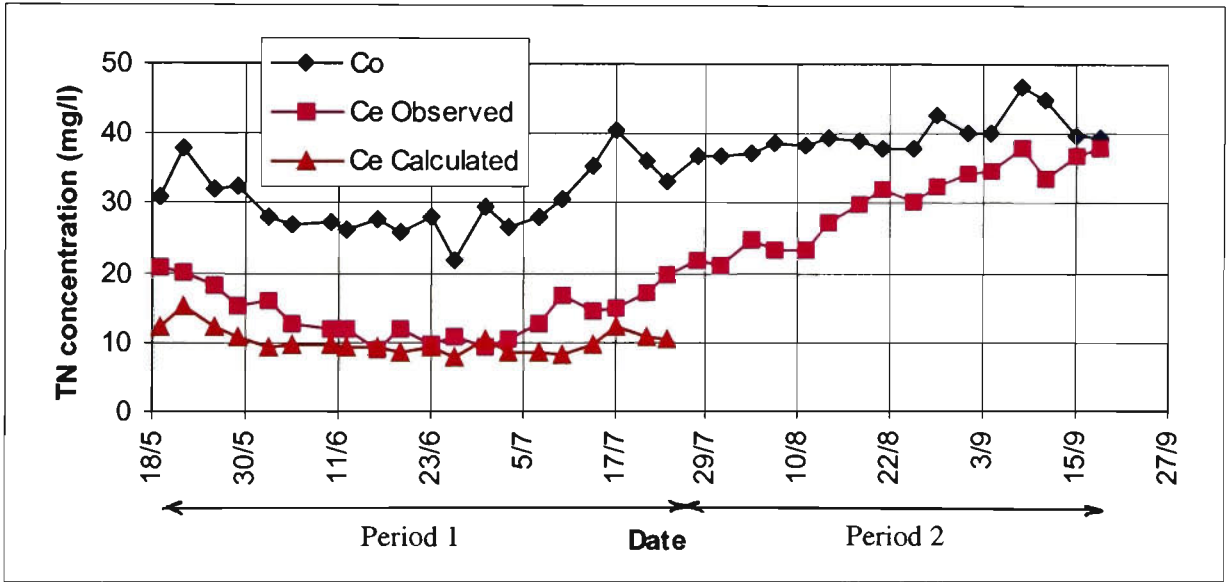


Figure 6.4 Observed and Calculated TN Effluent Concentrations (Using First-order Reaction Kinetics Model) for Bay 7

It must also be noted that the differences between the observed and calculated effluent concentrations of bays 4 and 7 are higher than those of bay 3. This is to be expected, since the model was calibrated for bay 3 and not for bays 4 and 7. One reason could be the higher hydraulic loading rates of bays 4 and 7 (i.e. 40 and 50 mm/d) and their influence on the model parameters. However, the hydraulic loading rates are implicitly taken into account, as the computations in the modelling process used the detention time of the bays. The other possible reason is different losses in different bays. The

effect of losses are also implicitly lumped into these parameters. Similar to bay 3, the calculated TN effluent concentrations are lower than the observed concentration for bays 4 and 7. The average under-prediction is 29 and 27% respectively.

6.2.2.2 TP Removal Model

In deriving the first-order reaction kinetics model for TP removal, the same modelling process as for TN was followed. Throughout the trial study at the WTP, which was conducted over 22 weeks, TP measurements were carried out once a week. Therefore, a total of 22 measured data prints were obtained for TP. However, as stated in Section 6.2.1, a TP removal model was developed only for Period 1 using bay 3 data. This period only had 9 data points. The TP removal model was also based on the basic form of the reaction kinetics as shown in Equation 6.10.

$$C_e / C_o = \exp (-k_T t)$$

Period 1

(6.10)

A similar procedure to that of Section 6.2.2.1 was followed. The resultant regression plots and equations are shown in Figure 6.5.

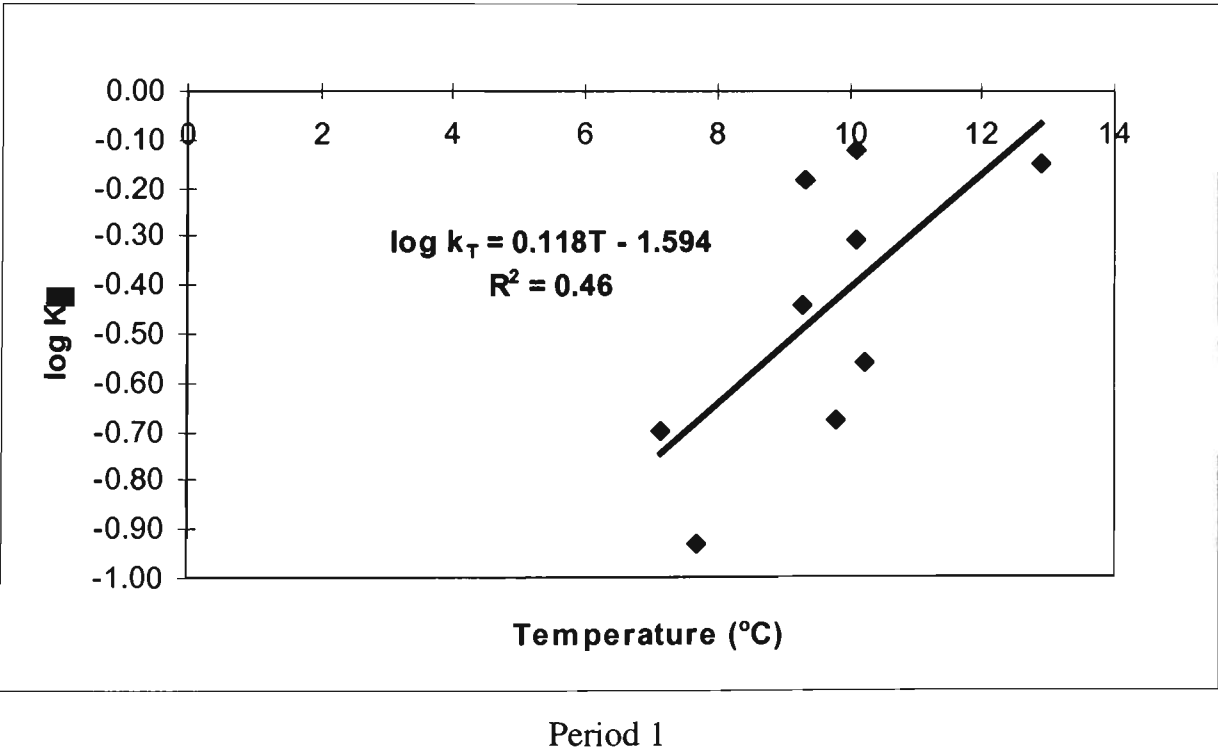


Figure 6.5 Regression Plots for First-Order Reaction Kinetics Model for TP

According to Figure 6.5, the regression equation is reproduced below:

$$\log k_T = 0.118T - 1.594$$

Period 1

(6.11)

Using the same procedure as outlined in Section 6.2.2.1, the θ and k_{20} values were obtained and are presented in Table 6.2:

Table 6.2 θ and k_{20} Values for TP Model

	Period 1
θ	1.314
k_{20}	5.962

Therefore, the TP removal model is as follows:

$$C_e / C_o = \exp (-k_T t)$$

(6.12)

$$k_T = 5.962 (1.314)^{(T-20)}$$

(6.13)

Similar to the TN model, the effluent concentrations were computed using Equations (6.12) and (6.13) with inputs of influent concentrations and effluent temperatures. These effluent concentrations were compared with the observed values, as shown in Figure 6.6. According to Figure 6.6, the observed and calculated values of TP show a close match, in spite of scatter of data points in Figure 6.5 and low R^2 .

After the TP removal model for Bay 3 was developed (i.e. Equations (6.12) and (6.13)), the θ and k_{20} values were then used to compute the TP effluent concentrations of bays 4 and 7 to validate the model for the other bays. To get the matching effluent concentrations for matching with the computed, the appropriate detention times as given in Table 4.8 were used in Equation (6.12). The plots of observed and calculated TP effluent concentrations for bays 3, 4 and 7 are presented in Figures 6.6, 6.7 and 6.8 respectively.

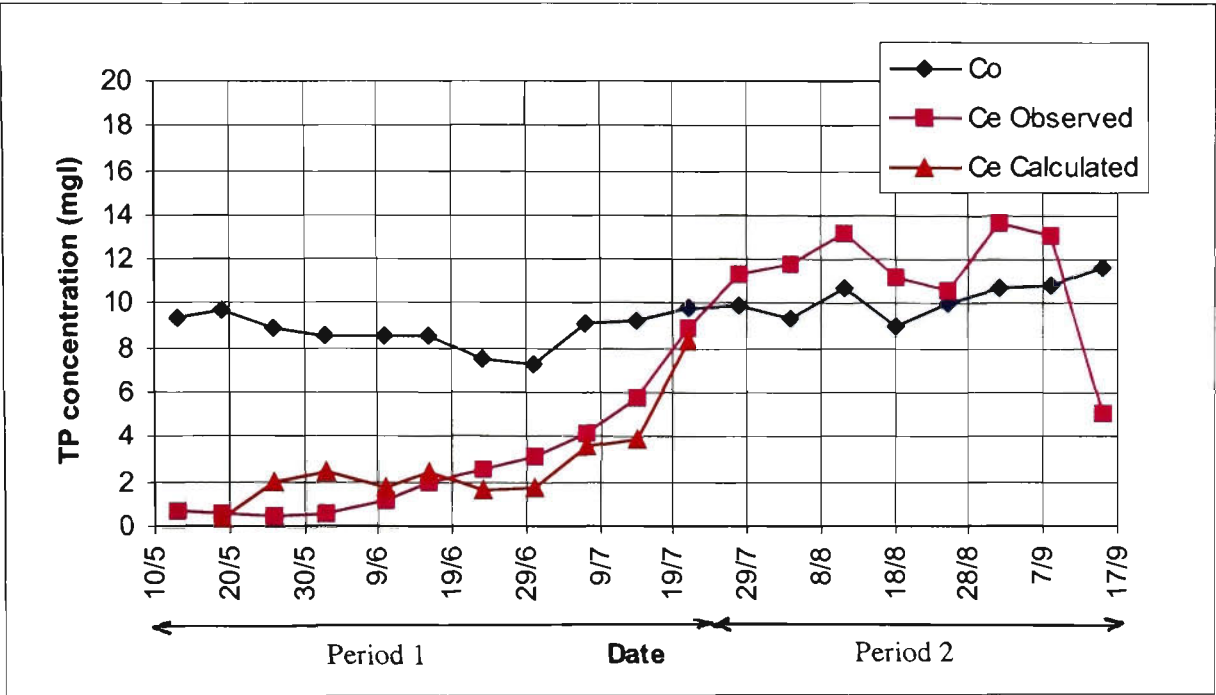


Figure 6.6 Observed and Calculated TP Effluent Concentrations (Using First-Order Reaction Kinetics Model) for Bay 3

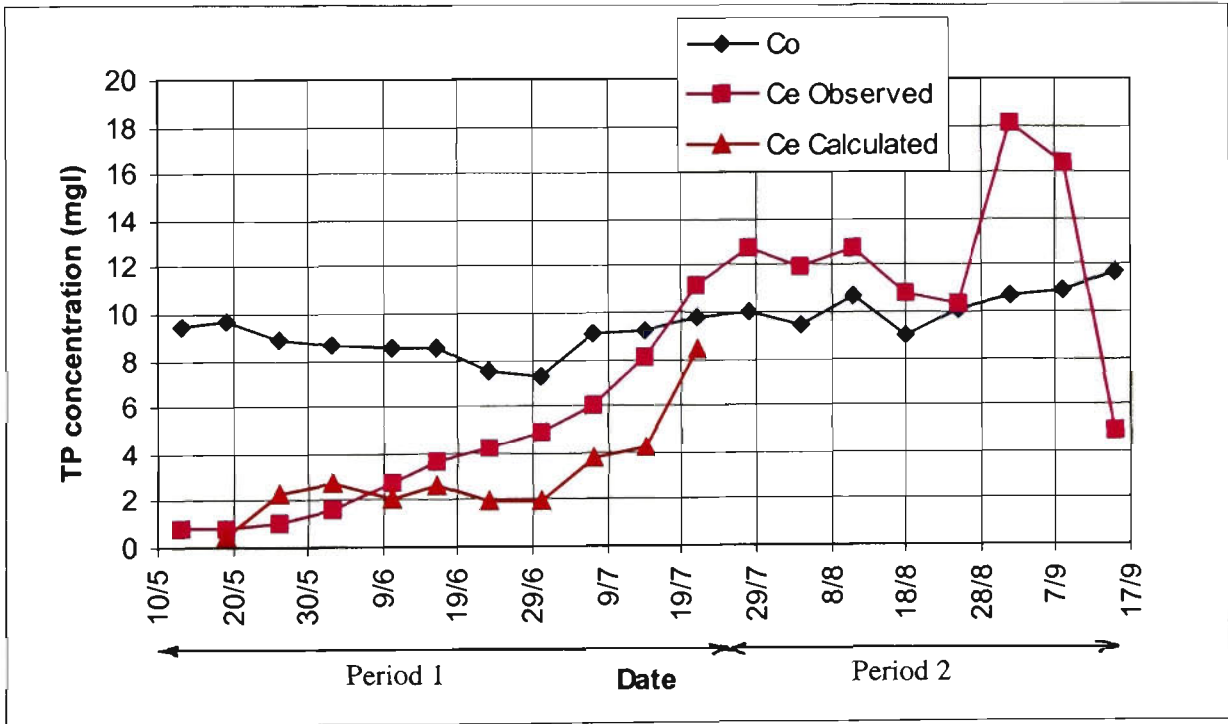


Figure 6.7 Observed and Calculated TP Effluent Concentrations (Using First-Order Reaction Kinetics Model) for Bay 4

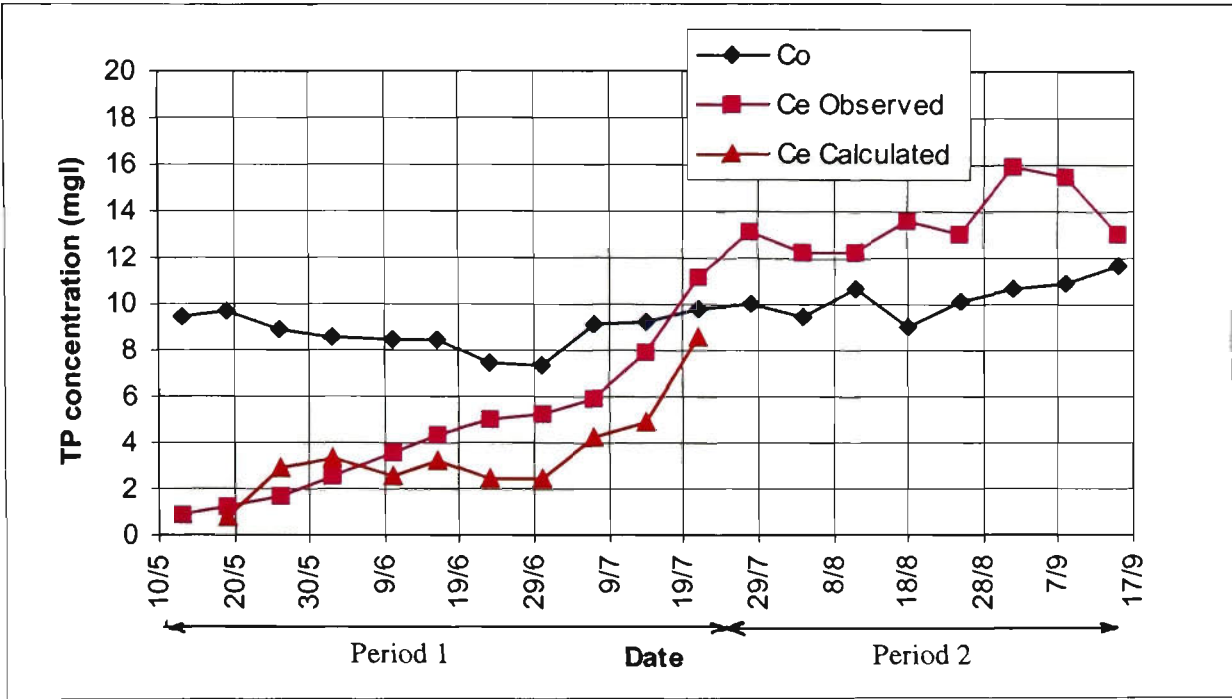


Figure 6.8 Observed and Calculated TP Effluent Concentrations (Using First-Order Reaction Kinetics Model) for Bay 7

According to Figure 6.7, the plots of observed and calculated effluent TP for bay 4 also show similar characteristics as that of bay 3 (i.e. Figure 6.6). From this figure, it can be observed that the computed TP values are lower than the observed TP values for most of the time during Period 1. Similar results can be seen for bay 7 in Figure 6.8. The average absolute errors in underestimating these values are 87, 52 and 39% respectively. It must be noted that, in some cases computed values were higher and in the other cases, they were lower.

6.2.3 Simple Regression Model

As described in Section 6.1, the simple regression model is based on regression of influent and effluent concentrations. Similar to Section 6.2.2, the data from bay 3 were used to develop the simple regression models for both TN and TP, considering matching pairs and neglecting the same data outliers. The model was only developed for Period 1 as in Section 6.2.2. The model was validated later using data from bays 4 and 7. The results and plots of the simple regression analysis for both TN and TP are presented in the following sections.

6.2.3.1 TN Removal Model

A regression plot of influent TN and corresponding effluent TN of bay 3 for Period 1 is shown in Figure 6.9. The TN regression model for Period 1 of bay 3 is extracted from Figure 6.9, as follows:

$$\text{TN (out)} = 0.15 \text{ TN (in)} + 2.36 \qquad R^2 = 0.11 \qquad \text{Period 1} \qquad (6.14)$$

where TN (out) = TN concentrations at the outlet, mg/l
TN (in) = TN concentrations at the inlet, mg/l

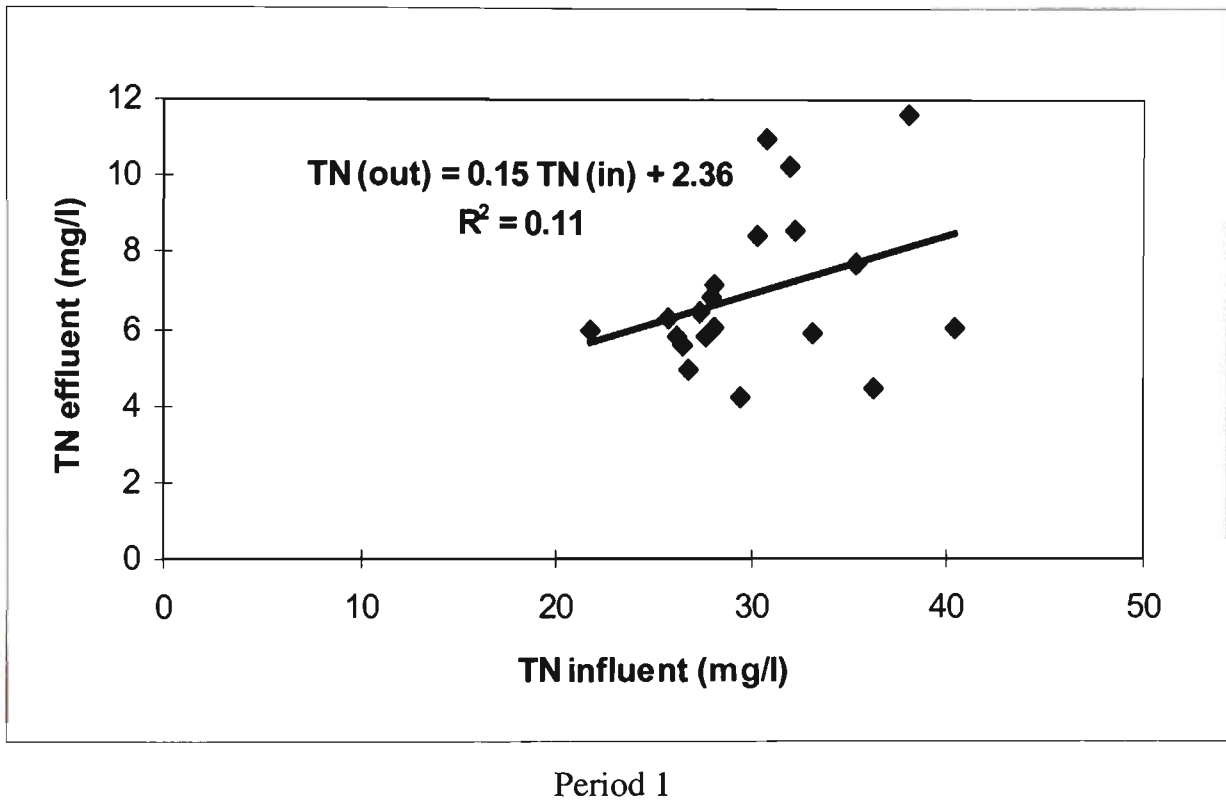


Figure 6.9 TN Regression Model for Bay 3

According to Figure 6.9, the influent and effluent values of TN show a wide scatter with a low R^2 . This may be due to the quality variability of the influent TN concentrations. Similar to Section 6.2.2.1, the effluent TN for all three bays were calculated using Equation (6.14) and the observed influent TN values. These observed and calculated values for all three bays are shown in Figures 6.10, 6.11 and 6.12.

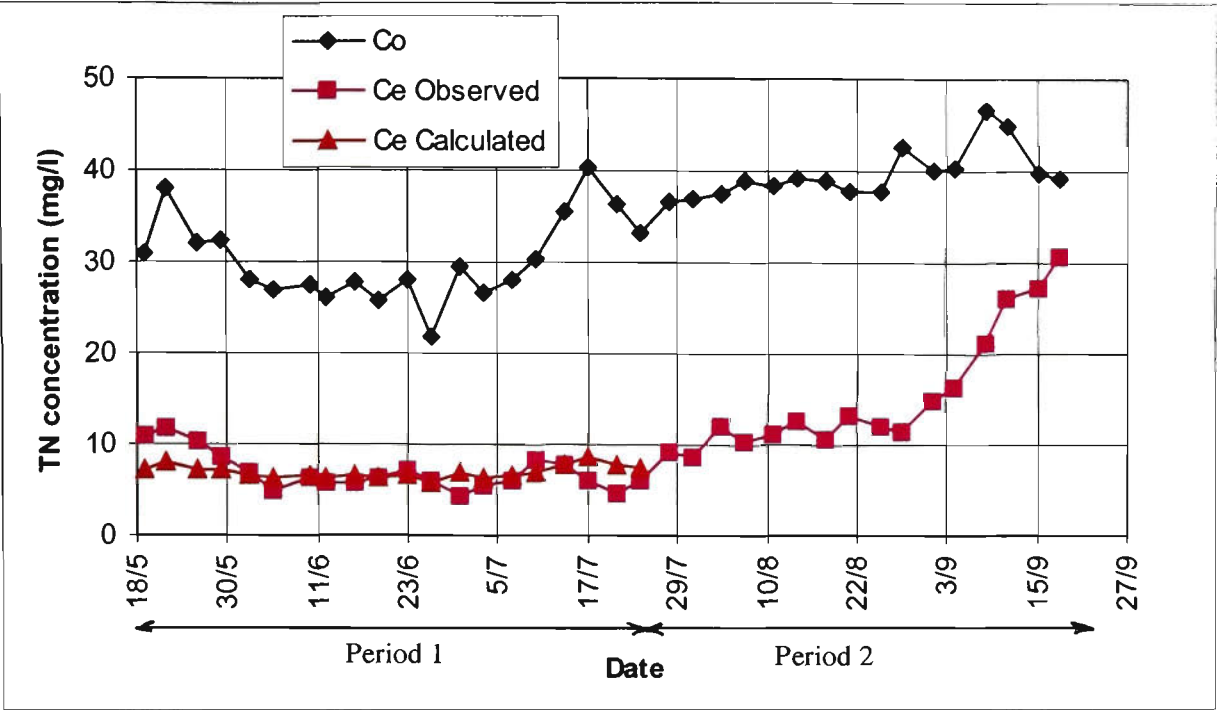


Figure 6.10 Observed and Calculated TN Effluent Concentrations (Using Simple Regression Model) for Bay 3

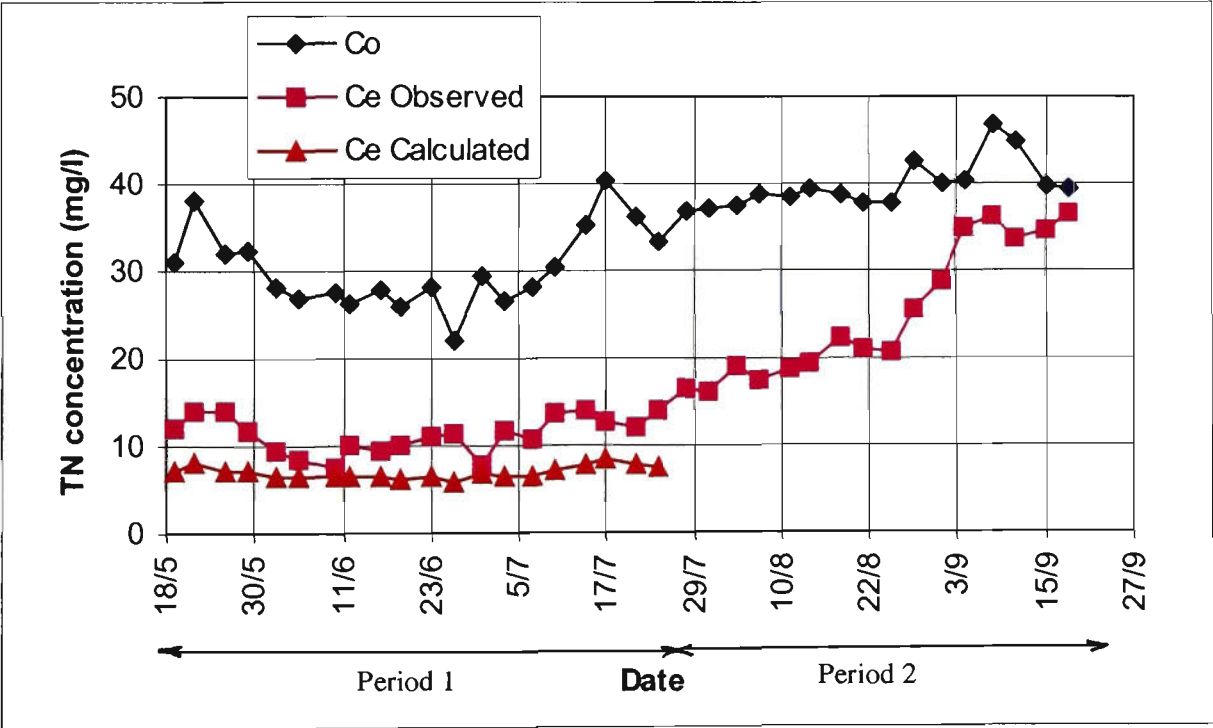


Figure 6.11 Observed and Calculated TN Effluent Concentrations (Using Simple Regression Model) for Bay 4

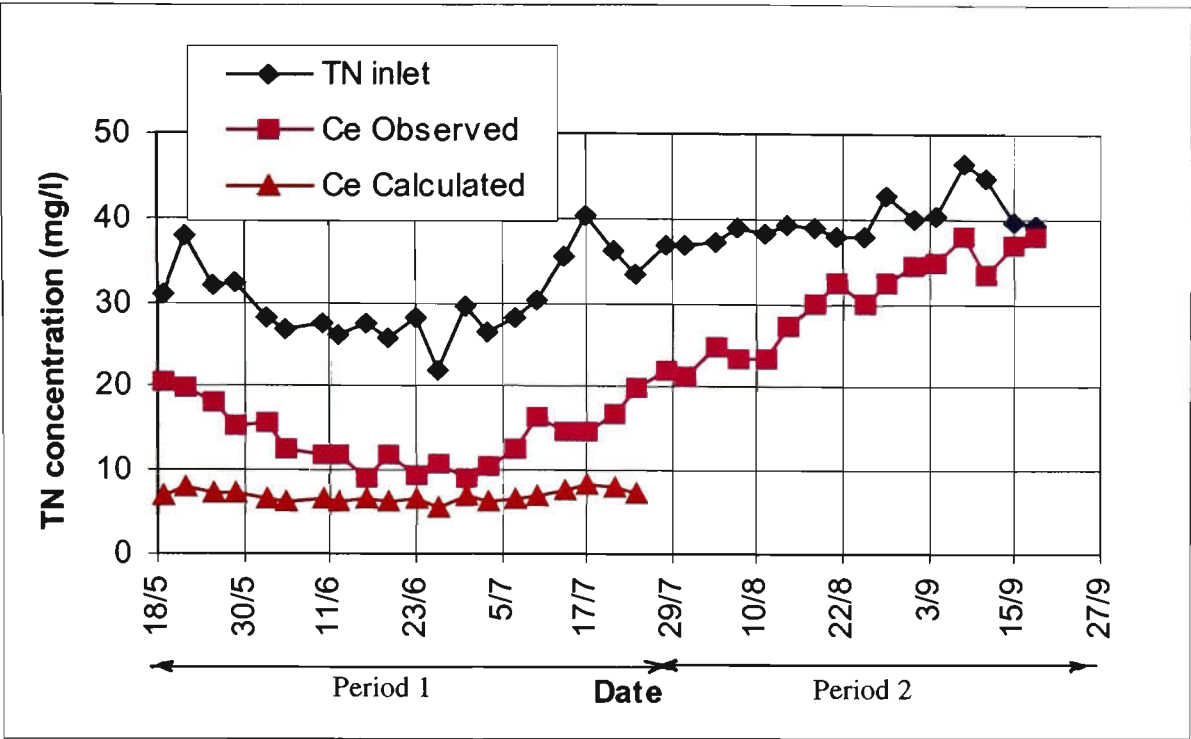
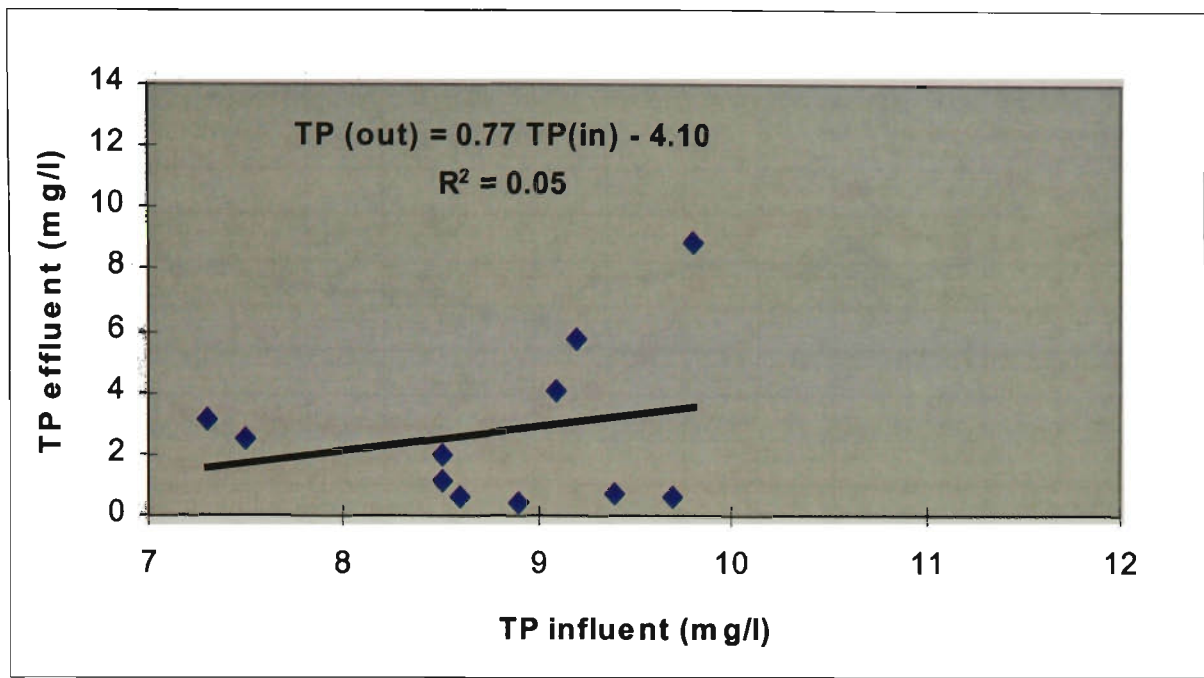


Figure 6.12 Observed and Calculated TN Effluent Concentrations (Using Simple Regression Model) for Bay 7

According to Figure 6.10, the observed and computed values for bay 3 are closely matched in spite of scatter in the regression plots of Figure 6.9 and a low correlation coefficient. A similar pattern can be seen for bays 4 and 7 between the calculated and observed TN effluent concentrations (Figures 6.11 and 6.12). However, the values of observed and calculated effluent TN are not as close as for bay 3. The possible reasons are again given under Section 6.2.2.1.

6.2.3.2 TP Removal Model

Similar to the TN removal model in Section 6.2.3.1, a TP removal model was developed using the linear regression method. The coefficients used for the TP model are again based on the bay 3 data. Similar to Section 6.2.2, the data from bay 3 were used to develop the simple regression model for TP, considering matching pairs. Only Period 1 was considered as in Section 6.2.2. The model was validated later using data from bays 4 and 7. The regression plot for Period 1 is presented in Figure 6.13.



Period 1

Figure 6.13 Regression Plots for Bay 3

From Figure 6.13, the TP model for bay 3 based on simple regression analysis is presented below for Period 1:

$$TP (out) = 0.77 TP (in) - 4.10$$

$$R^2 = 0.05$$

Period 1

(6.15)

Similar to the TN removal model in Section 6.2.3.1, the TP removal model defined by Equation (6.15) is used to calculate the effluent TP concentrations for bays 3, 4 and 7, and compared with the observed effluent concentration values. They are shown in Figures 6.14, 6.15 and 6.16 for bays 3, 4 and 7 respectively.

According to Figures 6.14 to 6.16, comparison of the observed and computed effluent TP concentrations of 3 bays are similar, as opposed to the performance of the TN removal model of Section 6.2.3.1. In these figures, the computed effluent concentrations of TP are lower than the influent TP concentrations during Period 1. However, the calculated values of TP are higher than the observed TP concentrations at some stages and lower than the observed concentrations at other stages of Period 1. Hence, the absolute average errors are 184, 111 and 81% for bays 3, 4 and 7

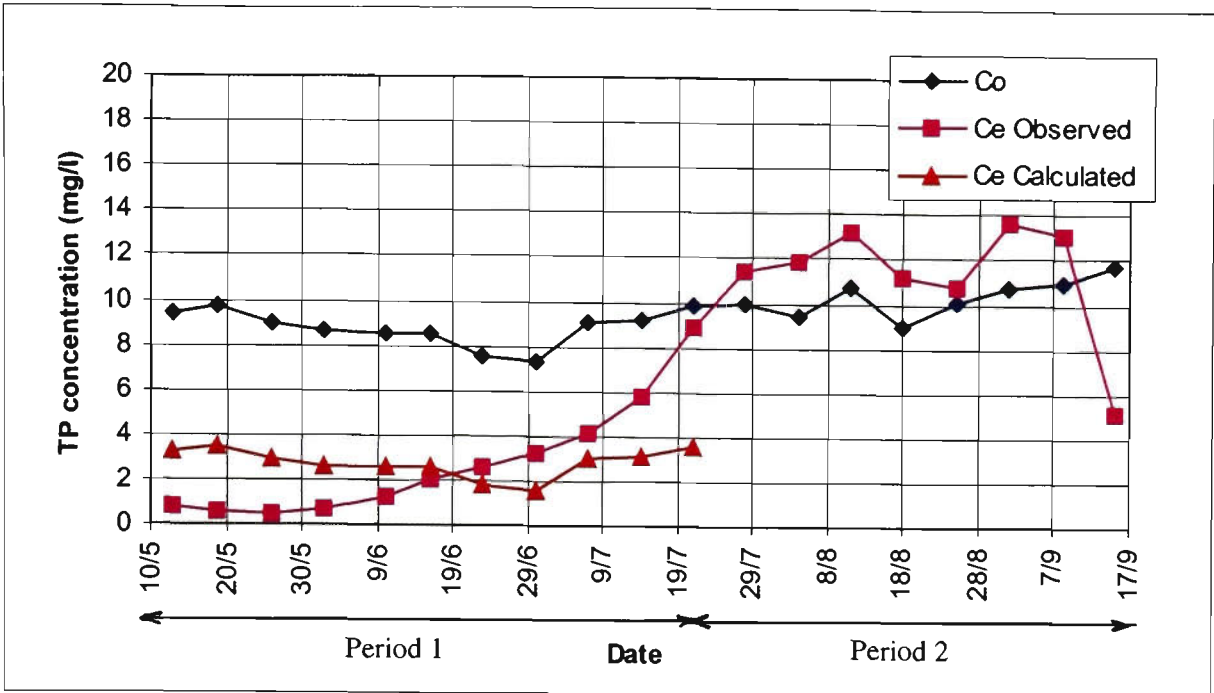


Figure 6.14 Observed and Calculated TP Effluent Concentrations (Using Simple Regression Model) for Bay 3

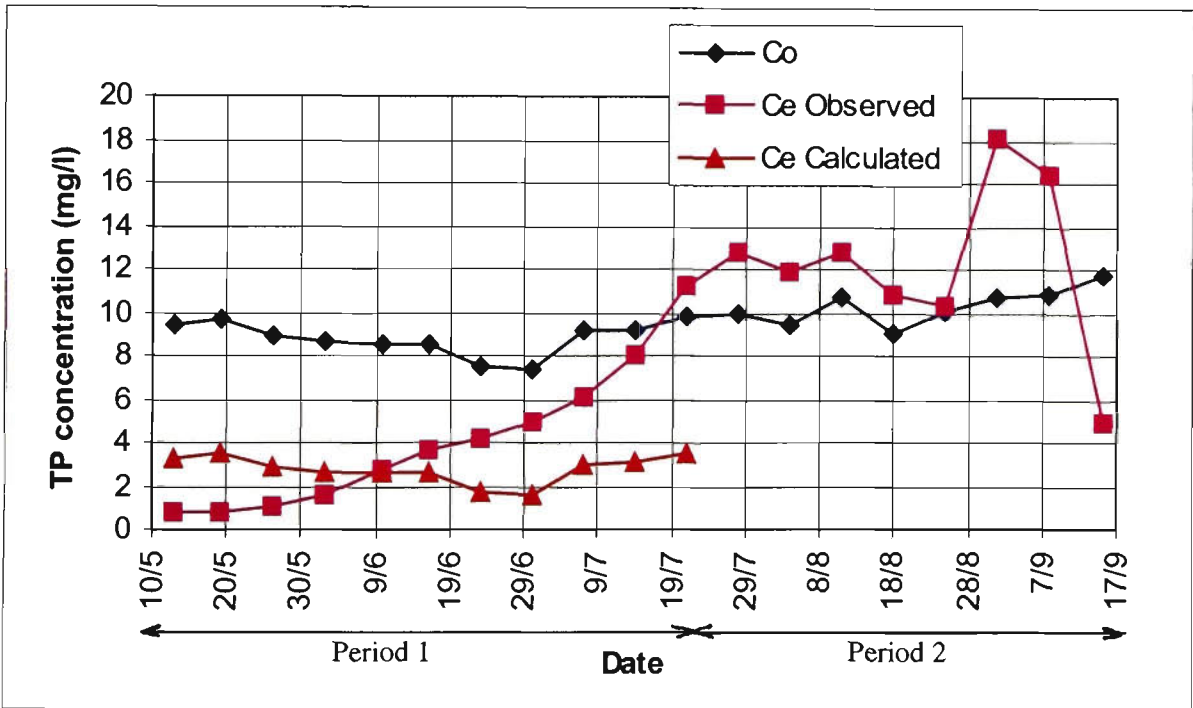


Figure 6.15 Observed and Calculated TP Effluent Concentrations (Using Simple Regression Model) for Bay 4

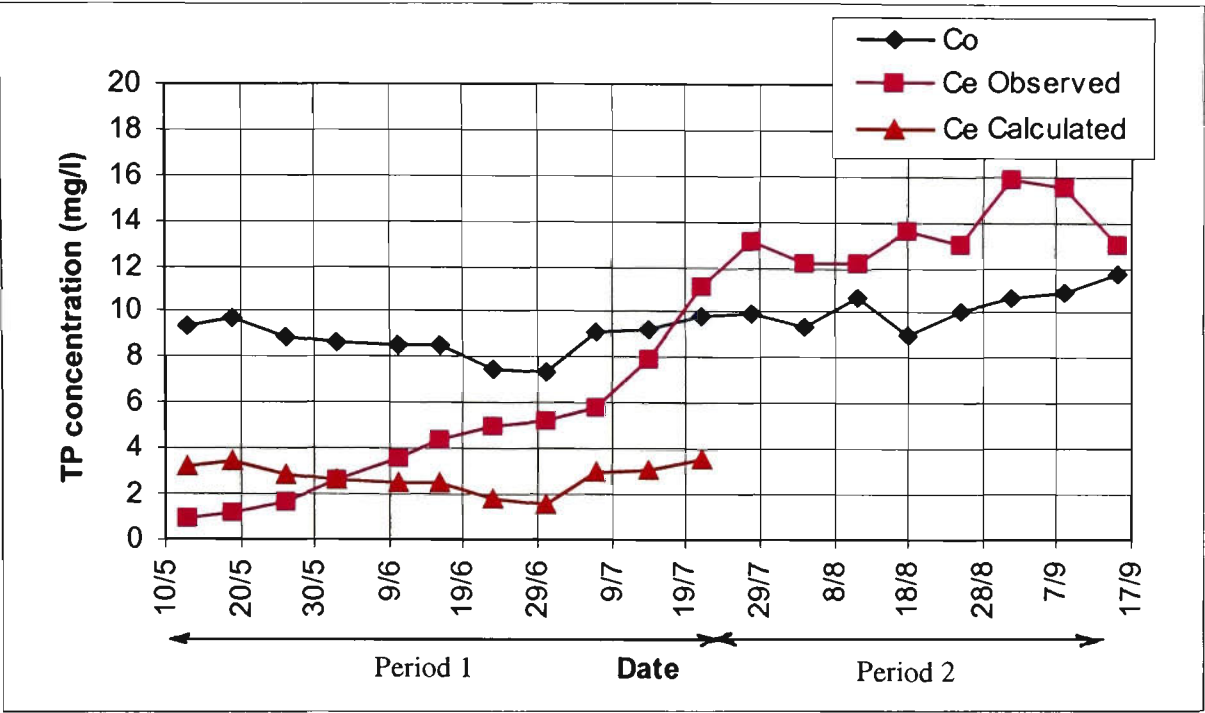


Figure 6.16 Observed and Calculated TP Effluent Concentrations (Using Simple Regression Model) for Bay 7

respectively. These errors seem quite high, however, these were to be expected since the regression resulted in very low correlation coefficient (i.e. $R^2 = 0.05$).

6.3 COMPARISON OF TWO TYPES OF MODELS

Two types of models, namely first-order reaction kinetics and simple regression models, were used for TN and TP modelling. Both models can be used for estimating the effluent concentrations based on the influent concentration, although the accuracy may not be very high. Therefore, to evaluate which model gives a better estimate of the effluent concentrations, they are compared on the basis of their correlation coefficients (i.e. the R^2 values) first, followed by a comparison of the absolute average errors in over-estimating and under-estimating of effluent concentrations and finally by the actual output from each model (i.e. the plots of observed and computed effluent concentrations). However, it must be noted that no comparisons of these models could be made with any other TN or TP removal models due to lack of available information on modelling work carried out for a site or conditions similar to those of the WTP trial bays.

6.3.1 TN Removal Models

The TN removal models are compared on the basis of their R^2 values and the model outputs. The R^2 value of $\log k_T$ vs temperature plots (i.e. Figure 6.1) for the reaction kinetics model was 0.26 for Period 1 for bay 3. For the same bay, an R^2 value of 0.11 was obtained for Period 1 using the simple regression model (i.e. Equation (6.14)). Therefore, on the basis of R^2 values, the reaction kinetics model seems to be more accurate in estimating TN effluent concentrations than the simple regression model.

To evaluate the accuracy of the two TN removal models (i.e. reaction kinetics and simple regression models), the average absolute errors in over-estimating and under-estimating of effluent TN concentrations using these models were also computed. Using the reaction kinetics model, these errors were 20, 29 and 27% for bays 3, 4 and 7 respectively. However, using the simple regression model, these errors were 21, 36 and 48% for the same bays. Based on these values, it can be said that the reaction kinetics model may result in more accurate estimates of the effluent TN concentrations than the simple regression model.

The plots of observed and computed TN obtained through the use of both models could also be compared to evaluate the accuracy of the two models. Considering TN computed through reaction kinetics model in bay 3 (i.e. Figure 6.2), and computed TN through simple regression model (i.e. Figure 6.10), Figure 6.2 shows much closer match between the observed and the computed values of effluent TN concentrations. Similarly, comparing Figures 6.3 and 6.4 (i.e. reaction kinetic models output for bays 4 and 7 respectively) and Figures 6.11 and 6.12 (i.e. simple regression models output for bays 4 and 7 respectively), Figures 6.3 and 6.4 show closer match between the observed and computed values of effluent TN concentrations. Therefore, on the basis of this comparison as well as the absolute average errors and the R^2 values, the reaction kinetics model seems to give better estimates of the TN effluent concentrations.

It should be noted that the models are limited to Period 1 and the accuracy that can be expected from the reaction kinetics model (which is the better model in this case) is of

the order of 28%. This is based on the results of error bands produced for bays 4 and 7, for which calibrations were not done.

6.3.2 TP Removal Model

Similar to the comparisons made in Section 6.3.1 above, the TP removal models are also compared on the basis of their R^2 values, absolute average errors, and model outputs.

The $\log k_T$ vs temperature plots (i.e. Figure 6.5) for the reaction kinetics model resulted in R^2 value of 0.46 for Period 1. For the same bay (i.e. bay 3), an R^2 value of 0.05 was obtained for Period 1 via the simple regression model (i.e. Equation (6.15) and (6.24)). Therefore, it is suggested that the reaction kinetics model gives better estimates of the TP effluent concentrations in comparison to the simple regression model.

A comparison of the absolute average errors in over and under-estimating of effluent TP concentrations using the reaction kinetics and simple regression models was also carried out to evaluate the efficiency of these models. Based on reaction kinetics model, the absolute average errors were 87, 52 and 39% for bays 3, 4 and 7 respectively. However, using simple regression model, the errors were 184, 111 and 81% for the same bays. Therefore, on the basis of this comparison, the reaction kinetics model seems to give a better estimate of the effluent TP concentrations than the simple regression model.

As mentioned above, the final method is to compare the observed and computed TP concentrations as presented graphically. Considering bay 3 first, the observed and computed effluent TP concentrations using reaction kinetics model (i.e. Figure 6.6) shows a better match than the simple regression model (as shown in Figure 6.14). Similarly comparing Figures 6.7 and 6.8 (i.e. observed and computed effluent TP concentrations for bays 4 and 7 respectively using the reaction kinetics model) to Figures 6.15 and 6.16 (i.e. observed and computed effluent TP concentrations for bays 4 and 7 respectively using the simple regression model), Figures 6.7 and 6.8 show a

closer match between the observed and computed effluent TP concentrations. Therefore, comparisons based on the R^2 values, the absolute average errors and the graphical plots, suggest that the first-order reaction kinetics model gives better estimates of the effluent TP concentrations than the simple regression model. However, the accuracy that can be expected from the reaction kinetics model (which is the better model in this case) is of the order of 45%. This is based on the results of error bands produced for bays 4 and 7, for which calibrations were not done.

6.4 CONCLUSIONS

The nutrient modelling of the WTP grass filtration bays was carried out using first-order reaction kinetics and simple regression models, for Period 1 of the trials only. The data from the three representative bays (i.e. bays 3, 4 and 7) were used for modelling purposes. Similar to the analysis for the hydraulics and water quality chapters, the modelling work was initially based on the two periods (i.e. Period 1 and Period 2). However, since the data for Period 2 resulted in very low correlation coefficients, TN and TP models were only developed for Period 1. The data of bay 3 were used in derivation of model parameters. The models were later validated using the data of bays 4 and 7.

A first-order reaction kinetics model which was based on an exponential decay theory was used to obtain separate temperature-dependent first-order reaction rate constant (k_T) values for Period 1. The k_T values were derived for both TN and TP parameters.

A simple regression model based on linear regression analysis was also used to obtain a simple regression model for TN and TP parameters of the WTP trial bays. This model was also derived for Period 1 of the trials only.

Comparison of the first-order reaction kinetics and the simple regression models derived for the representative bays showed that in predicting effluent concentrations of TN and TP, the reaction kinetics model gives better estimates than the simple regression model. This is based on the comparison of these models with respect to their correlation coefficients, their average absolute error in over or under-estimating

the effluent concentrations and the outputs from the model (i.e. plots of observed and computed effluent concentrations). However, even the first-order reaction kinetics model has poor predictive capabilities as can be seen from low R^2 values and the absolute average errors between the observed and the computed values of each of TN and TP.

CHAPTER 7

COMPARISON OF TRIAL RESULTS WITH OTHER STUDIES IN VICTORIA

7.1 INTRODUCTION

This chapter's heading implies that only comparisons are provided between the results of the Western Treatment Plant (WTP) trial study and other similar studies in Victoria. However, it also provides a comparison between the quality of influent of the WTP trials and of the influent of the potential prototype grass filtration system at WTP.

7.2 COMPARISON OF EFFLUENT DATA FROM TRIAL BAYS AND THE CURRENTLY OPERATING GRASS FILTRATION SYSTEM AT WTP

At present grass filtration treatment is practiced in WTP with influent wastewater, which has undergone sedimentation treatment. The effluent of the currently operating grass filtration system is discharged to Port Phillip Bay via one of four EPA licensed outlets, the Murtcaim outlet (see Figure 1.1). The water quality measurements on ammonia ($\text{NH}_3\text{-N}$), total nitrogen (TN), dissolved oxygen (DO) and five-day biochemical oxygen demand (BOD_5) from the Murtcaim outlet were supplied by Melbourne Water for the same period as the trials discussed in this thesis for comparison of effluent quality. These were the only water quality parameters measured at the Murtcaim outlet.

Effluent quality of the three representative bays (i.e. bays 3, 4 and 7) of the trials are compared with those of the Murtcaim outlet although the influent quality was different because of the different operations of pre-treatment. Nevertheless, it is possible to get some useful information of the treatment efficiencies. These data are shown in Table 7.1. It must be noted that no influent DO and BOD₅ data for the influent to the grass filtration bays draining to the Murtcaim outlet were available. Therefore, no comparison of removal efficiencies were made for BOD₅. The hydraulic loading rate was also not available for the grass filtration bays draining to the Murtcaim outlet, although it is believed to be between 14 and 18 mm/d.

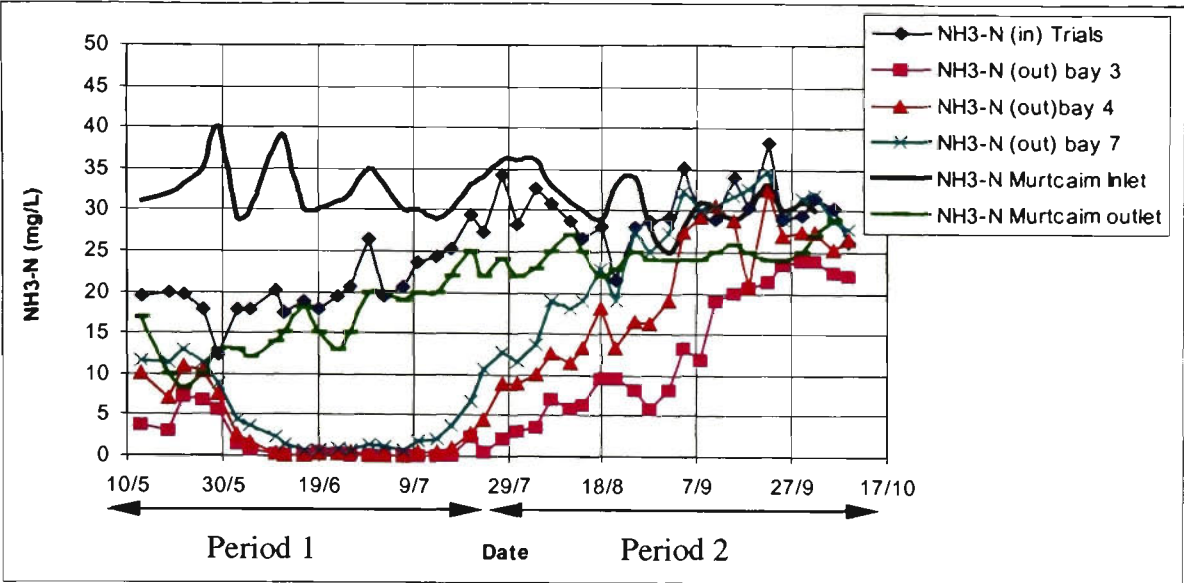
In Table 7.1, the average water quality data shown are arithmetic averages of the concentrations for the study period. According to these averages based on concentration, the NH₃-N removal efficiencies of the trial bays range from about 40% to 70% for the representative bays, while the NH₃-N removal efficiency of the currently operating WTP system is about 42%. Similarly, the TN removal efficiencies of the trial bays range from about 35% to 63%, while the TN removal efficiency of the currently operating WTP system is about 52%. These removal efficiencies suggest that the trial bays (especially the bays with lower hydraulic loading rate) are more efficient in nitrogen removal than the currently operating grass filtration system of the WTP. According to Table 7.1, the WTP current system effluent DO concentration is higher than the effluent DO concentrations of the trial bays.

Figure 7.1 shows the temporal plots of NH₃-N, TN, DO and BOD₅ for the trial bays and the currently operating grass filtration system at the WTP. Although NH₃-N and TN concentrations of influent to the current WTP grass filtration system are significantly higher than those of the influent used for the trials, the WTP system appears to reduce the concentrations of these parameters to levels that are not significantly higher than those of the trial results, probably due to lower hydraulic loading rates, believed to be between 14 and 18 mm/d. As can be seen from Figure 7.1, the influent concentrations of NH₃-N and TN stay fairly constant during the filtration period, unlike in the trials. However, effluent concentrations show an increasing trend similar to that observed for the trials (see Figure 7.1). On the other

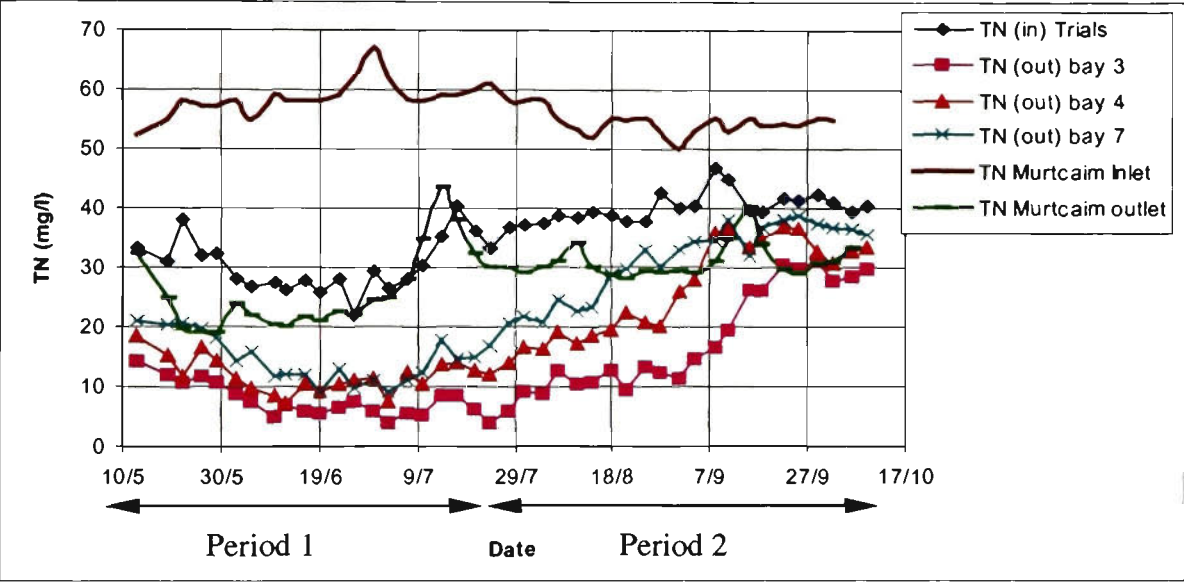
Table 7.1 Comparison of WTP Trials Effluent Quality with those of Murtcaim Outlet

Parameter	1997 Trials (WTP)			Murtcaim Outlet (WTP)
	Bay 3	Bay 4	Bay 7	
Hydraulic Loading (mm/d)	30	40	50	N/A
NH ₃ -N [influent/effluent] (mg/l)	25.4/7.5	25.4/11.8	25.4/15.3	31.8/18.7
TN [influent/effluent] (mg/l)	35.3/12.9	35.3/19.2	35.3/23.0	57.1/27.2
DO [effluent] (mg/l)	1.8	1.8	1.4	3.0
BOD ₅ [influent/effluent] (mg/l)	32.1/12.9	32.1/17.9	32.1/18.1	N/A /11.8

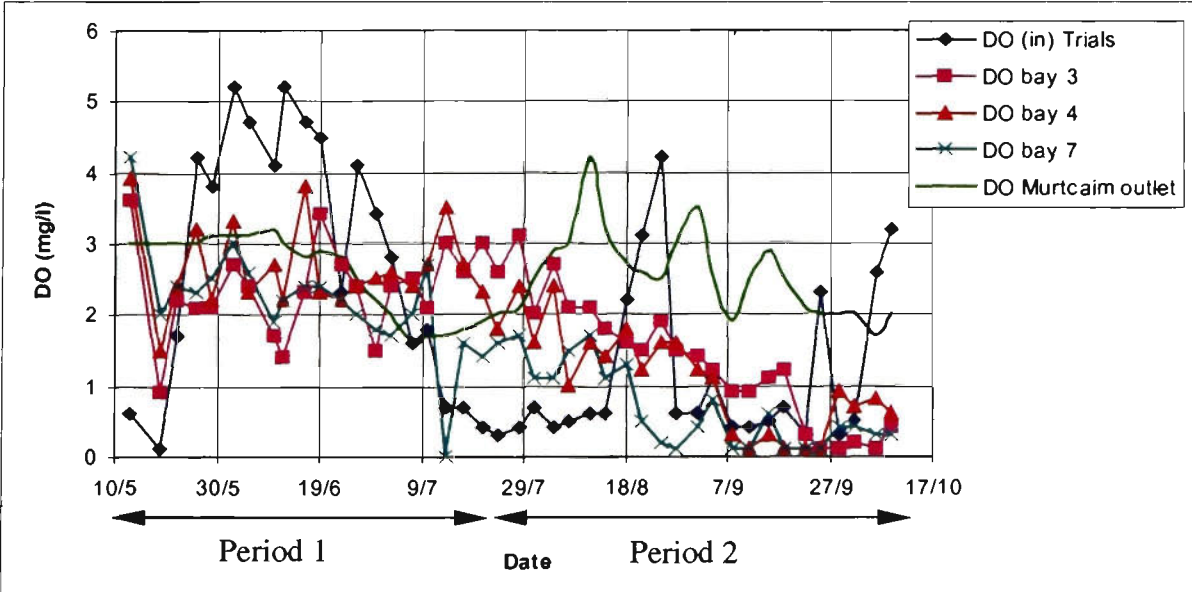
N/A – Not Available



(a) NH₃-N

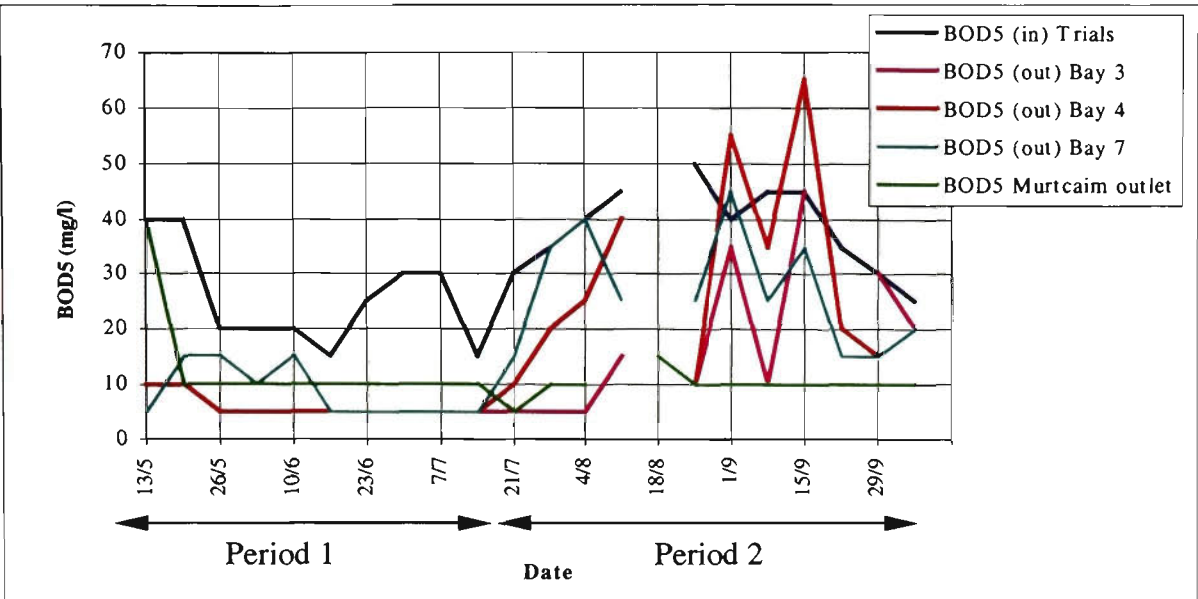


(b) TN



(c) DO

Figure 7.1 Temporal Plots of Trial Bays and Murtcaim Outlet Data



(d) BOD₅

Figure 7.1 Temporal Plots of Trial Bays and Murtcaim Outlet Data (Continued...)

hand, effluent BOD₅ concentrations of the WTP system were lower than those of the trial bays. Although no data was available for influent BOD₅ to the currently operating system, a reasonable estimate for these BOD₅ levels based on typical plant inflow levels for primary settling removal rates might be somewhere between 150-200 mg/l. As well as the different BOD₅ inflows, the difference in the BOD₅ removal between the two systems may also be due to the low hydraulic loading of the WTP system, which is believed to be between 14 to 18 mm/d in comparison to the trial bays having hydraulic loading rates in the range of 30 to 50 mm/d.

7.3 COMPARISON OF EFFLUENT DATA FROM TRIAL BAYS AND PAKENHAM GRASS FILTRATION SYSTEM

The Pakenham grass filtration system which is situated to the southeast of Melbourne uses grass bays all year round. However, for comparison purposes only winter data from the Pakenham system were considered. The Pakenham system data were available for 1995 and 1996, but not for 1997. The available data from the Pakenham system were given as averages of concentrations. For this system, no DO data were given. The data for both systems are shown in Table 7.2.

Table 7.2 Comparison of WTP Trials Results with those of Pakenham Grass Filtration System

Parameter	1997 Trials (WTP)				Pakenham	
	Bay 3	Bay 4	Bay 7		1995	1996
Average Hydraulic Loading (mm/d)	30	40	50		25	22
NH ₃ -N [influent/effluent] (mg/l)	25.4/7.5	25.4/11.8	25.4/15.3		29.7/4.7	25.3/4.0
TN [influent/effluent] (mg/l)	35.3/12.9	35.3/19.2	35.3/23.0		N/A /19.1	30.5/17.6
DO [effluent] (mg/l)	1.8	1.8	1.4		N/A	N/A
BOD ₅ [influent/effluent] (mg/l)	32.1/12.9	32.1/17.9	32.1/18.1		40.6/11.6	22.3/15.2

N/A – Not Available

According to this table, the quality of influent wastewater (i.e. with respect to average $\text{NH}_3\text{-N}$, TN and BOD_5 concentrations) in the Pakenham system is similar to that of the Western Lagoon used for the trials in this project. The $\text{NH}_3\text{-N}$ removal efficiency of the trial bays ranged from 40% to 70%, while the $\text{NH}_3\text{-N}$ removal efficiency of the Pakenham system was 84% for both 1995 and 1996 winter results. However, the TN removal efficiency of the Pakenham system during winter 1996 was 42%, while the TN removal efficiency of the trial bays ranged from 35% to 63%. The hydraulic loading rate of the Pakenham system was 25 and 22 mm/d for the winter of 1995 and 1996 respectively as opposed to the trial bays which had hydraulic loading rates ranging between 30 to 50 mm/d. As can be seen from the above discussions, the Pakenham system showed higher efficiency in removal of $\text{NH}_3\text{-N}$. This is to be expected since the hydraulic loading rate of the Pakenham system is lower than that of the trial bays. However, TN removal does not show this behaviour, which cannot be explained.

The BOD_5 removal efficiencies of the trial bays ranged from 43% to 60%, however, the BOD_5 removal efficiency of the Pakenham system was 71% and 32% for the 1995 and 1996 winter periods respectively. This shows that the Pakenham system was more efficient in BOD_5 removal during 1995 than the trial bays of the WTP, which is expected because of lower hydraulic loading rate. However, this is not the case in 1996 which was also found with TN. This suggests that the effluent data of 1996 may not be accurate at least for TN and BOD_5 . Attempts were made to explain the reduction in performance at Pakenham in 1996, but met with little success.

7.4 COMPARISON OF INFLUENT QUALITY OF WTP TRIAL SYSTEM AND PROPOSED PROTOTYPE SYSTEM

As was discussed in Section 1.2, the results from this trial project will be considered in the design of a full scale (i.e. prototype) grass filtration system WTP. The grass filtration bays used for these trials were supplied with wastewater from a minor lagoon (called Western Lagoon). However, the major lagoons of WTP are expected to

provide the influent for the potential prototype grass filtration system (via 15-East Drain). Thus, a comparison of the influent quality of the prototype grass filtration system with that of the WTP trial system is necessary. The 15-East drain effluent data (which represent the influent to the proposed prototype system) were provided by Melbourne Water and are summarised in Table 7.3. The influent data of the trials are also summarised in Table 7.3 for comparison purposes.

To test the hypotheses that the influent supplied to the trial bays was of similar quality as that for the prototype system, statistical tests were performed. Two independent statistical tests, namely, Mean and Variance tests were performed on the data shown in Table 7.3. The Mean and Variance tests were performed on concurrent data collected between 13th of May and 12th of October 1997 from both sources.

The Mean test provides information on whether the two samples have the same mean within the statistical accuracy. Similar information is provided by the Variance test on the standard deviation of the samples. The two tests together indicate whether the two samples have similar statistical properties, especially the mean and the standard deviation. These methods are also detailed in standard statistical text books such as Neter et al. (1989).

Both Mean and Variance tests were conducted based on a 95% confidence interval. The results from the Mean and the Variance tests are summarised in Tables 7.4 and 7.5 respectively. According to the hypothesis test results shown in Tables 7.4 and 7.5 it is evident that the majority of the water quality parameters have similar statistical properties from both sources, within the statistical accuracy. This proves that the quality of the influent irrigated onto the trial bays was similar quality to that which would be used for the full scale grass filtration system.

The results from the Mean test (see Table 7.4) show that the wastewaters of the prototype system and the WTP trial system have different field pH levels. The Variance test (see Table 7.5) shows that the TSS concentrations of the two sources did not match.

Table 7.3 Summary of Statistical Data of the Trial System and WTP Prototype Influent Quality (mg/l) Throughout Winter 1997

Parameter	Trial System		Prototype System	
	Mean	Standard Deviation	Mean	Standard Deviation
NO _x -N	0.3	0.2	-	-
NO ₃ -N	0.2	0.1	1.6	1.0
NO ₂ -N	0.1	0.1	10.7	4.9
NH ₃ -N	25.4	5.9	20.3	8.8
TKN	35.0	6.2	-	-
TN	35.3	6.1	37.6	5.1
BOD ₅	32.1	10.6	-	-
BOD5(Filter)	6.2	3.9	-	-
CBOD ₅	21.6	13.9	-	-
COD	188.7	95.6	-	-
Colour	233.7	75.7	100.0	0
DO	1.9	1.7	6.5	1.2
pH	7.9	0.2	7.6	0.1
OP-P	3.6	2.2	-	-
TP	9.7	1.2	10.3	0.9
TSS	45.3	19.4	31.1	6.7
Coliforms	1196.6	2707.3	-	-

- Data not available

7.5 SUMMARY

This chapter mainly focused on comparison of the effluent data of the WTP trial bays to the data from the currently operational grass filtration system of the WTP and the Pakenham grass filtration system. These comparisons were made to examine how closely the data and the removal efficiency of various pollutants of the trial bays of

Table 7.4 Mean Test Results

Source	Parameter	Mean (mg/l)	Standard Deviation (mg/l)	Variance	t	Test Result
Prototype System	ORG_N	5.0	2.6	6.8	1.980	Ho
WTP Trial System	ORG_N	0.3	0.2	0.042		
Prototype System	NO ₃ -N	10.7	4.9	24.0	1.980	Ho
WTP Trial System	NO ₃ -N	0.2	0.1	0.038		
Prototype System	NO ₂ -N	1.6	1.0	1.0	1.980	Ho
WTP Trial System	NO ₂ -N	0.1	0.1	0.042		
Prototype System	NH ₃ -N	20.3	8.8	77.9	1.980	Ho
WTP Trial System	NH ₃ -N	25.4	5.9	35.3		
Prototype System	TN	37.6	5.1	25.7	1.980	Ho
WTP Trial System	TN	35.3	6.1	36.9		
Prototype System	BOD ₅	-	-	-	2.010	-
WTP Trial System	BOD ₅	32.1	10.6	111.4		
Prototype System	COLOUR	100.0	0.0	9545.5	1.980	Ho
WTP Trial System	COLOUR	233.7	75.7	5859.6		
Prototype System	DO	6.5	1.2	20.0	1.980	Ho
WTP Trial System	DO	1.9	1.7	2.8		
Prototype System	pH(field)	7.6	0.1	0.03	1.980	Ha
WTP Trial System	pH(field)	7.9	0.2	0.10		
Prototype System	TP	10.3	0.9	0.80	2.010	Ho
WTP Trial System	TP	9.7	1.2	1.5		
Prototype System	TSS	31.1	6.7	45.4	2.010	Ho
WTP Trial System	TSS	45.2	19.3	374.9		

Ho = hypothesis accepted (Neter et al., 1989)

Ha = hypothesis rejected (Neter et al., 1989)

t = t statistic for confidence limit of 95%.

- = data not available

Table 7.5 Variance Test Results

Source	Parameter	Mean(mg/l)	Standard Deviation (mg/l)	Variance	s1	s2	s1/s2	L	U	Test Result
Prototype System	ORG_N	5.0	2.6	6.8	6.8	0.042	161.9	76.2	320.0	Ho
WTP Trial System	ORG_N	0.3	0.2	0.042						
Prototype System	NO ₃ -N	10.7	4.9	24.0	24.0	0.035	685.7	328.4	1379.5	Ho
WTP Trial System	NO ₃ -N	0.2	0.1	0.038						
Prototype System	NO ₂ -N	1.6	1.0	1.0	3.6	0.042	85.7	40.5	170.0	Ho
WTP Trial System	NO ₂ -N	0.1	0.1	0.042						
Prototype System	NH ₃ -N	20.3	8.8	77.9	77.9	35.30	2.2	1.1	4.4	Ho
WTP Trial System	NH ₃ -N	25.4	5.9	35.3						
Prototype System	TN	37.6	5.1	25.7	25.7	36.90	0.7	0.3	1.4	Ho
WTP Trial System	TN	35.3	6.1	36.9						
Prototype System	BOD ₅	-	-	-	-	-	-	-	-	-
WTP Trial System	BOD ₅	32.1	10.6	111.4						
Prototype System	COLOUR	100.0	0.0	9545.5	9545.5	5859.6	1.6	0.8	3.3	Ho
WTP Trial System	COLOUR	233.7	75.7	5859.6						
Prototype System	DO	6.5	1.2	20.0	20.0	2.8	7.1	3.4	14.2	Ho
WTP Trial System	DO	1.9	1.7	2.8						
Prototype System	pH(field)	7.6	0.1	0.03	0.03	0.10	0.3	0.1	0.6	Ho
WTP Trial System	pH(field)	7.9	0.2	0.10						
Prototype System	TP	10.3	0.9	0.80	0.80	1.5	0.5	0.2	1.2	Ho
WTP Trial System	TP	9.7	1.2	1.5						
Prototype System	TSS	31.1	6.7	45.4	45.4	374.9	0.1	0.1	0.3	Ha
WTP Trial System	TSS	45.2	19.3	374.9						

Ho = hypothesis accepted (Neter et al., 1989)
Ha = hypothesis rejected (Neter et al., 1989)
S1 = variance of sample data of prototype system
S2 = variance of sample data of trial system
L = lower confidence limit
U = upper confidence limit
. = data not available

the WTP matched with those of the above systems. A comparison of the influent quality of WTP trials and the prototype system was also made. Comparison of the effluent data of WTP trial bays and the currently operational grass filtration system at the WTP showed that the trial bays were more efficient in removal of $\text{NH}_3\text{-N}$ and TN. Bay 3 which was the bay with the closest hydraulic loading (i.e. 30 mm/d) to the currently operating system, still had higher removal efficiency than the current system, which was believed to have a hydraulic loading in the range of 14-18 mm/d. The DO effluent value of the WTP system was higher than that of the trial bays. The reason for this was not known. However, the BOD_5 effluent value of the WTP system was lower than that of the trial bays, which was believed to be related to its low hydraulic loading rate of 14-18 mm/d.

The Pakenham system was more efficient in removal of $\text{NH}_3\text{-N}$ and slightly less efficient in removal of TN when compared to the trial system at the WTP. The BOD_5 removal efficiency of the two systems were also compared and hence it was found that in winter of 1995 the Pakenham system was more efficient in BOD_5 removal than the WTP trials. However, the Pakenham system showed lower removal efficiencies of both TN and BOD_5 during 1996 with the same hydraulic loading rate for both 1995 and 1996 when compared to the WTP trial system. This suggested that the Pakenham effluent data of 1996 may have been inaccurate. Other causes for these low removal efficiencies at Pakenham could not be identified, although lots of attempts were made to do so. The comparisons showed that, in general, hydraulic loading can affect the nitrogen removal efficiency of grass filtration system.

The comparison of influent concentrations of the WTP trials and those of the WTP proposed prototype system showed that most important water quality parameters of the two systems had similar statistical properties within statistical accuracy. The only parameters which were different were pH and TSS of the influent.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

The following conclusions were drawn from the 6-month long monitoring program of the trial grass filtration system at the Western Treatment Plant (WTP) in Werribee, Victoria (Australia) and subsequent analysis of the collected data.

1. Extensive review of the available literature on natural wastewater treatment showed that grass filtration has been used as a tertiary treatment process world-wide. It was found that in most countries, full-scale systems were only implemented upon successful completion of field trials. In Australia, most of the cities situated along the coastline made use of grass filtration (which includes constructed wetlands) as a tertiary treatment of domestic wastewaters. In particular, Queensland had operated several wetlands in the last two decades, which used grass filtration.
 2. The trial study, involved an extensive data collection and water quality monitoring program. During this monitoring program, several problems were encountered. These included loss of valuable flow data due to unavailability (during the early part of the program) and malfunctioning of flow measuring equipment (i.e. data loggers), and cross flows between some adjacent bays due to poor construction of check banks. The loss of data posed difficulties in computing flow and mass balances.
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3. As a result of the loss of flow data from the inlets of bays 2 to 7 and the outlet of bay 1, no mass balance computations could be performed for the periods when these data losses occurred. Therefore, to remedy this problem the missing information was estimated using linear regression of available data. The linear regressions resulted in high correlation coefficients.
 4. Flow balance analysis of the trial bays showed some flow losses and gains within the bays throughout the entire study period. After careful analysis of all inflows and outflows, rainfall and evaporation data, it was found that some losses had occurred through evapotranspiration, some through percolation into the ground and the remainder through cross flows between adjacent bays. Estimation of these losses was necessary to compute the flow and mass balance of the bays, which in turn was regressed to compute the nutrient removal efficiency of the bays. The cross flows due to leaking check banks were the main cause of flow losses and gains within some bays. These bays were bays 2, 4 and 5. With proper construction of check banks, these cross flows could have been avoided.
 5. Analysis of the water quality data throughout the entire duration of the study showed two distinctively different patterns for pollutant removals, one during the first half of the study and the other during the second half. For this reason, the study period was sub-divided into Period 1 and Period 2 (i.e. first and second half of the trials) each with almost the same duration. Based on these two periods, the behaviour of the grass filtration bays was able to be studied more accurately for removal of pollutants. These two periods allowed better understanding of the grass filtration bays behaviour, and especially their dependence on the ageing of the vegetation.
 6. From the seven grass filtration trial bays used for this study, three bays with the most reliable results were chosen as the representative bays. These were bays 3, 4 and 7. Most discussions were based on the results of these bays. These bays were chosen as the representative bays as they represented one of each of the higher hydraulic loading rates of 30, 40 and 50 mm/d. They showed minimal losses, and produced reliable and accurate data.
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7. The laboratory analysis of field samples provided concentrations of several physical, chemical, biological and nutrient parameters. Physical parameters included temperature, pH, redox potential, total suspended solids (TSS) and colour. It was found that the wastewater got cooler as it passed through the grass and was mostly alkaline within the grass filtration bays. Redox potential measurements showed that oxidation of wastes through dissolved oxygen were more effective during Period 1 than during Period 2. TSS removal was found to be quite high on the trial bays during Period 1 and most of Period 2. This was due to the dense coverage of the grass on the bays which acted as a filtering media in retaining the suspended solids.
 8. Of the physical parameters measured in this study, only colour violated the EPA effluent discharge limits. The reason for higher colour values was that the wastewater picked up more colour producing acids during its stay in the bays.
 9. DO concentrations were variable throughout the trials. The variations in the DO levels within the bays were due to photosynthetic activity of the grass through the root zones. These DO variations also affected the nitrogen removals. Even at low DO levels, the nitrogen removal occurred through nitrification and denitrification. Some of the nitrification occurred within the bays by the release of DO through the root zone interface.
 10. The chemical parameters measured and analysed in this study included BOD_5 , $CBOD_5$, $CBOD(Filt)$ and COD. With respect to these parameters, the grass filtration bays were more efficient in removal of BOD_5 , $CBOD_5$ and $CBOD(Filt)$ during Period 1 of the trials compared to Period 2. This was mainly due to the lower influent concentrations of these parameters during Period 1. However, COD removal was higher during Period 2 of the trials.
 11. The biological parameters measured and analysed in the study included nitrogen and phosphorus (i.e. the nutrients), while the only microbiological parameter measured was faecal coliforms. Nitrogen and phosphorus, however, were discussed separately under nutrients. Measurement and analysis of faecal
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coliforms showed that the trial bays reduced a large amount of coliforms. Although the reductions were almost the same during the two periods, Period 2 showed slightly higher coliform reduction than Period 1. The reductions of the bacteria occurred mainly through sedimentation within the trial bays.

12. Since nutrients were the major concern of this project, more emphasis was given to TN and TP throughout the analysis. The removal efficiencies of nutrients were computed for the representative bays using both concentration and mass (or load) balance. If there are no losses in the bays, under steady flow conditions, the removal efficiencies based on concentration and load should give exactly the same result. Therefore, for this case, the removal efficiencies can be computed simply from the concentration results provided by the laboratory analysis of the field samples. However, the reality is that there are losses and flow is not steady in the bays. Therefore, the removal efficiencies should be computed in terms of mass loads, which require both inflow and outflow information, and concentration data. For this project, the removal efficiencies of nutrients computed based on both concentration and load suggested that the bays were efficient in TN removal in both periods but they were inefficient in TP removal, especially during Period 2.
 13. TN analysis based on concentration and mass balances of the representative bays showed that the maximum TN removal occurred within the lower loaded bay, while the minimum TN removal occurred in the higher loaded bay. Therefore, the TN removal efficiency of the grass filtration trial bays was found to be inversely related to the hydraulic loading rate of the bays.
 14. It was also found that the TN removal efficiency was affected by the detention time of the bays. After performing two tracer studies to determine the detention times of the bays (which included bays with hydraulic loading rates of 30, 40 and 50 mm/d), it was found that the higher loaded bays had lower detention times while the lower loaded bays had higher detention times, which was expected. Therefore, the bays with higher detention time had the highest removal efficiency. Hydraulic loading rate directly affects the volume of wastewater that can be disposed of from a bay, while the detention time of the bays determines the time it
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takes for that volume of wastewater to travel through the bay, which affects the effluent quality. Therefore, when designing full-scale grass filtration plants, both these parameters should be considered in the design process. These two parameters are inversely related.

15. The temporal plots of nitrogen throughout the trial study showed that TN removal was variable throughout the season. This variability was such that the nitrogen removal was much greater during Period 1 compared to Period 2. Individual nitrogen forms had similar temporal plots. Several factors contributed to this temporal variability. These included the lower influent nitrogen concentrations during the early parts of the trials, the varying DO levels within the bays which affected the nitrification/denitrification processes, and the ageing of vegetation which affected the nitrogen uptake capacity.
 16. The low influent nitrogen concentrations and the varying DO levels within the bays are factors which cannot be controlled to improve the efficiency of grass filtration bays in removal of nitrogen. However, the ageing of vegetation is one which can be controlled. A possible controlling mechanism would be to have a shorter irrigation season which may not extend beyond the point when the vegetation has reached its peak growth. This will ensure that the maximum nitrogen removal will take place during the entire duration of the irrigation season.
 17. Nitrification/denitrification, plant uptake, volatilisation and sedimentation/cross flow/infiltration were identified as the nitrogen removal mechanisms in this study. A detailed analysis of the nitrogen sinks on the bays showed that 67% of the nitrogen removal from the bays had occurred through nitrification/denitrification. Therefore, nitrification/denitrification was identified as the major nitrogen removal mechanism on grass filtration bays, while plant uptake, volatilisation and sedimentation/cross flow/infiltration also contributed to some nitrogen removal.
 18. Based on the concentration and mass balance analysis of results from the representative bays, it was found that phosphorus removal was inversely related to hydraulic loading rate and was significantly affected by the detention time of the
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bay, especially during Period 2. Longer detention time of the bays allows longer contact time between the wastewater and the soil matrix, as well as the wastewater and the vegetation, enabling some of the phosphorus to be absorbed by the soil matrix and some taken up by the vegetation. The relationship of hydraulic loading rate and detention time with the pollutants removal rate was similar to those for nitrogen removal. The removal efficiency of phosphorus via Italian Ryegrass decreased as the season progressed due to the ageing of vegetation on the bays.

19. The early stages of trials showed some removal of phosphorus, while the remainder of the trial period showed phosphorus production. The low phosphorus removal was due to inability of plants to take-up phosphorus. The phosphorus production may be due to the wash-off of dead micro-organisms.
 20. Plant uptake was identified as the major phosphorus removal mechanism, while sedimentation and soil adsorption were also identified as possible phosphorus removal mechanisms. Once the Italian Ryegrass had matured, its phosphorus uptake capacity was also reduced. This left the soil as the next available medium to take-up the phosphorus from the wastewater. Another factor which affected the phosphorus removal was the deposits from the previous season. Phosphorus deposits from the previous season tend to interfere with the phosphorus removal, causing more deposit of phosphorus (or its production) in the wastewater, especially if the site has been used for wastewater irrigation for many years.
 21. The analysis of the data collected for this project showed that the hydraulic loading rate can significantly affect the removal efficiency of the bays. This is because the bays with higher hydraulic loading rate achieved lower reductions of pollutants, while the bays with lower hydraulic loading rate achieved higher reductions. Naturally, with lower hydraulic loading rates the volume of wastewater treated is less. Therefore, an optimal hydraulic loading rate, which could optimise both the treated volume of wastewater and the pollutant removal efficiency had to be determined. The range of hydraulic loading rates from which an optimal hydraulic loading rate could be derived was between 30 and 50 mm/d. Based on the results of this trial study, the hydraulic loading rate of 40 mm/d was
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chosen as the optimal rate for the bays. While discharging a higher flow of wastewater, reasonably high nutrient removal efficiencies (based on both concentration and loads) could be achieved at this hydraulic loading rate, at least for parts of the trial period. Nutrient removal efficiency of the trial bays were higher during Period 1 than Period 2. However, the total TN load discharged to the Port Phillip Bay based on daily flow at WTP is within limits of EPA licence, especially at the optimal hydraulic loading rate of 40 mm/d.

22. Some preliminary nutrient modelling work of the WTP grass filtration bays was carried out using first-order reaction kinetics and simple regression models (as cited in the literature) to determine the usefulness of these models for WTP. Since it was only a preliminary modelling study, only TN and TP were considered. The models were only developed for Period 1, since the regression analyses of the data of Period 2 resulted in very low correlation coefficients.
23. The first-order reaction kinetics modelling process involved the derivation of temperature-dependent reaction rate constants for TN and TP, and then the validation of these models. The rate constants were derived using data of bay 3 using regression analysis. The models were then developed using these rate constants and they were validated using data of bays 4 and 7.
24. The simple regression model, on the other hand involved linear regression of influent and effluent concentrations of TN and TP. The correlation coefficients for these regressions were quite low as well.
25. After evaluation of outputs and correlation coefficients of both models, it was found that the first-order reaction kinetics model produced better estimates of the effluent TN and TP concentrations of WTP trial bays than the simple regression model. However, even the reaction kinetics model has poor predictive capabilities as based on the low R^2 values and the absolute average errors between the observed and computed values of each of TN and TP. Both the first-order reaction kinetics and the simple regression models derived in this study are specific to the trial bays and trial Period 1 of the WTP and therefore, their use would only be

appropriate for sites of similar conditions (i.e. hydraulic, climatic, soils etc) to those of the WTP trial bays.

26. Results of this trial study were compared with other similar studies in Victoria to investigate how closely they matched with each other. Effluent data from this trial study were compared with the effluent data from the currently operational grass filtration system at the WTP and also with the effluent data from the Pakenham grass filtration system. Such comparisons are useful as they can provide information on how closely the data and the removal efficiencies of the pollutants match those of the other systems. The comparisons showed that the trial bays were more efficient in $\text{NH}_3\text{-N}$ and TN removal than the currently operational system at the WTP. Meanwhile the Pakenham system was more efficient in removal of $\text{NH}_3\text{-N}$ than the WTP trial bays and slightly less efficient in removal of TN in 1996.
 27. The proposed prototype grass filtration system is expected to receive (treated) wastewater from a different source to the one used in the trials. Therefore, the influent data of the trial bays were compared with the influent data from the proposed prototype grass filtration system at the WTP. This comparison showed that apart from pH and TSS, all other parameters of the trial bays and the proposed prototype system had the same statistical properties. However, pH and TSS were not considered important in this study, since the focus was on nutrient removal.
 28. A comparison of the trial bays results with those of the EPA discharge limits showed that effluent colour was the only parameter which violated the discharge limits. For all other parameters, and particularly nitrogen and phosphorus, effluent concentrations were in compliance with the EPA effluent discharge limits to Port Phillip Bay. The annual TN loads discharged from the representative bays were also in compliance with the annual TN load as specified in the EPA licence.
 29. Finally, the results from the trial study showed that grass filtration was suitable for winter treatment of secondary-treated effluent in terms of removal of many
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pollutants. In particular, it was found to be an efficient system in removal of nitrogen.

30. The nitrogen removal efficiency of the grass filtration bays was examined on the basis of its concentrations and loads. Based on the concentrations, the nitrogen removal efficiency of the trial bays was higher than that for the prototype system at the WTP especially during the first half (i.e. Period 1) of the trial study. However, during the second half, the nitrogen removal efficiency declined, indicating that the vegetation may have a shorter lifespan in terms of nitrogen uptake than was initially expected. However, based on the EPA licence limit on the annual TN load discharged into Port Phillip Bay, the grass filtration trial bays seem to produce effluent which has undergone significant polishing treatment and hence contains minimal TN loads.

8.2 RECOMMENDATIONS

Based on the results of the study, the following recommendations are made for Melbourne Water to carry out further trials on winter treatment of secondary-treated effluent using grass filtration system or design and monitoring of the full-scale treatment system. These recommendations are equally applicable to other grass filtration systems.

- The optimal hydraulic loading rate was found to be 40 mm/d in this study which optimised the treated wastewater volume and the removal efficiency. However, this was based on four hydraulic loading rates of 20, 30, 40 and 50 mm/d. Therefore, hydraulic loading rates between 35 and 45 mm/d are recommended as the optimal range for further trials or for the design of a full-scale system.
 - Emphasis should be placed on bay preparation. Check banks of adequate heights should be in place to ensure no or minimal leakage between the adjacent bays. Proper grading and compacting of the bay surface should be done to reduce any potential for water losses through holes or channels into the ground.
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- High priority should be given to improving the quality of flow measurements especially in terms of their availability for the entire duration of the trials, and regular maintenance in case of break downs. Without these measures, valuable flow data may be lost, as was found in this study.
 - Orifice plates should be used at both inlets and outlets of the bays instead of V-notch weirs used for outlets in this study. This is because of the difficulties in the calibration of V-notch weirs. These difficulties may result in production of less accurate rating curves for computation of flows.
 - Detailed monitoring of 24-hr DO profiles should be carried out during different time periods of the trials (i.e. once every 2 weeks) to identify diurnal nitrification and denitrification potential during various stages of the grass filtration process.
 - In future trials and/or monitoring of full-scale grass filtration systems, less frequent monitoring of degradable and non-degradable organic matter, TSS, and faecal coliform need be done, since high treatment efficiencies were achieved for these parameters and their limiting licence requirements were significantly lower than the effluent quality recorded in the current trials.
 - High priority should be given to nitrogen and phosphorus monitoring, since nitrogen and phosphorus are the nutrients causing the eutrophication of Port Phillip Bay.
 - Priority should be given to colour monitoring as well, since the effluent colour levels were higher than the allowable EPA limits in this study.
 - For maximum nutrient removal efficiency, the duration of the irrigation season may be adjusted to match the time it takes for the vegetation planted on the bays to reach their peak growth. This is because once the vegetation has reached peak growth, its nutrient uptake capacity beyond that point is reduced, leading to much lower uptake (and sometimes none at all) of the nutrients from the wastewater.
 - In this study, influent of the grass filtration system did not receive wastewater which is expected to be used for the proposed prototype grass filtration system, although the results of the trials were intended to be used in the design of the prototype system. In such a study, trials should use the influent from the source that is intended for the prototype.
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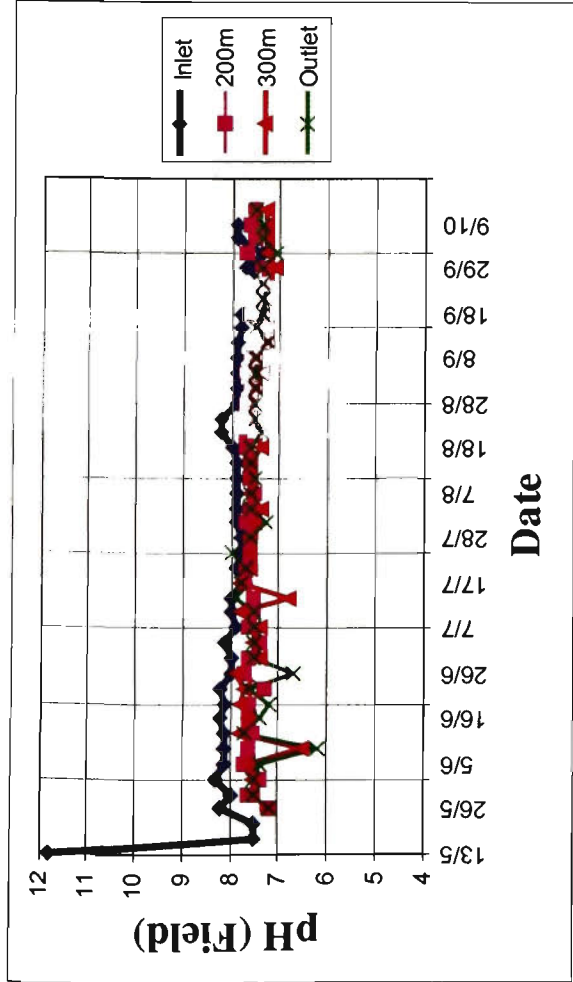
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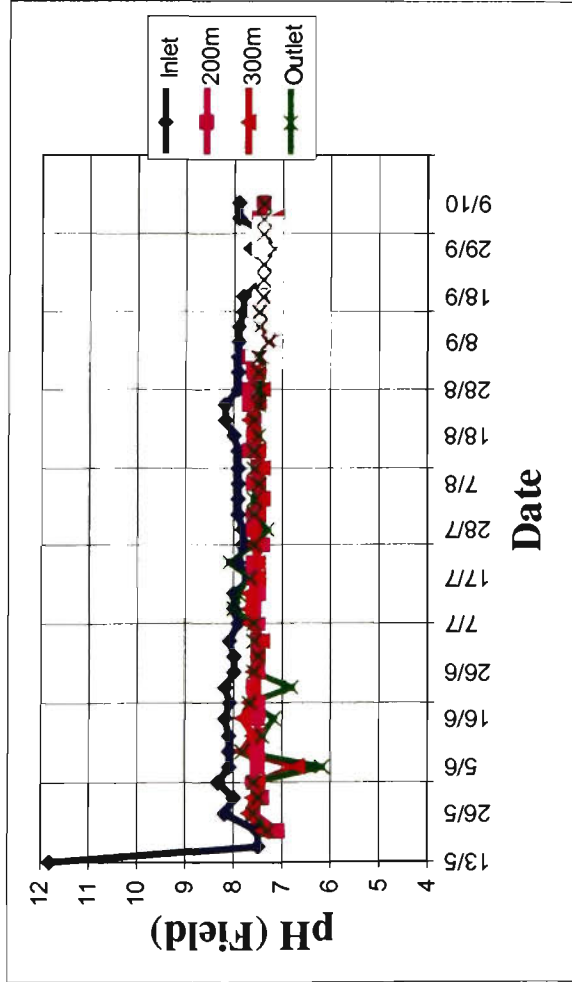
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-

APPENDICES

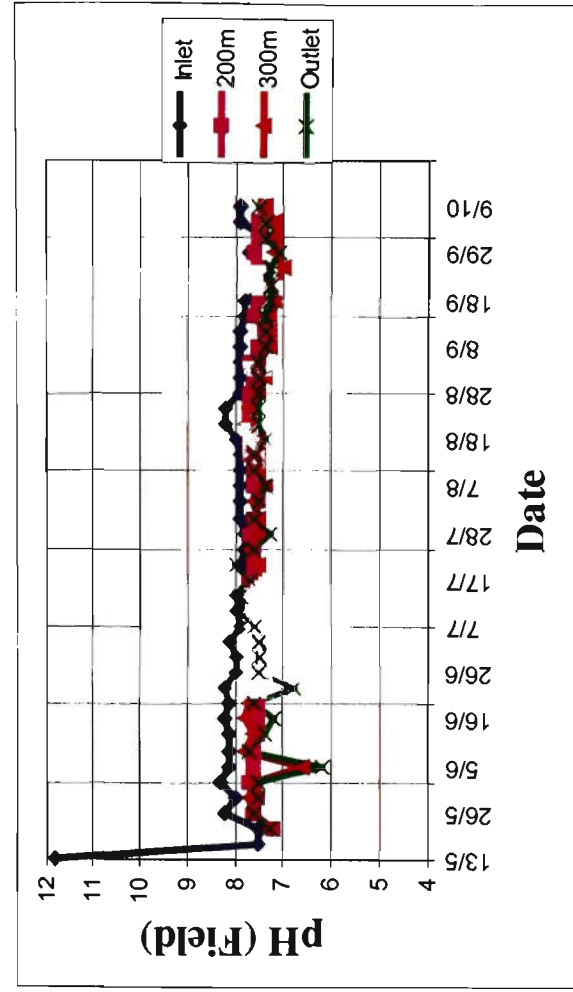
- Concentration plots for all seven bays
(Figures A.1 to A.12)
- Mass balance plots for all seven bays
(Figures A.13 to A.16)



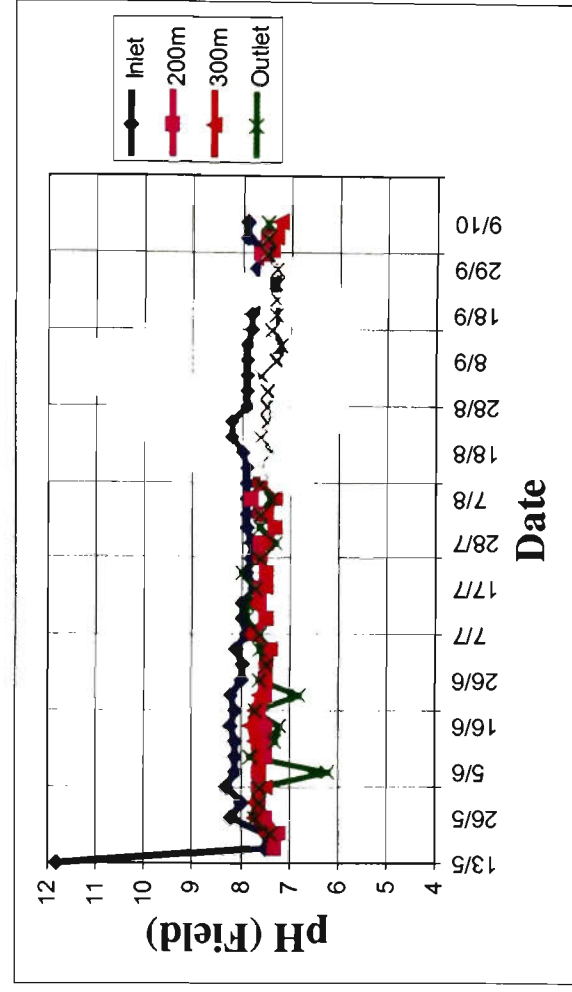
Bay 1



Bay 3

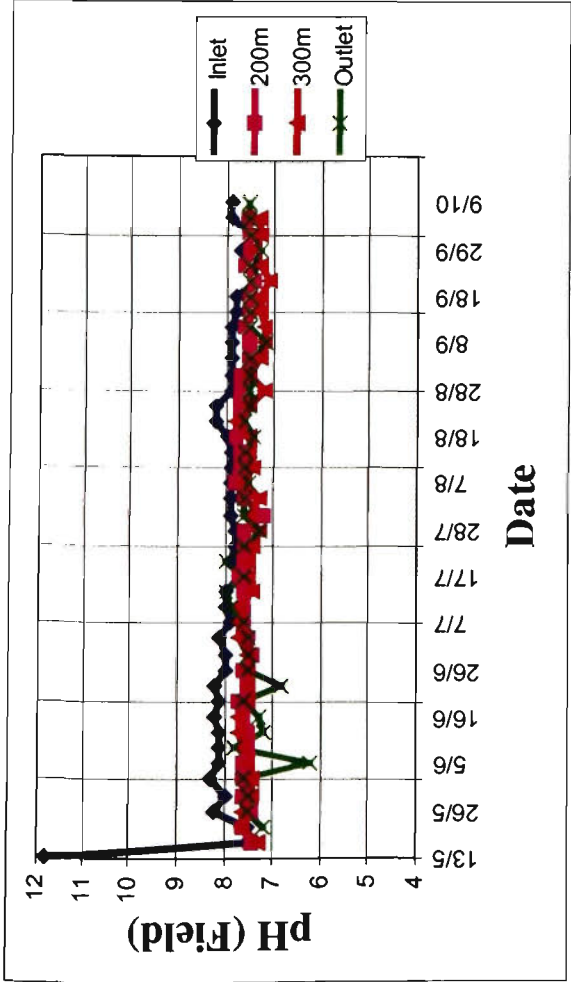


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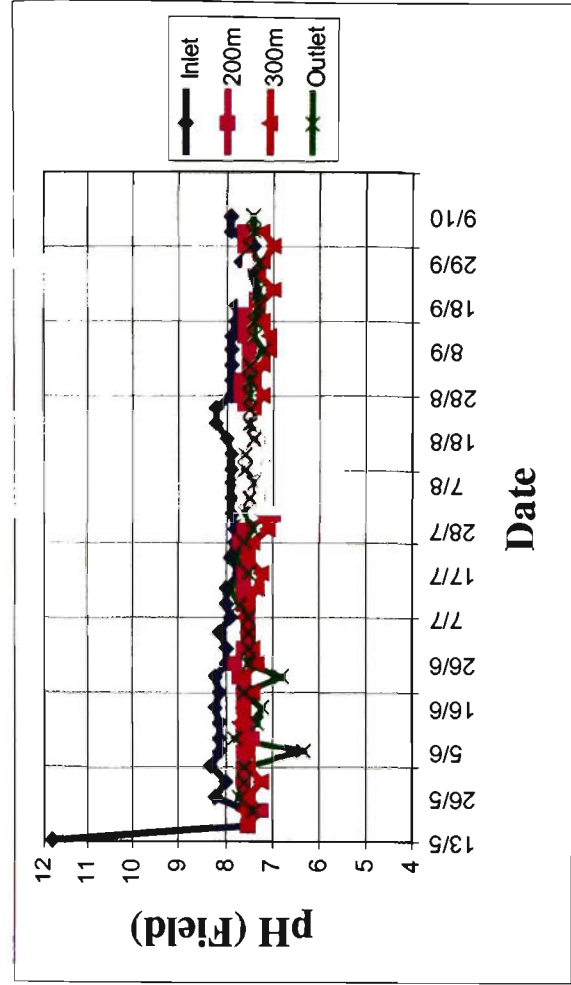


Bay 4

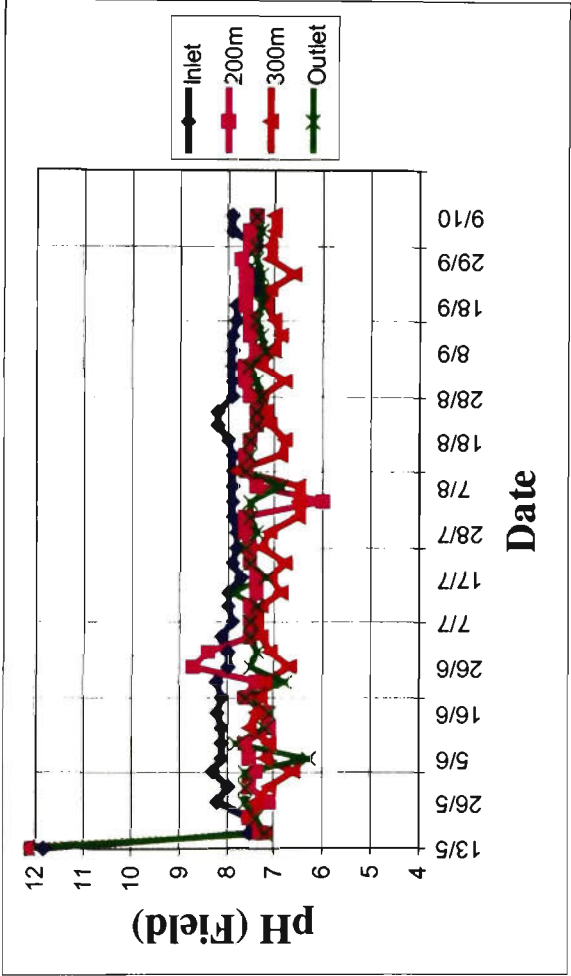
Figure A.1 pH Temporal Plots



Bay 5

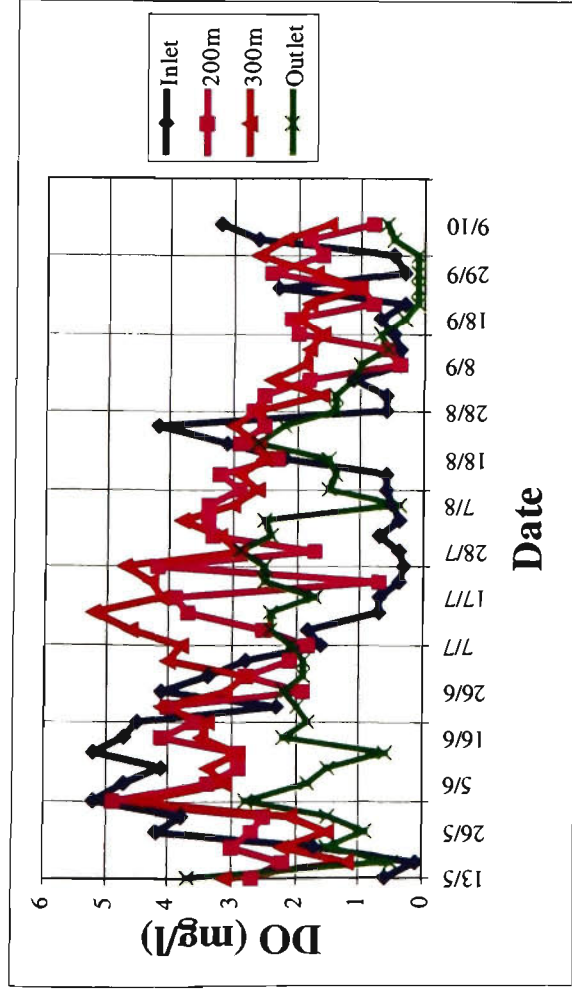


Bay 6

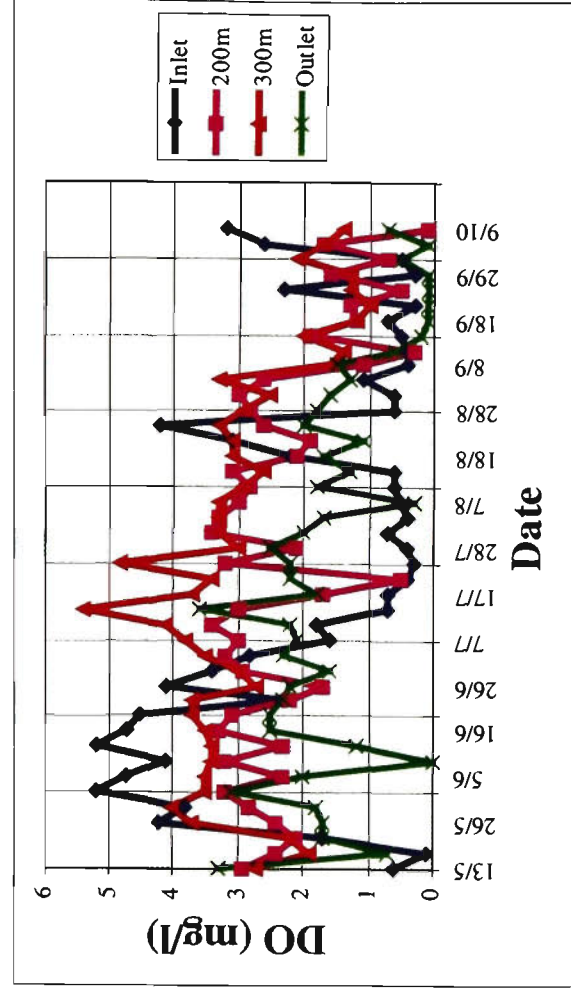


Bay 7

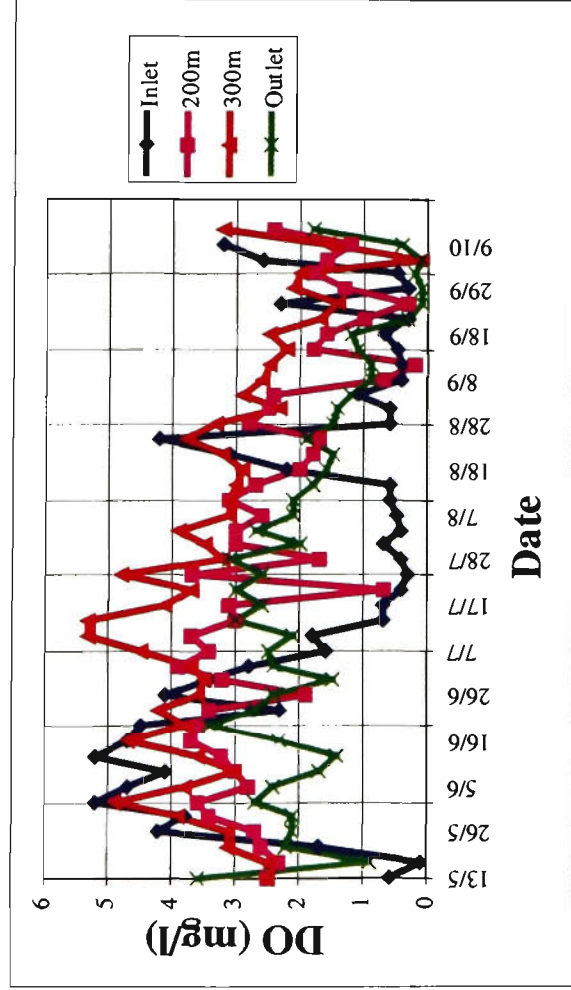
Figure A.1 pH Temporal Plots (Continued...)



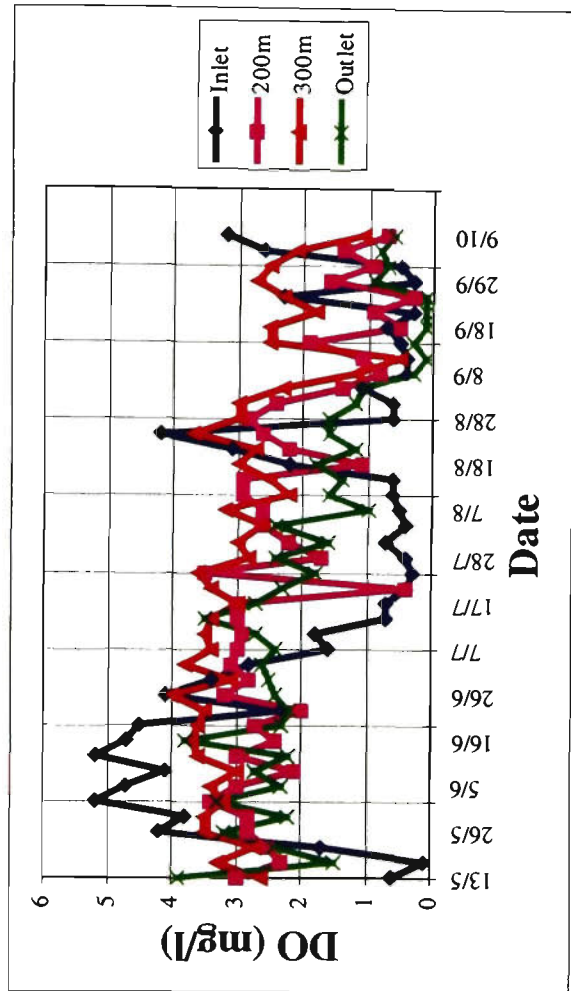
Bay 1



Bay 2



Bay 3



Bay 4

Figure A.2 DO Temporal Plots

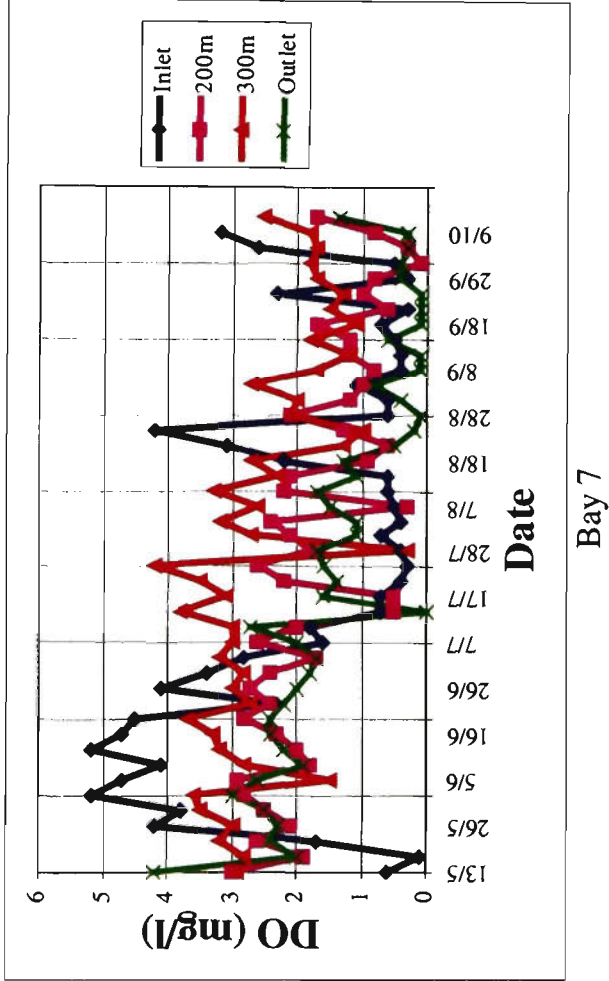
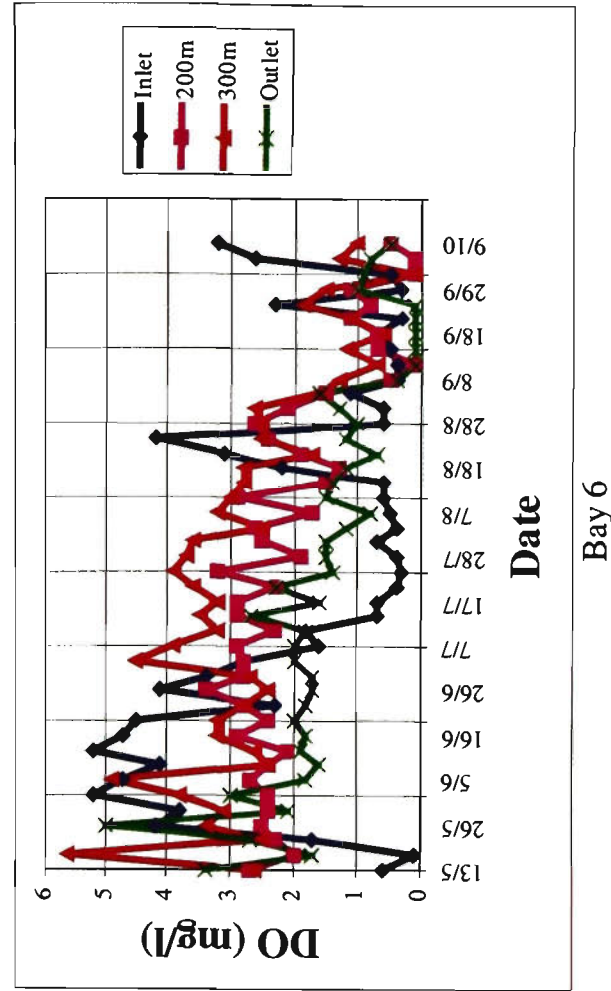
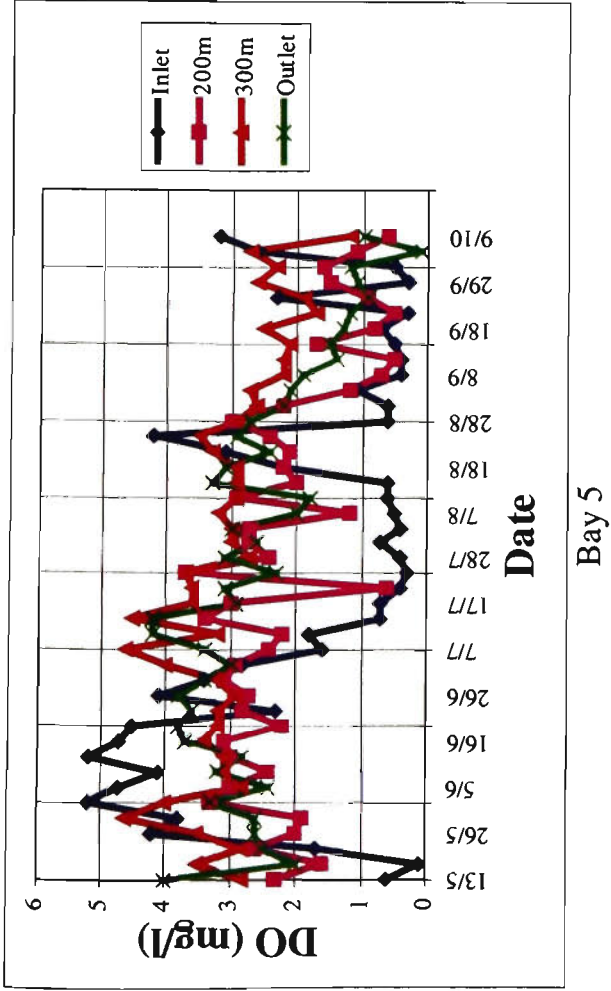


Figure A.2 DO Temporal Plots (Continued...)

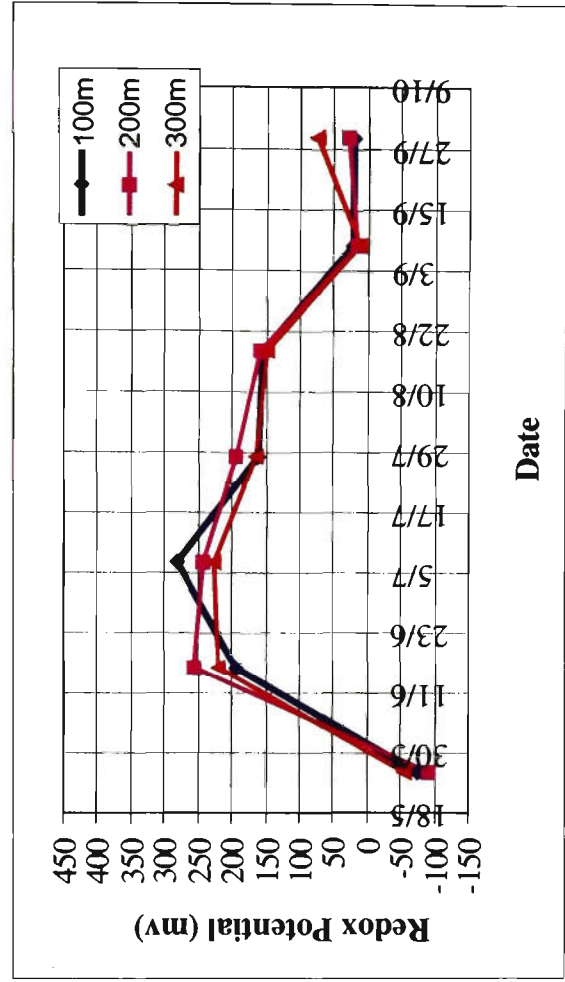
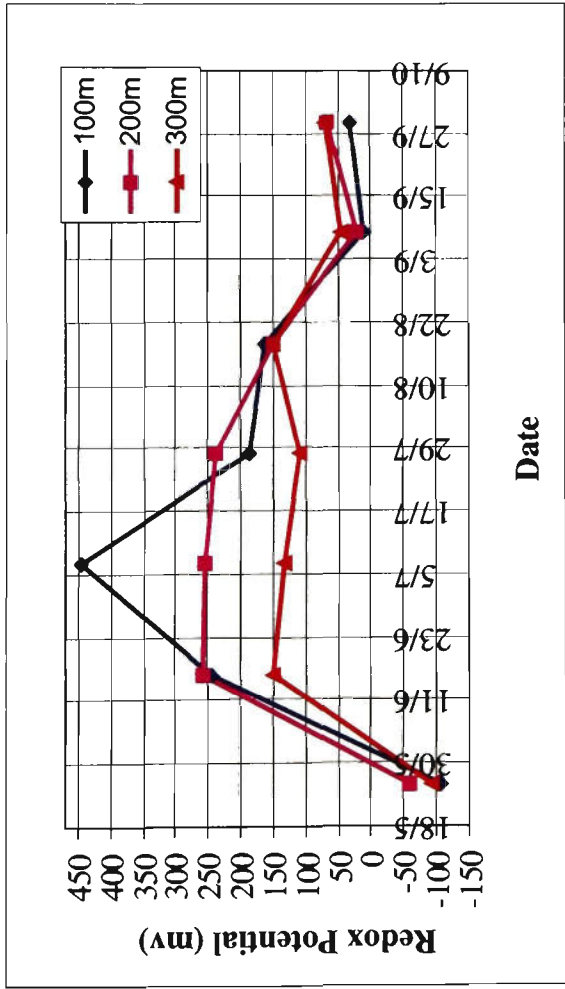
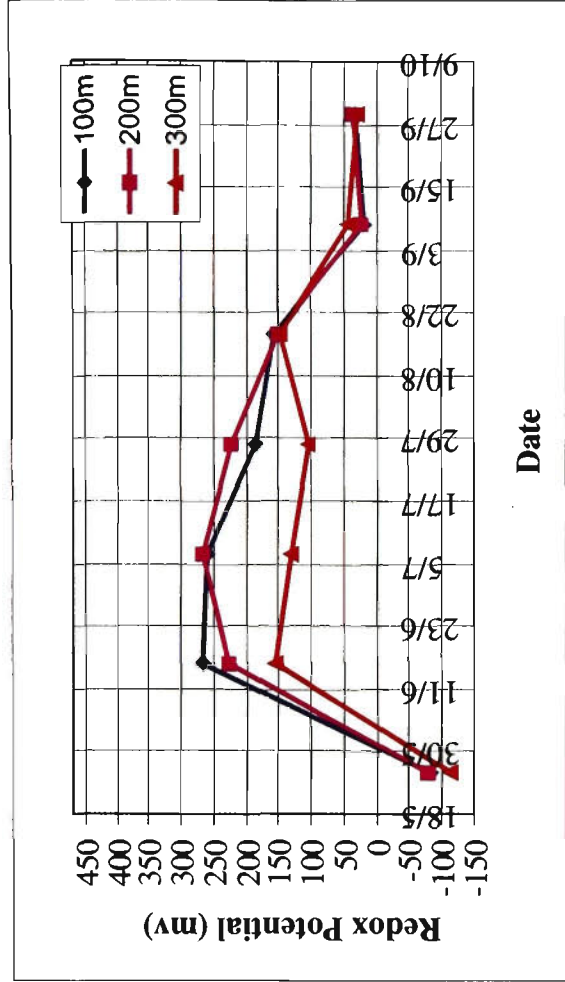
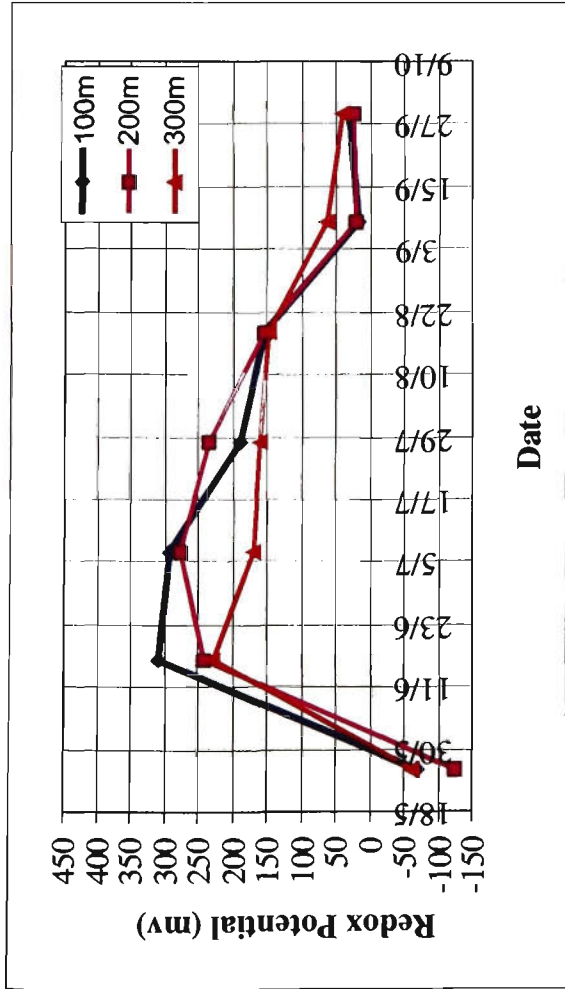


Figure A.3 Redox Potential Temporal Plots

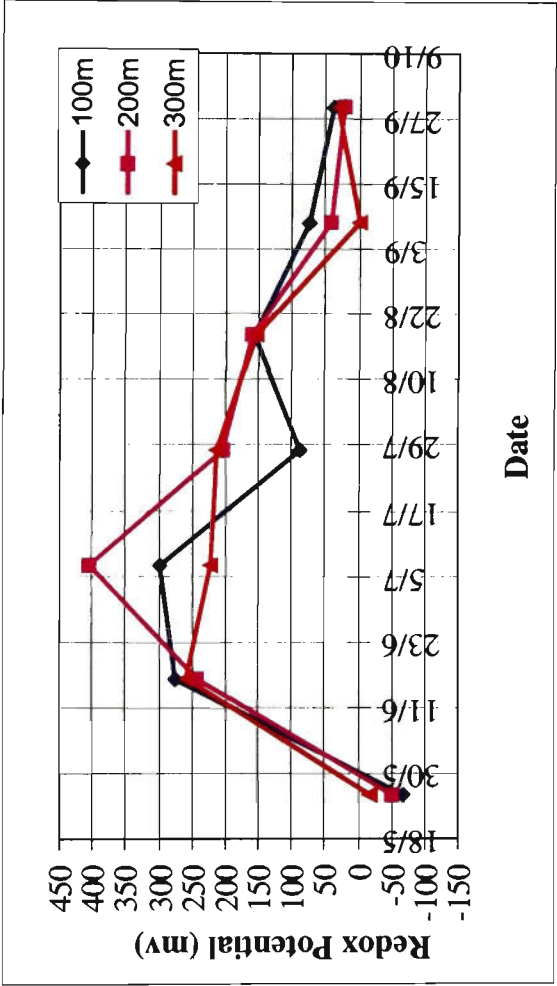
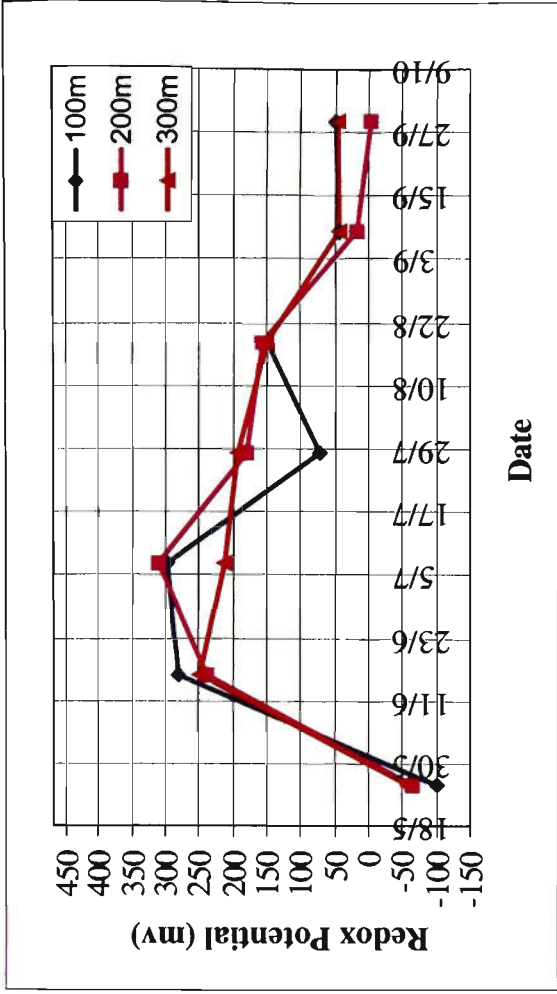
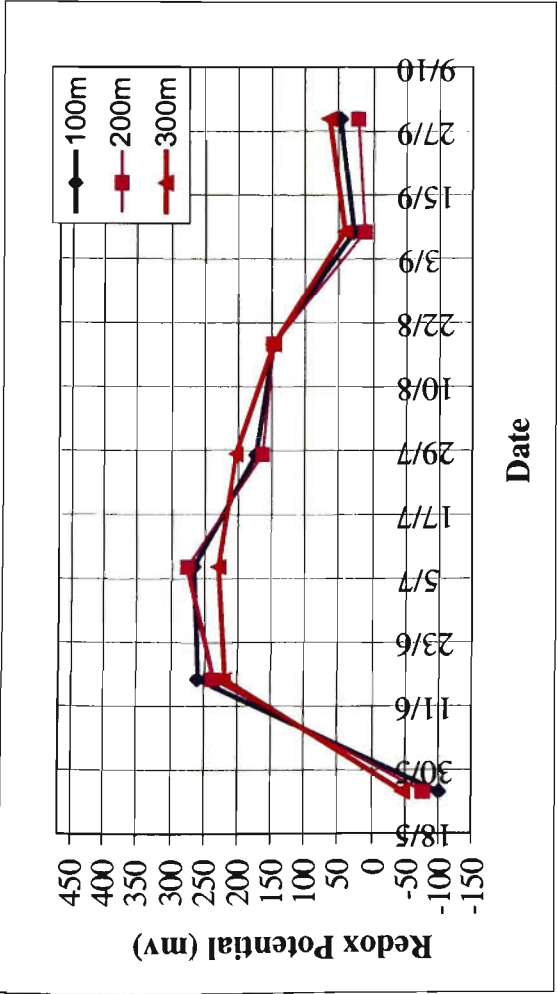


Figure A.3 Redox Potential Temporal Plots (Continued...)

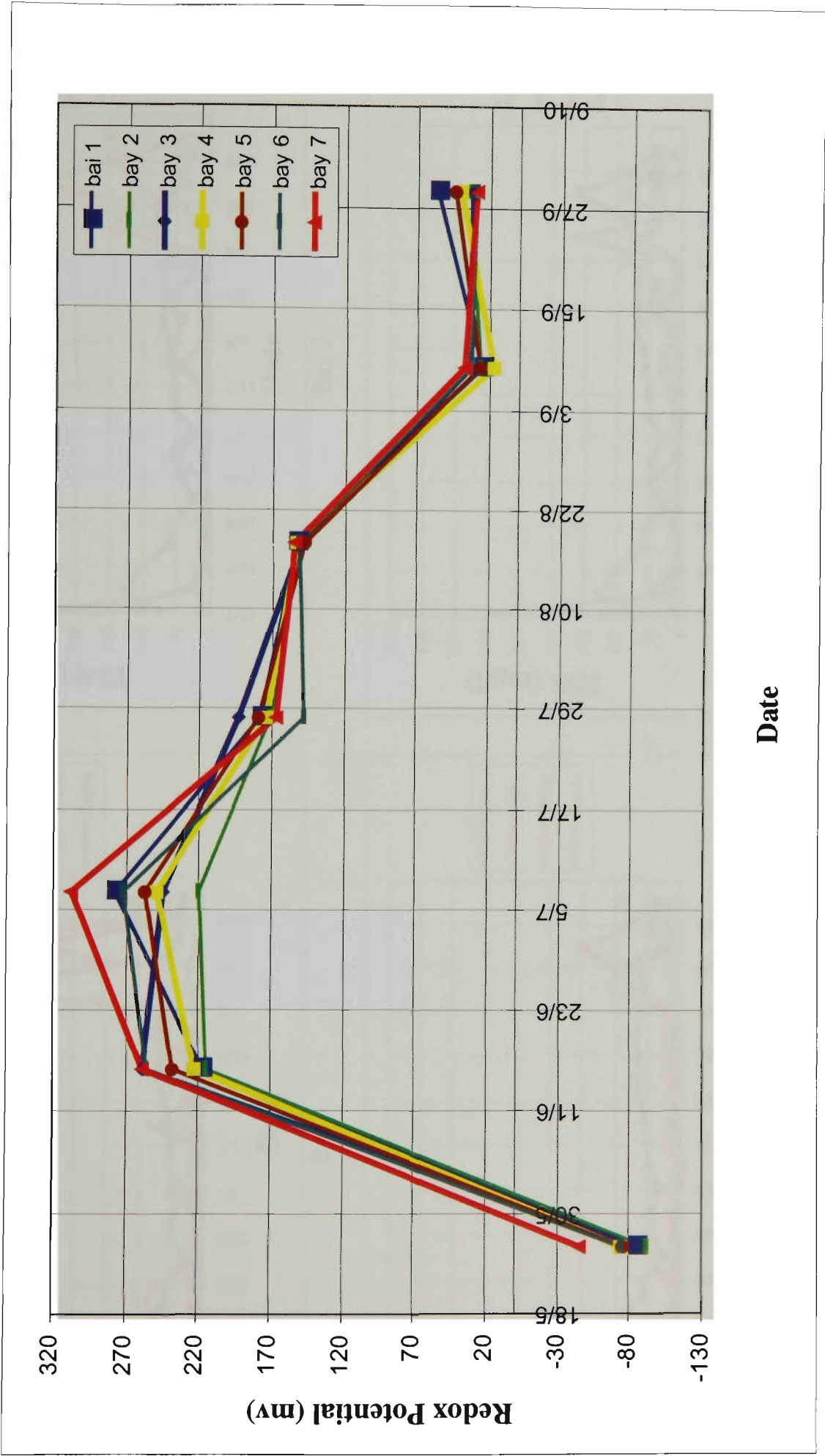
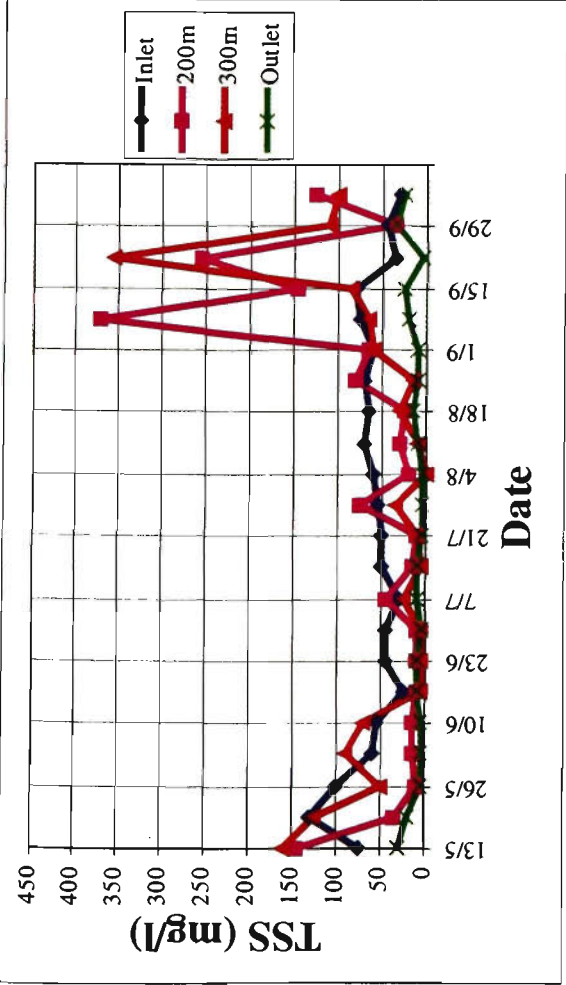
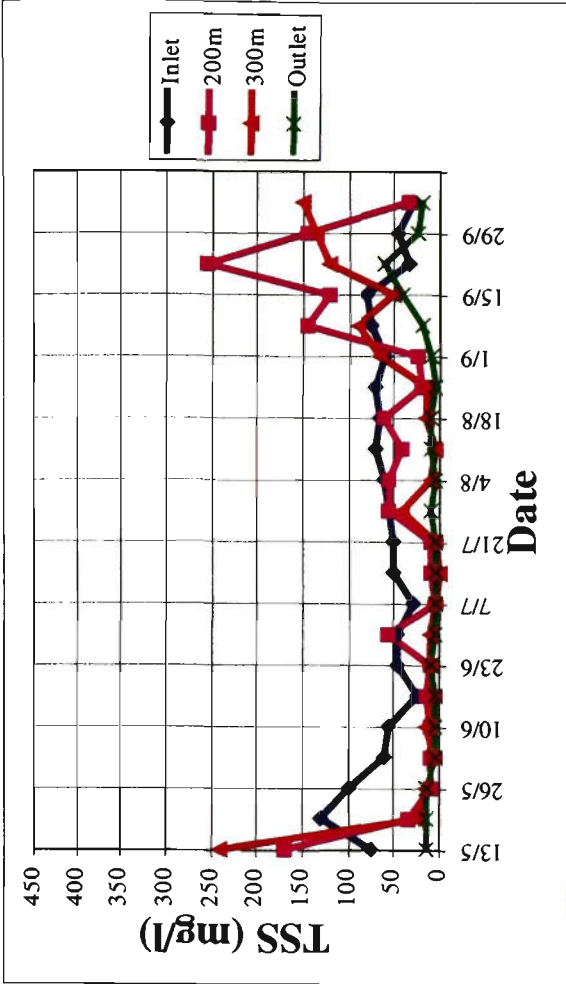


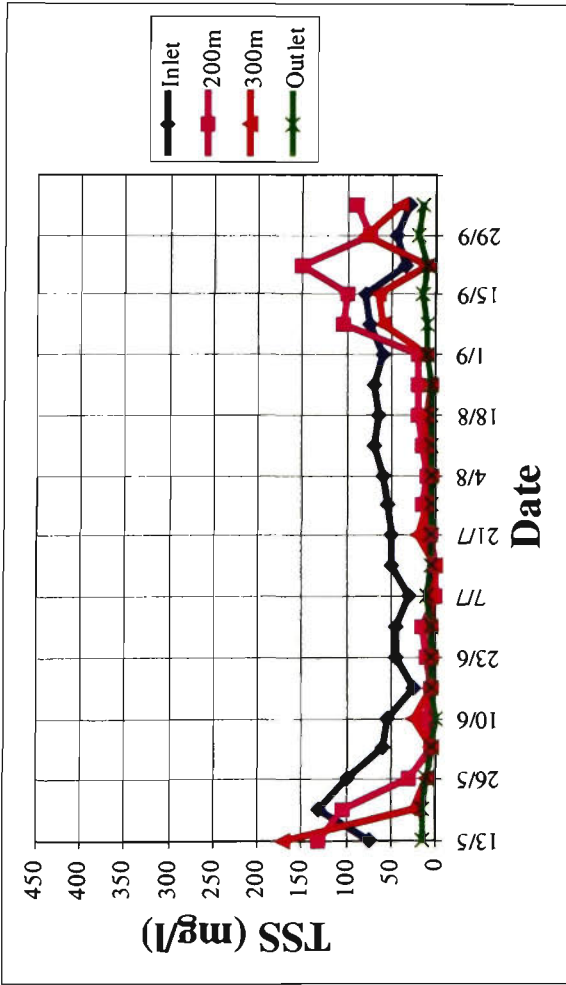
Figure A.4 Average Redox Potential Temporal Plots of all seven bays



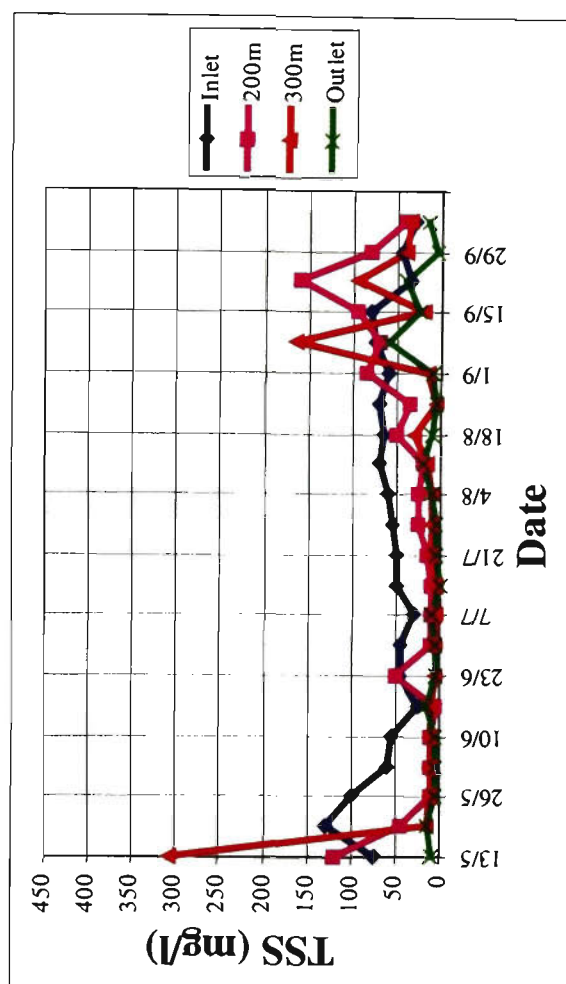
Bay 1



Bay 2

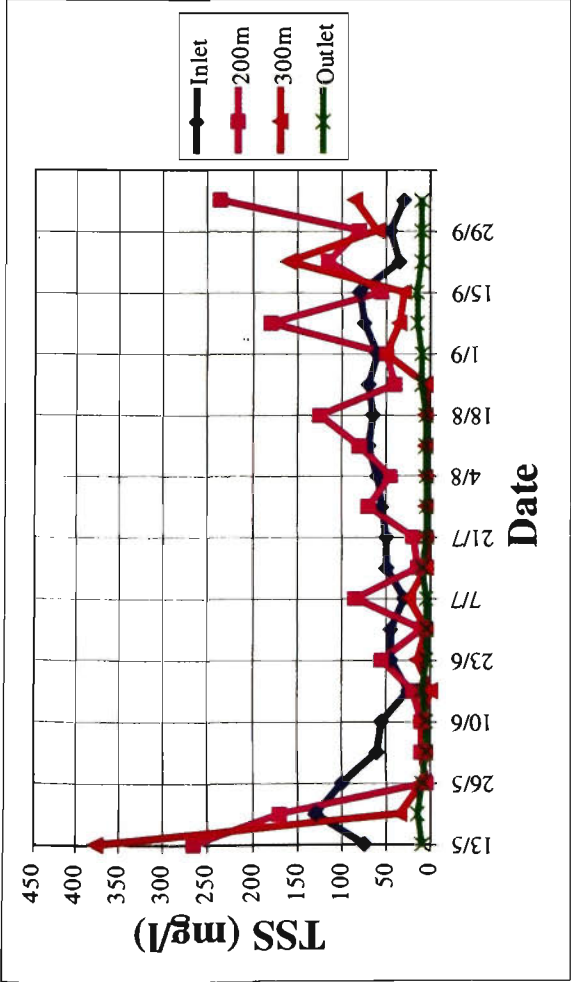


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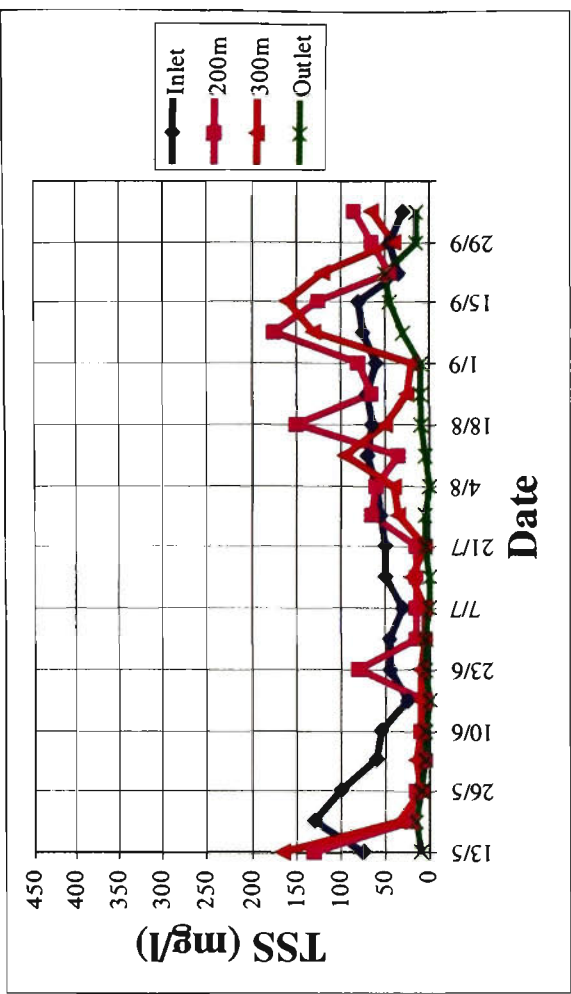


Bay 4

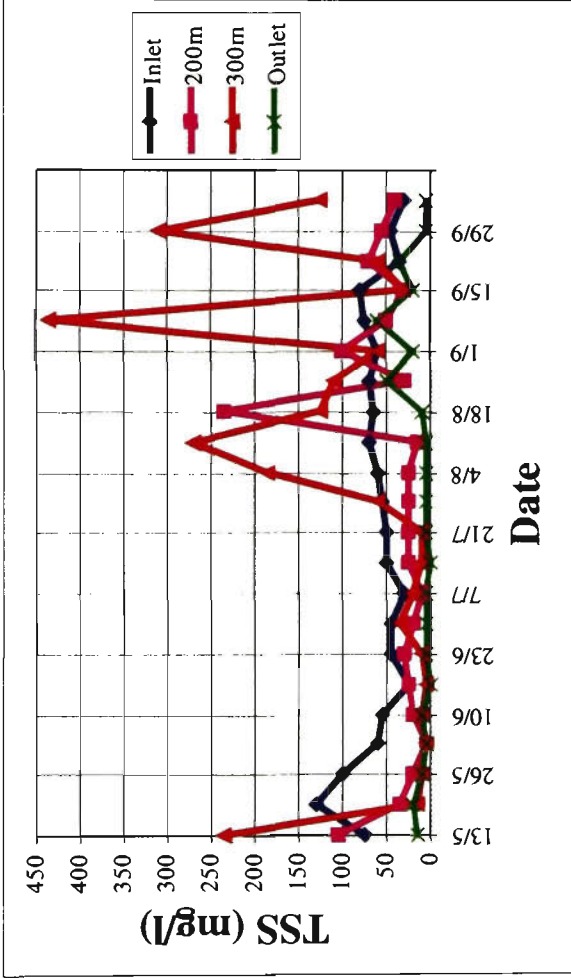
Figure A.5 TSS Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.5 TSS Temporal Plots (Continued)

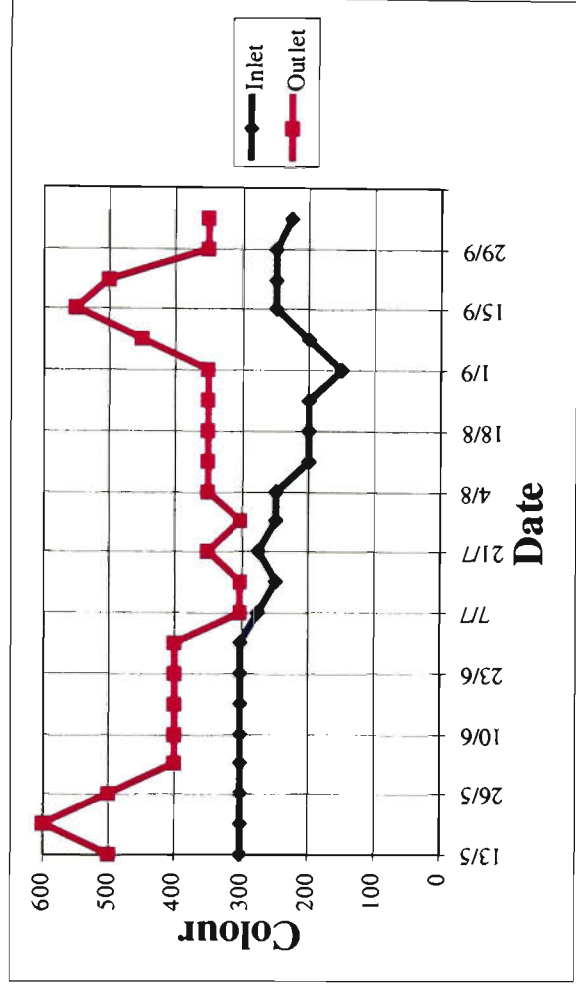
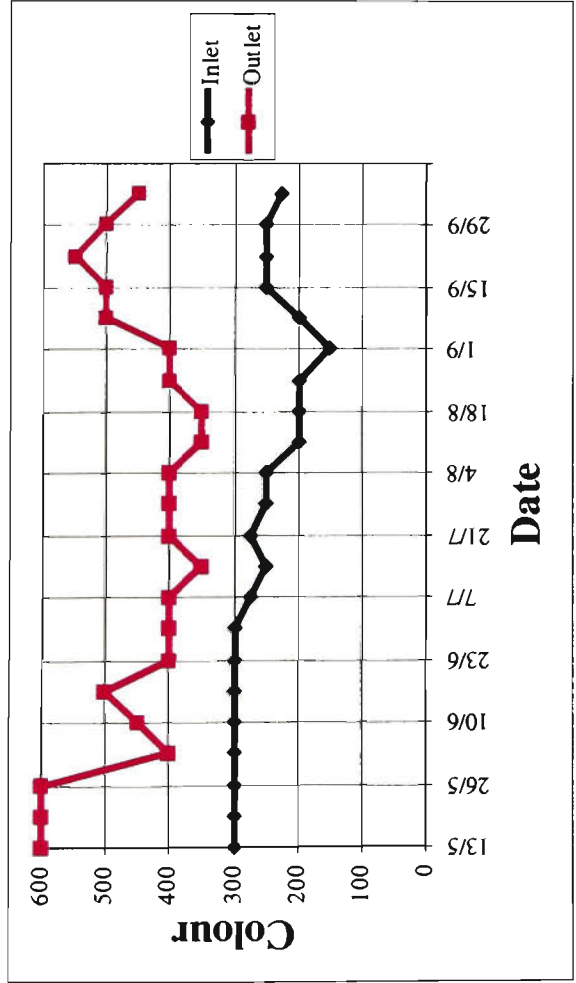
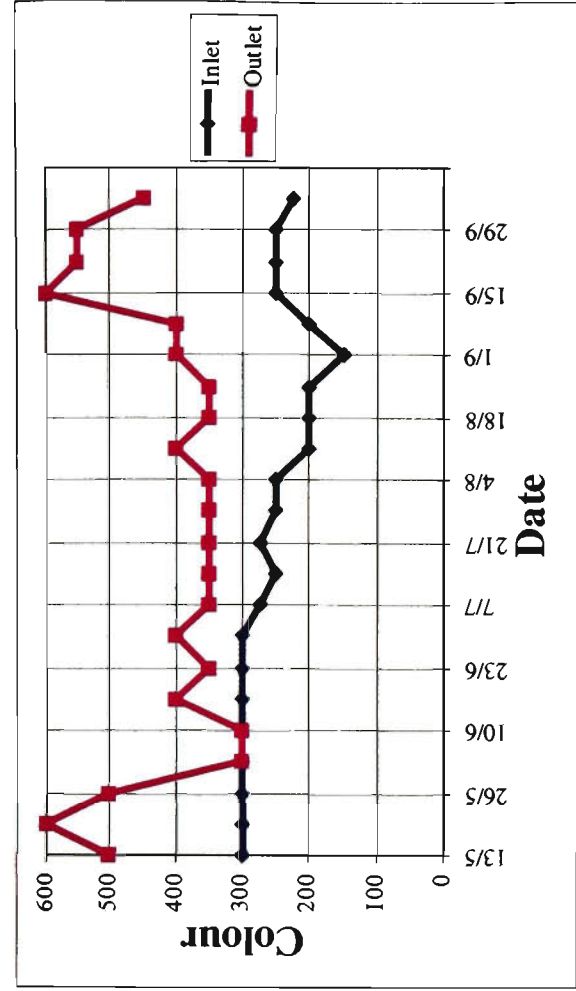
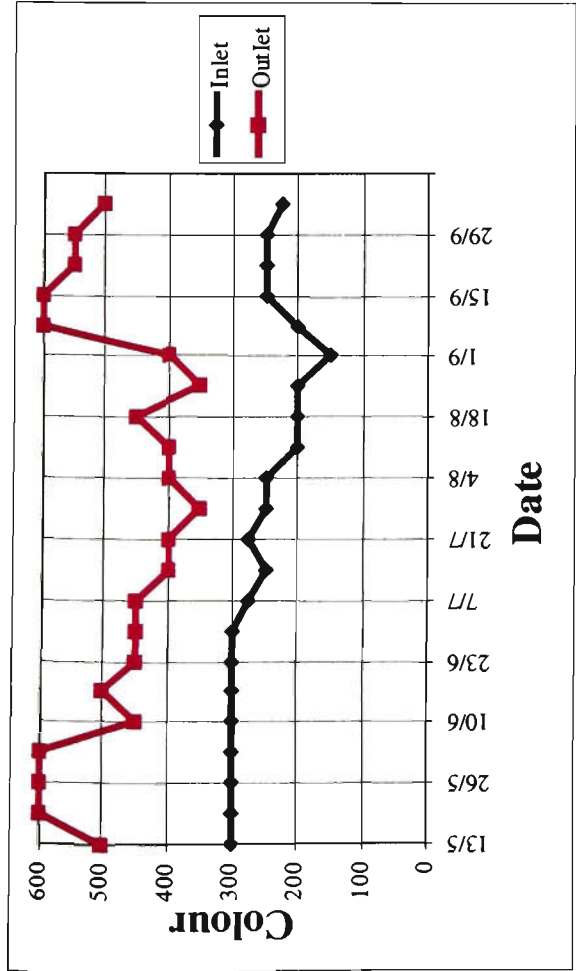
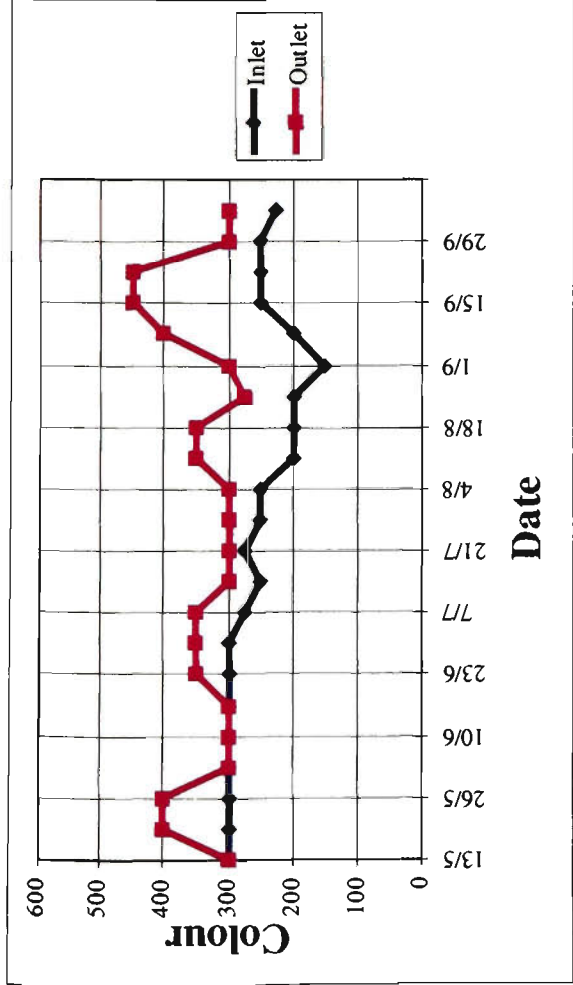
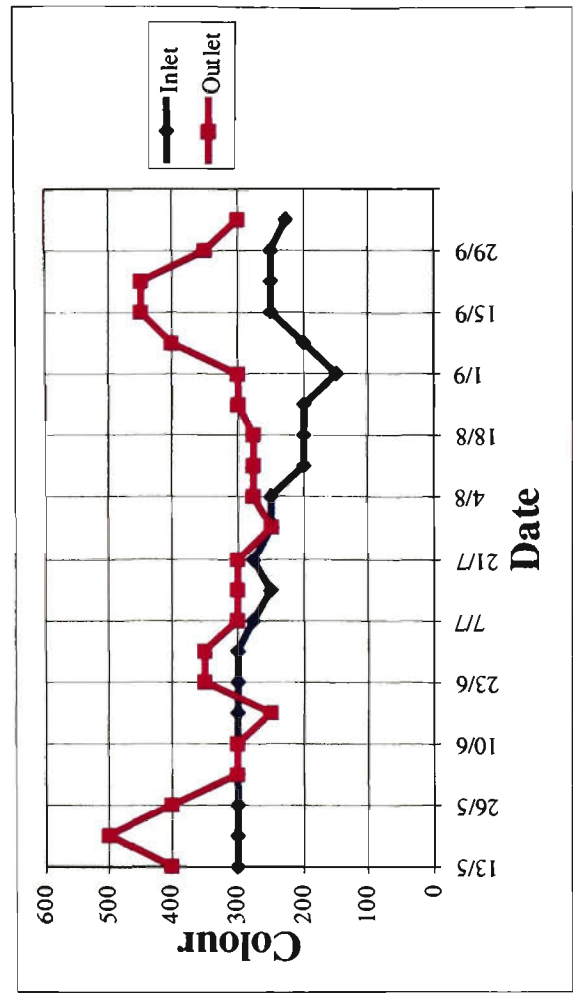


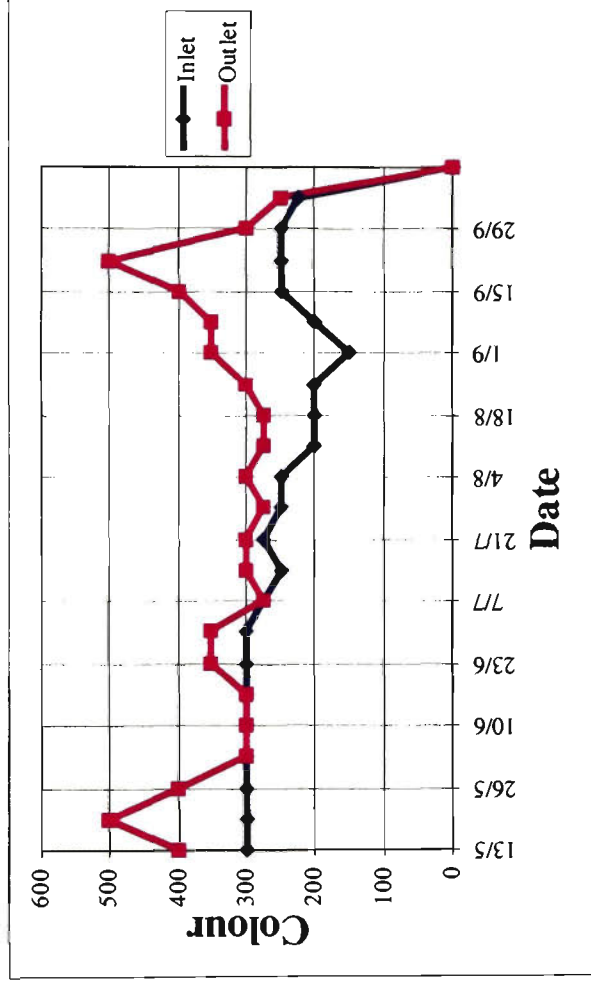
Figure A.6 Colour Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.6 Colour Temporal Plots (Continued)

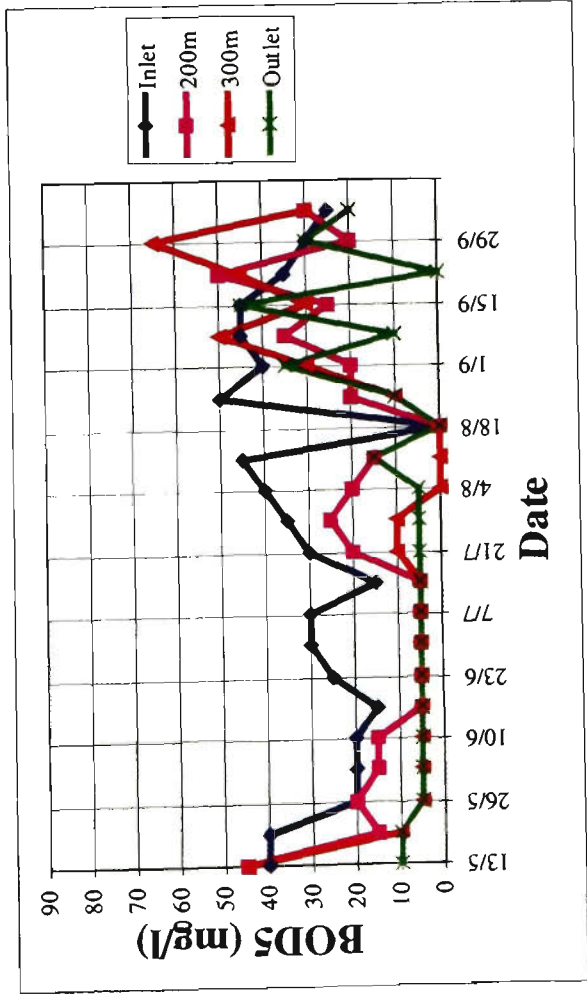
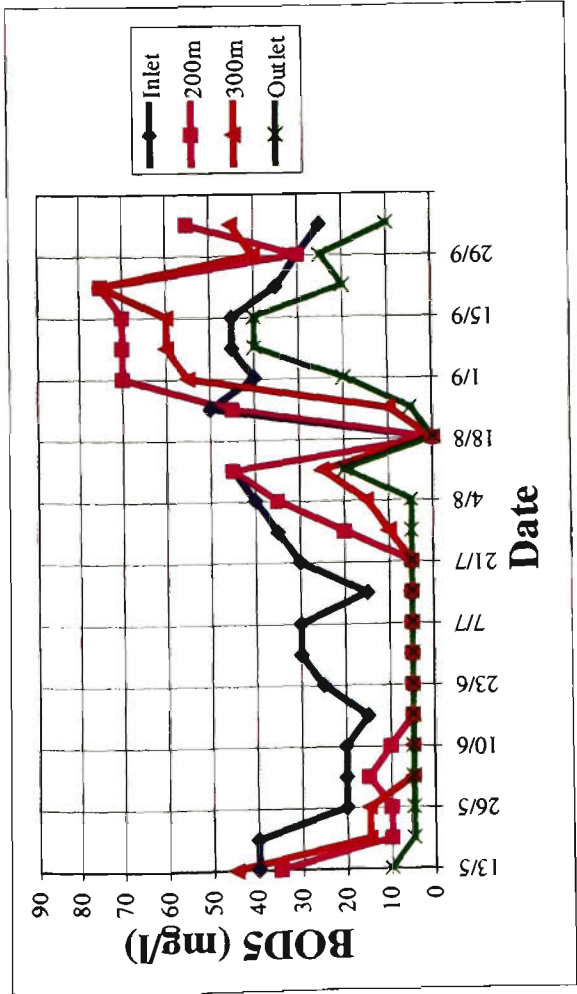
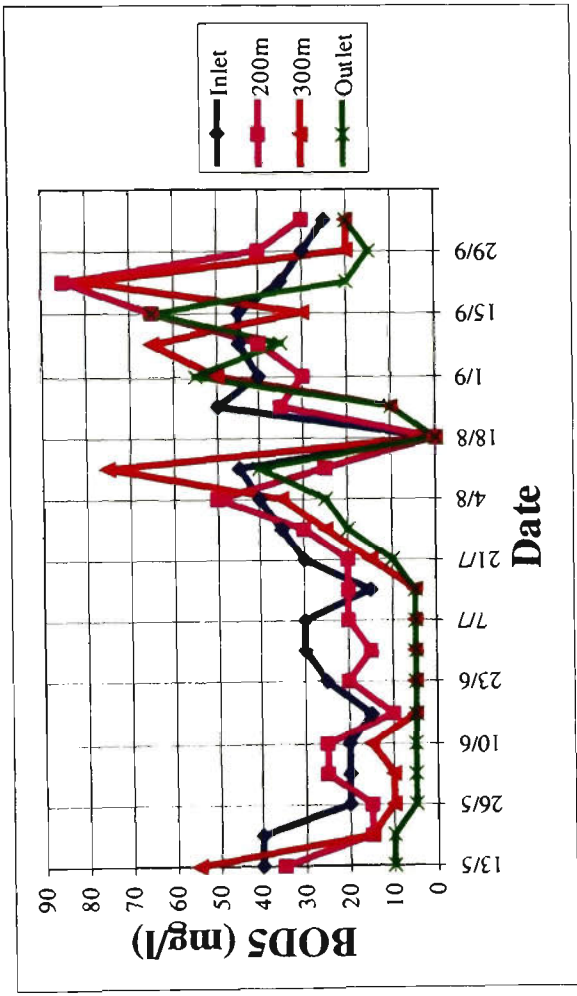
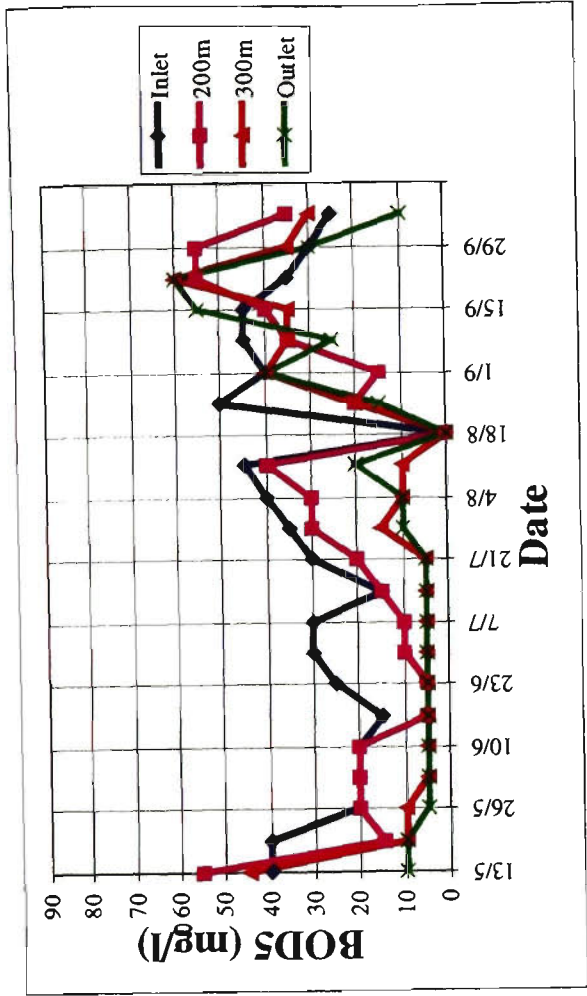
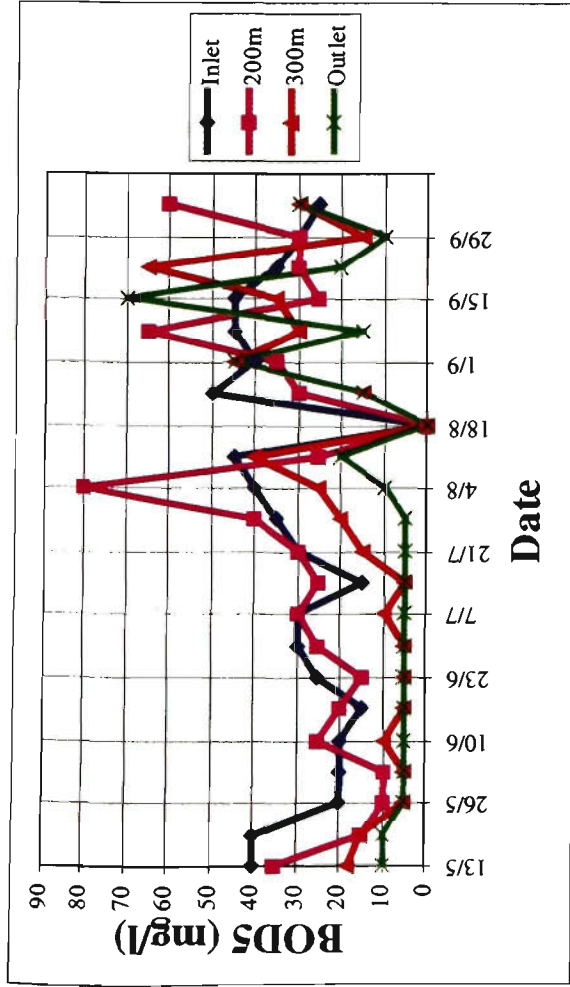
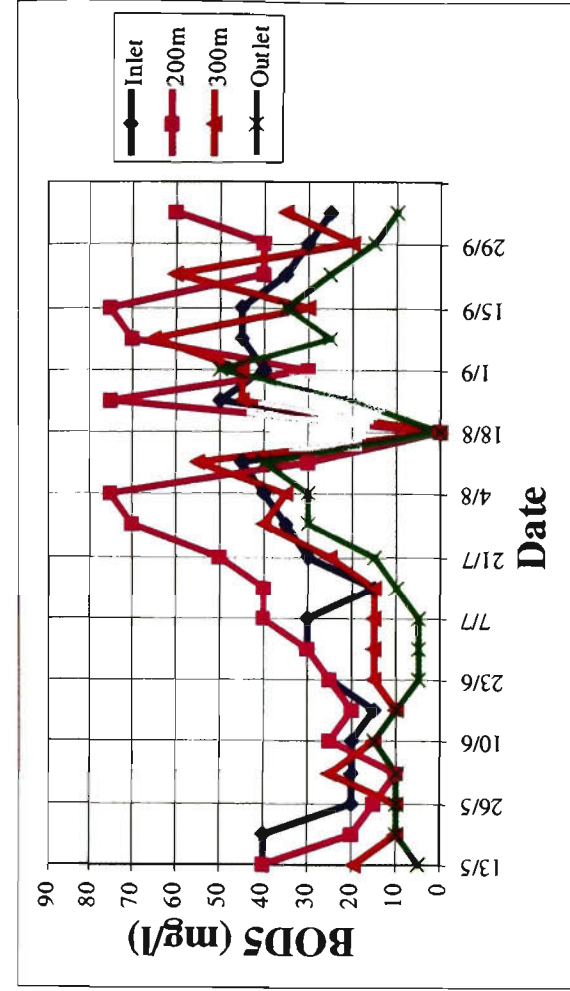


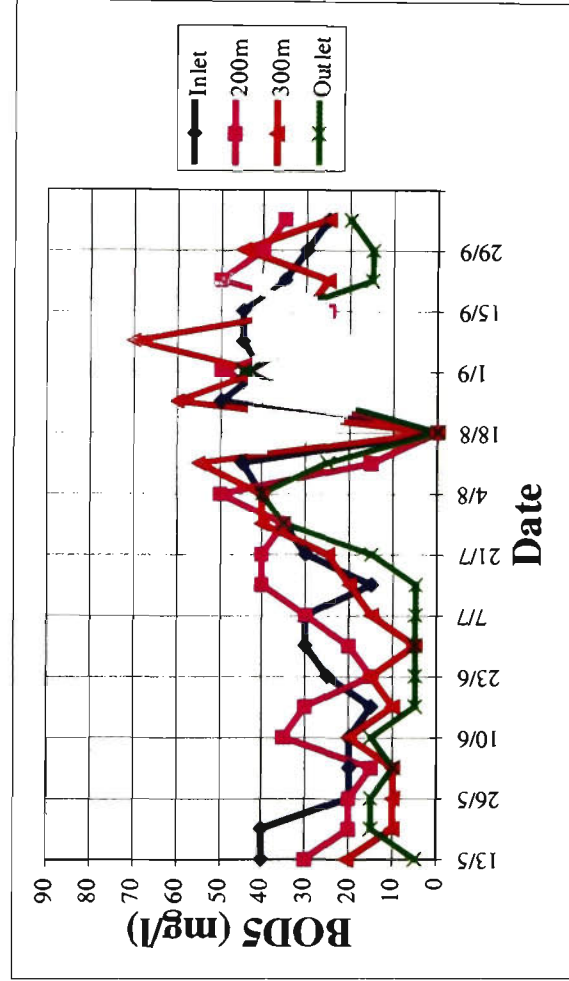
Figure A.7 BOD₅ Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.7 BOD₅ Temporal Plots (Continued...)

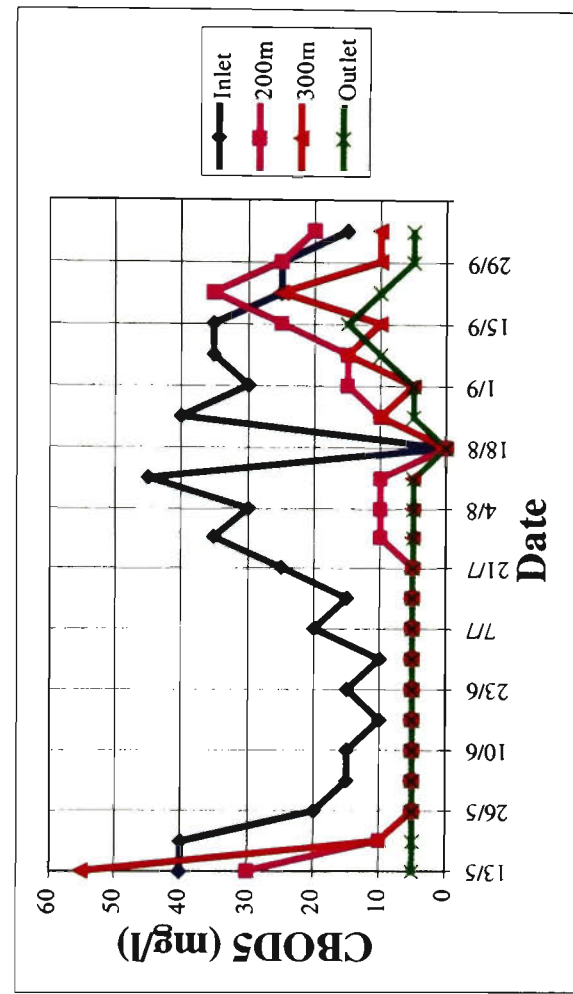
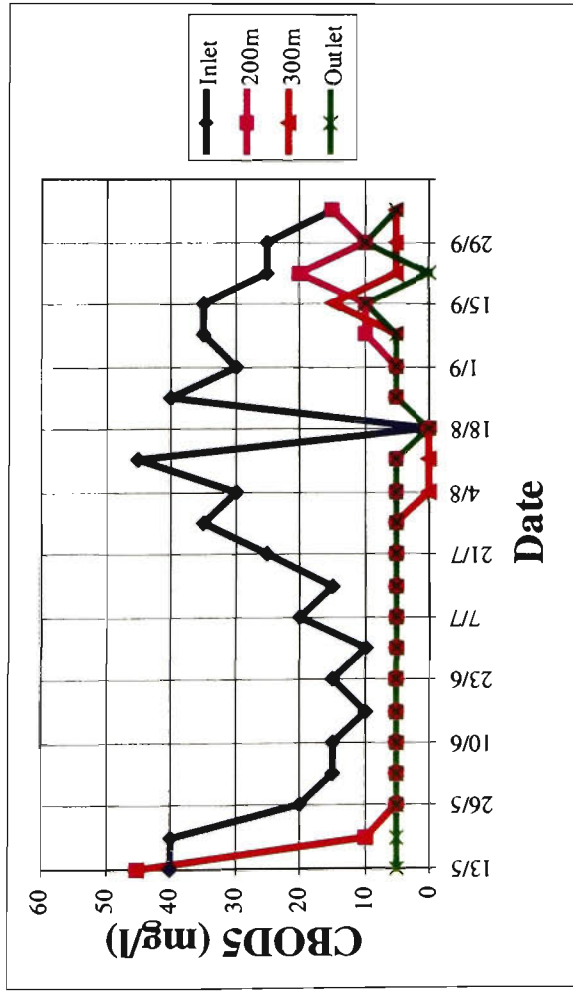
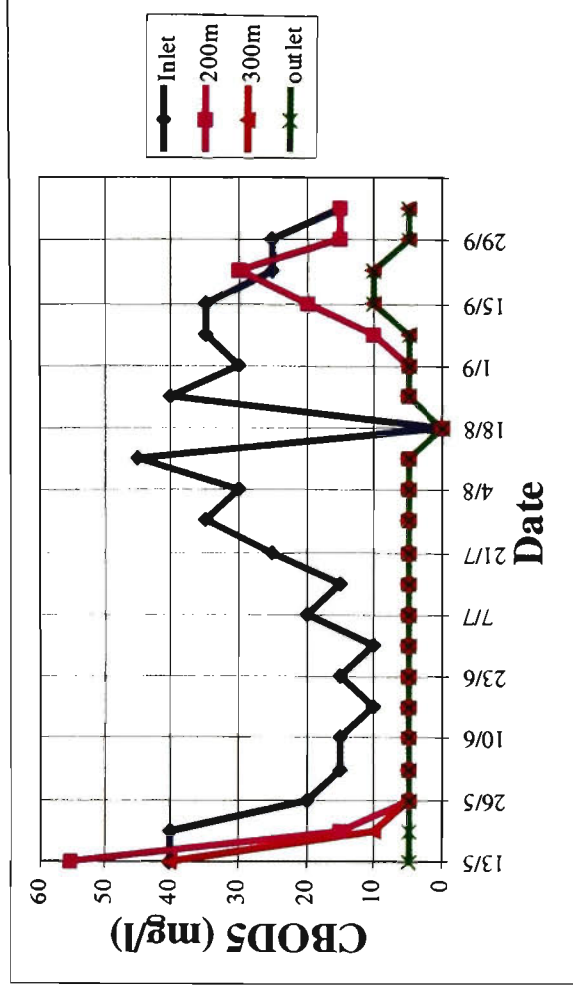
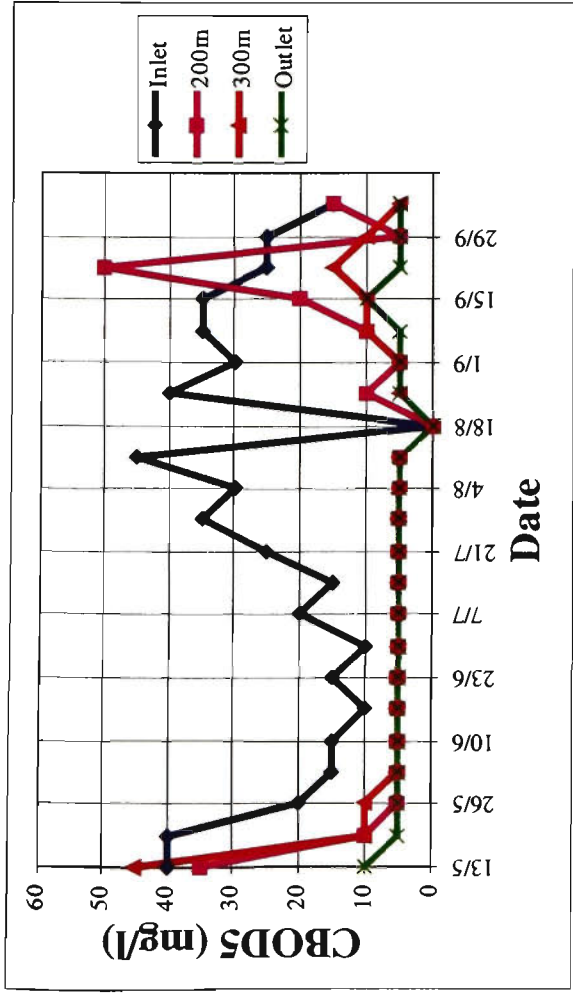
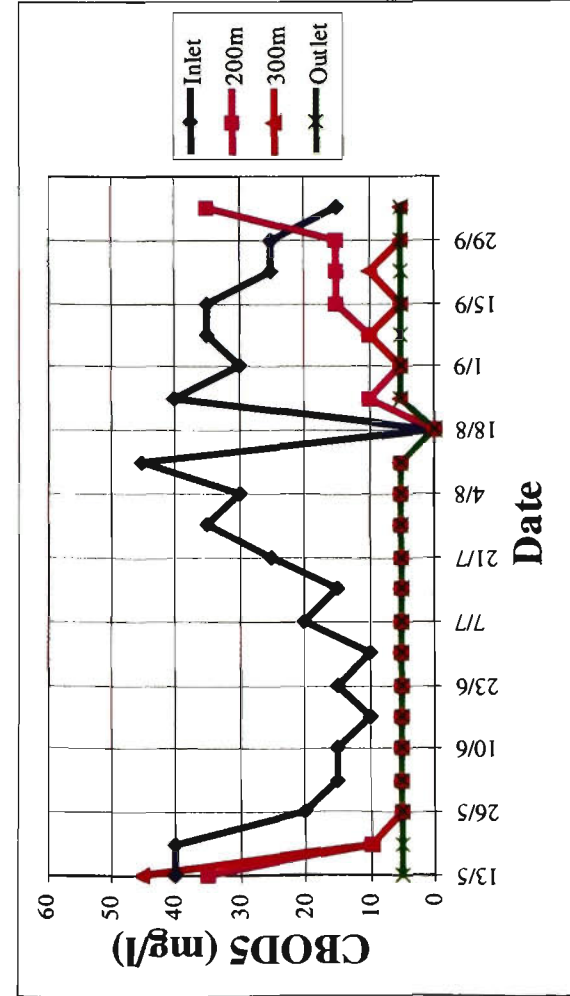
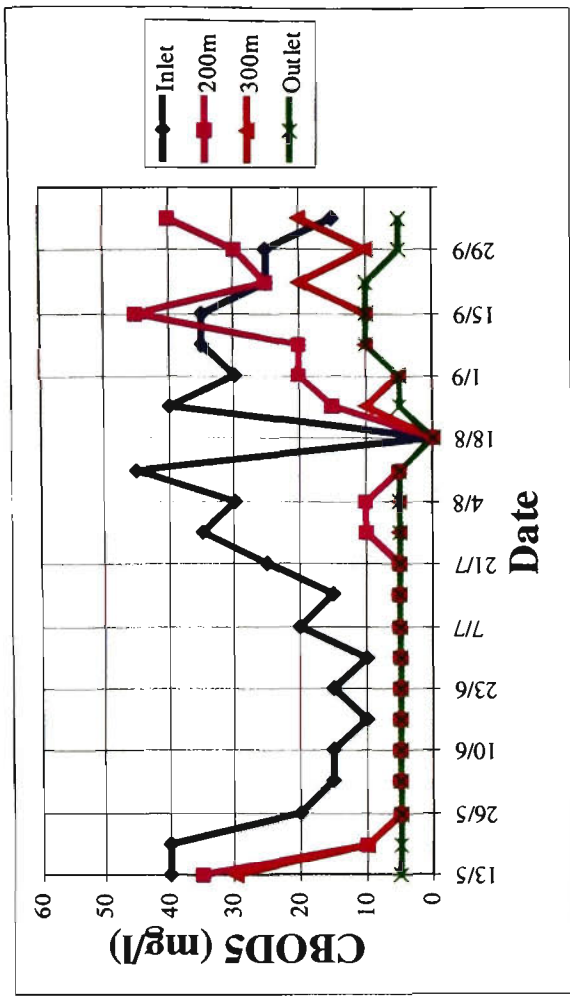


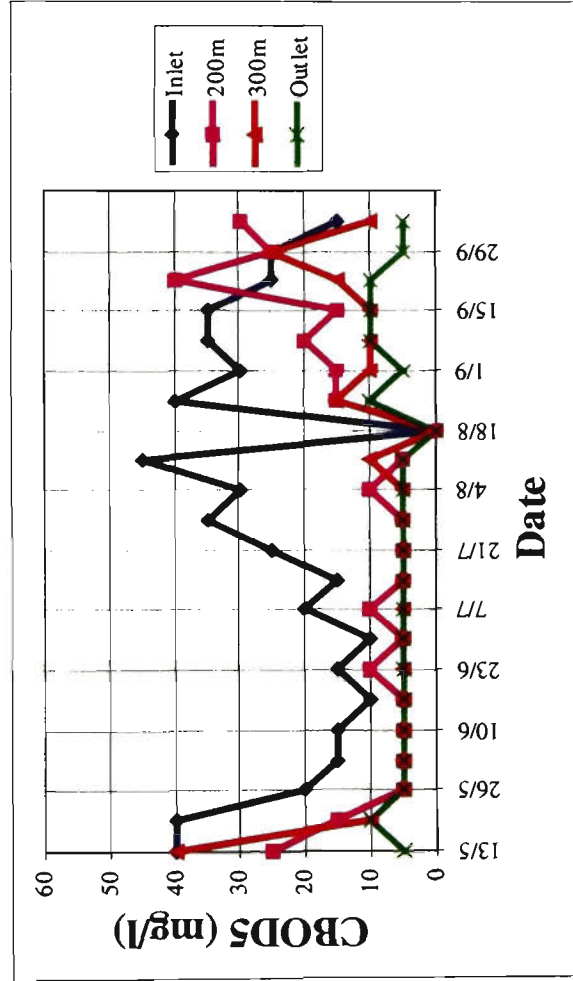
Figure A.8 CBOD₅ Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.8 CBOD₅ Temporal Plots (Continued)

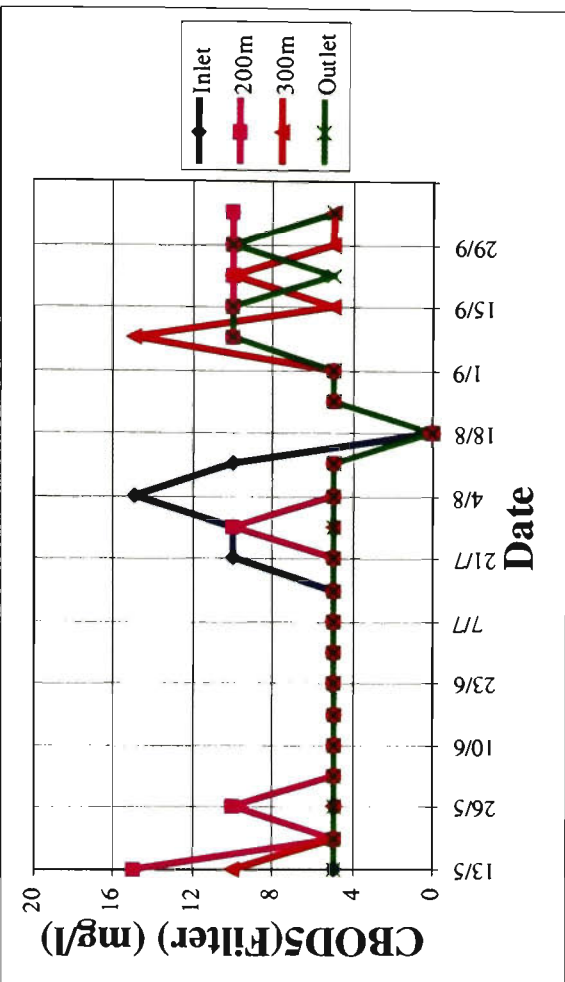
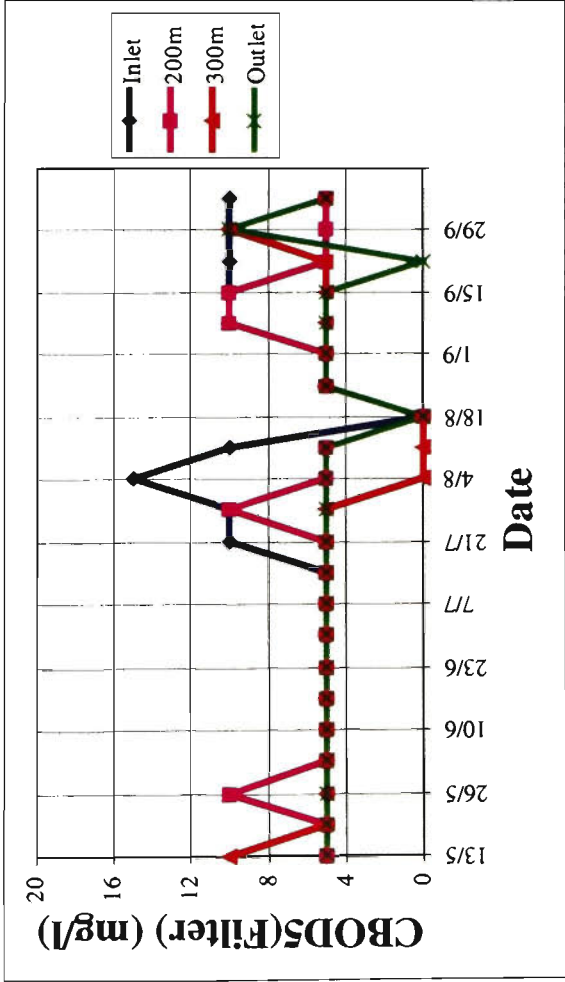
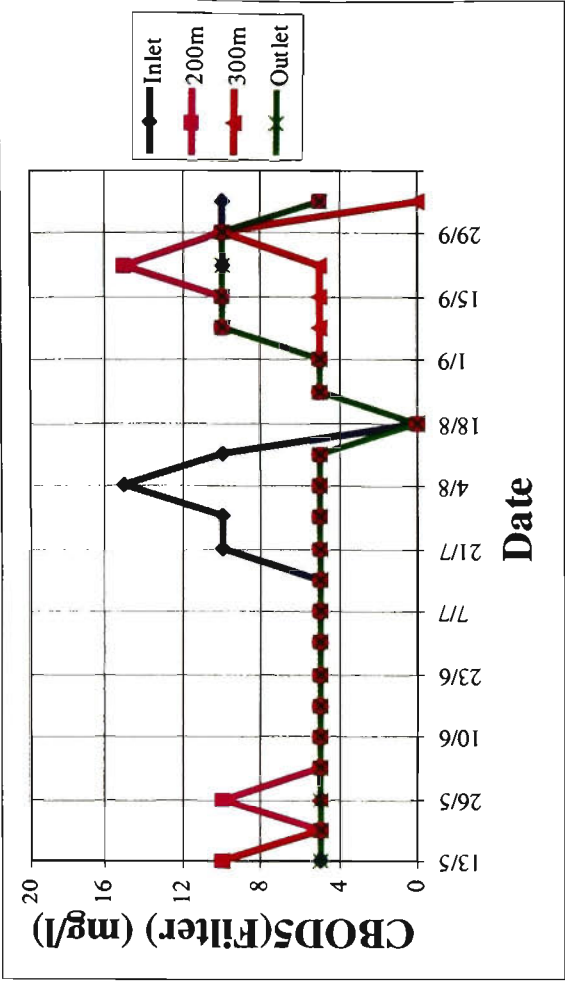
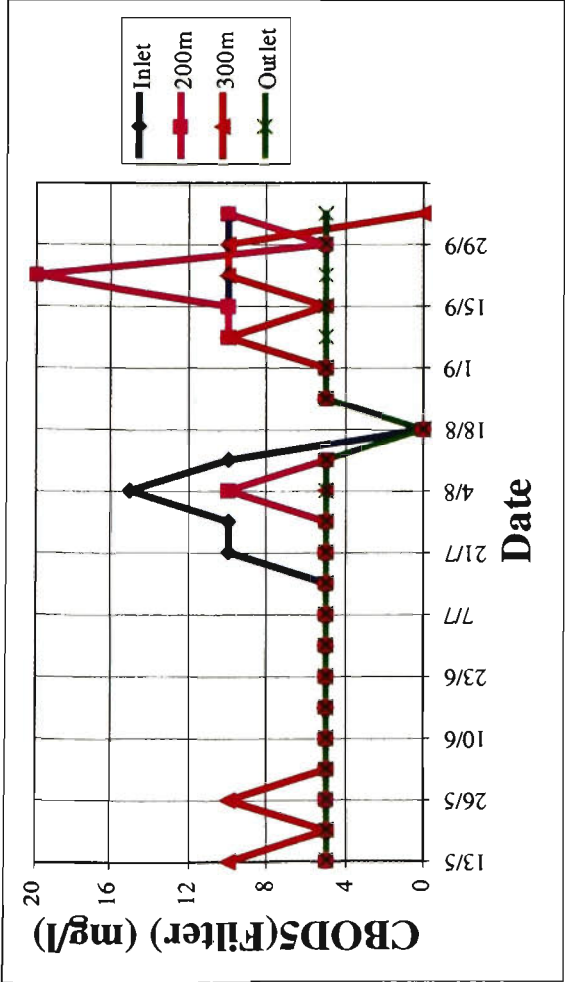
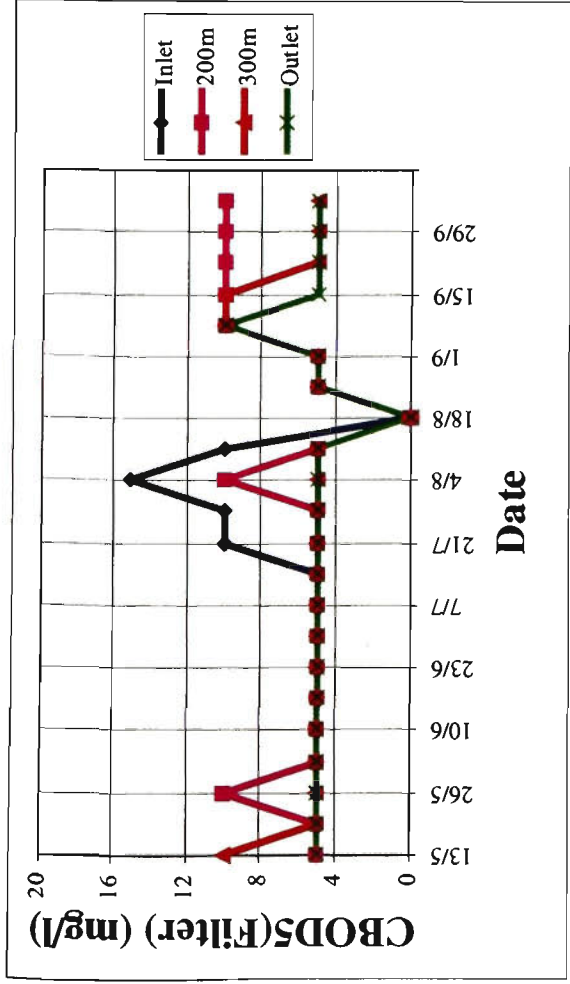
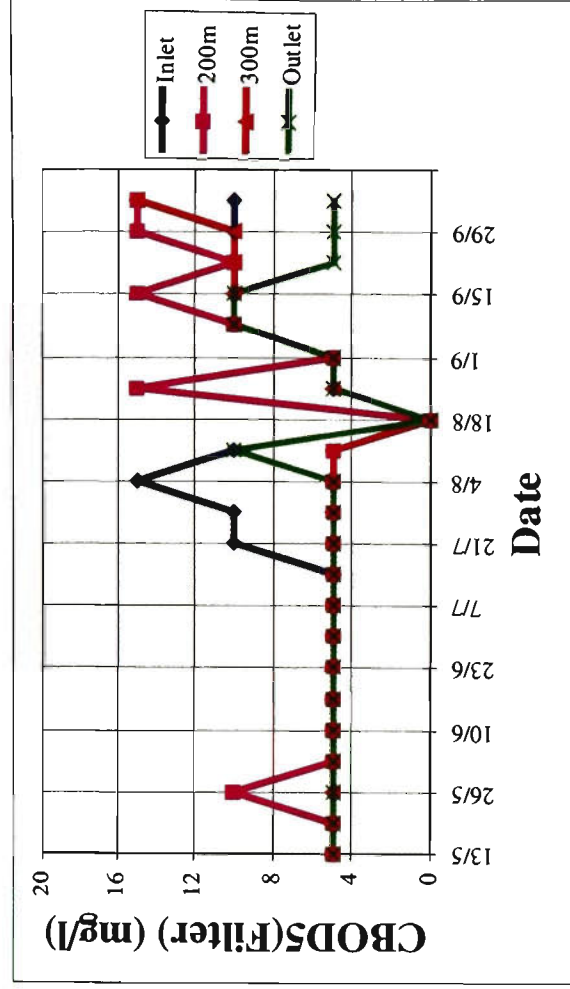


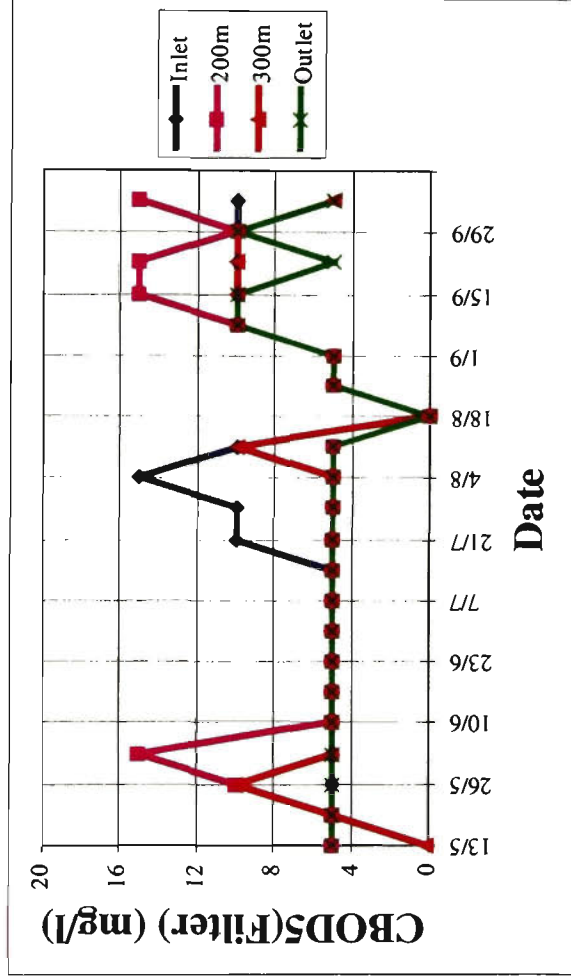
Figure A.9 CBOD₅(Filt) Temporal Plots



Bay 5

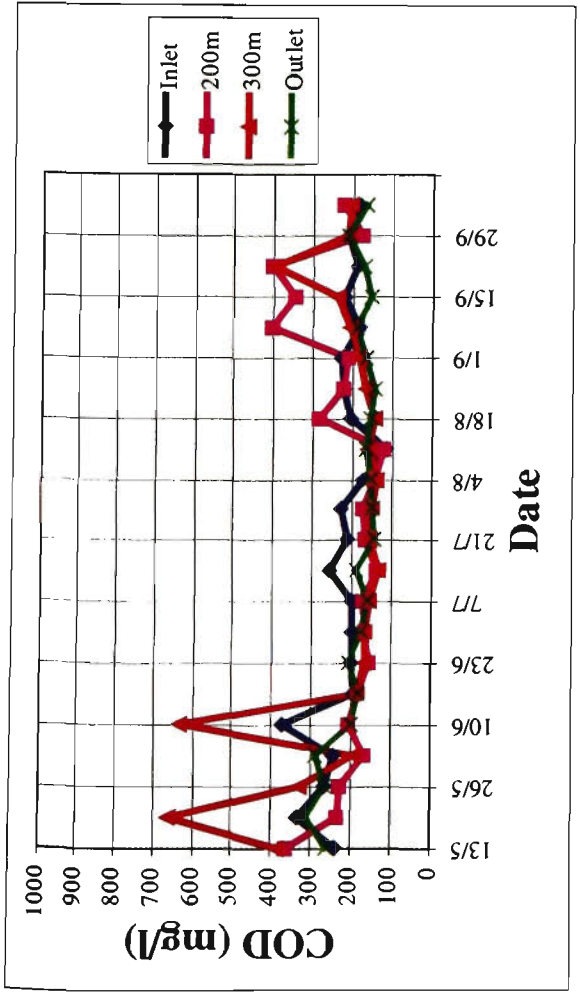


Bay 6

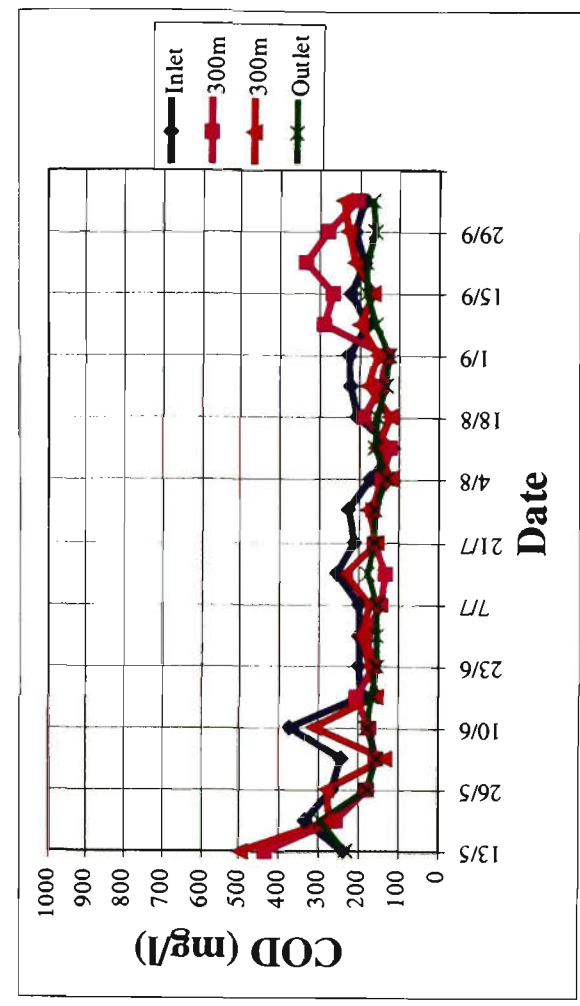


Bay 7

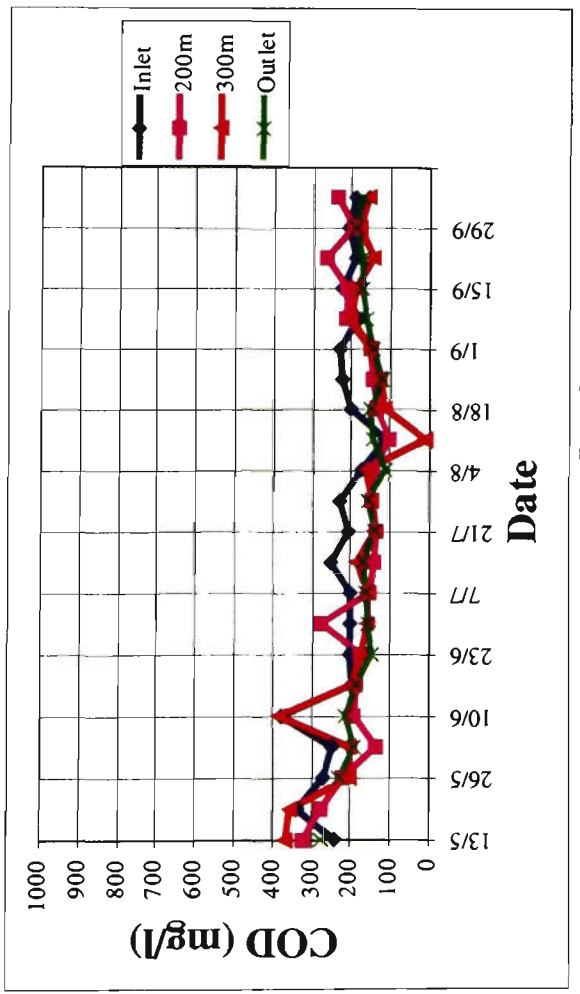
Figure A.9 CBOD₅(Filt) Temporal Plots (Continued)



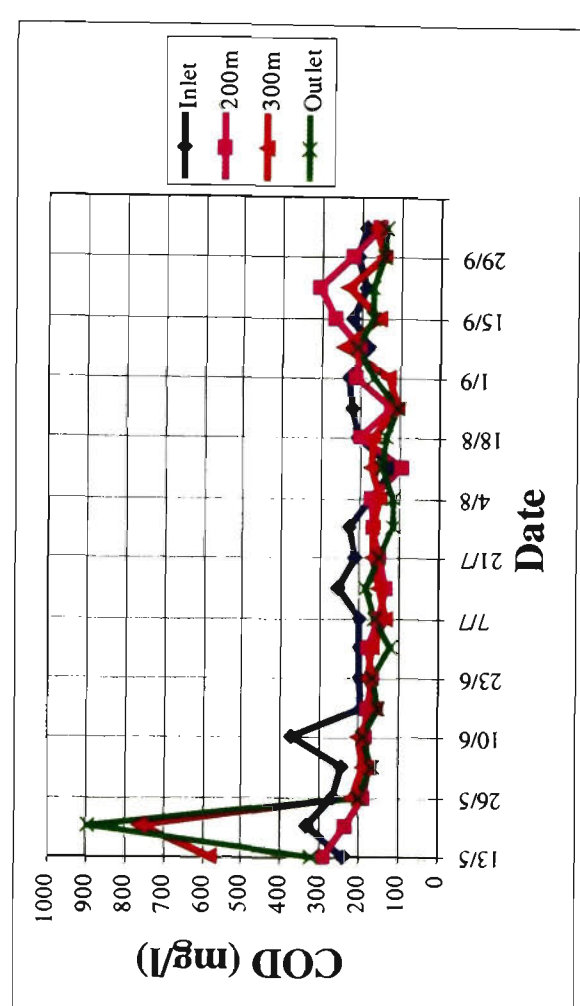
Bay 1



Bay 2

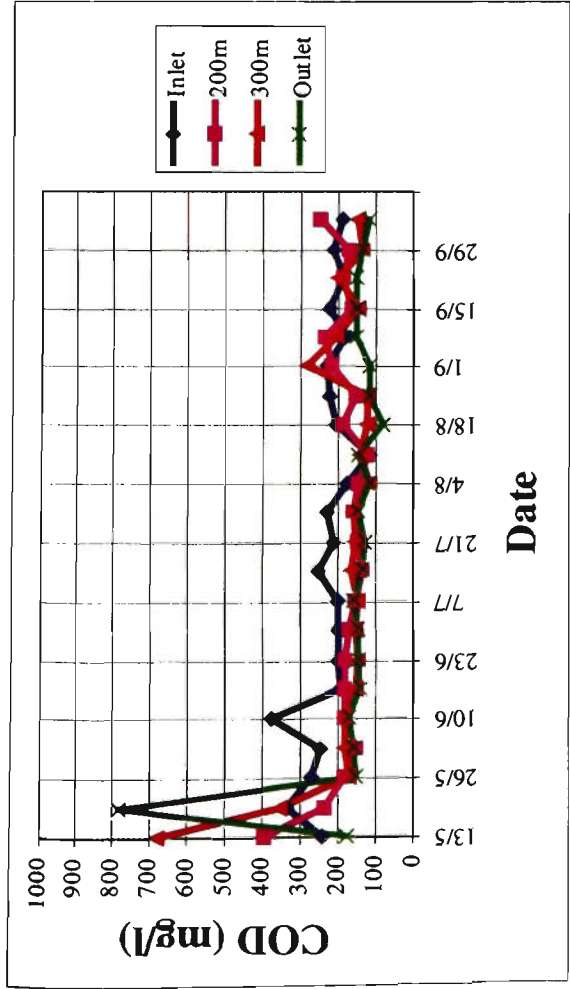


Bay 3

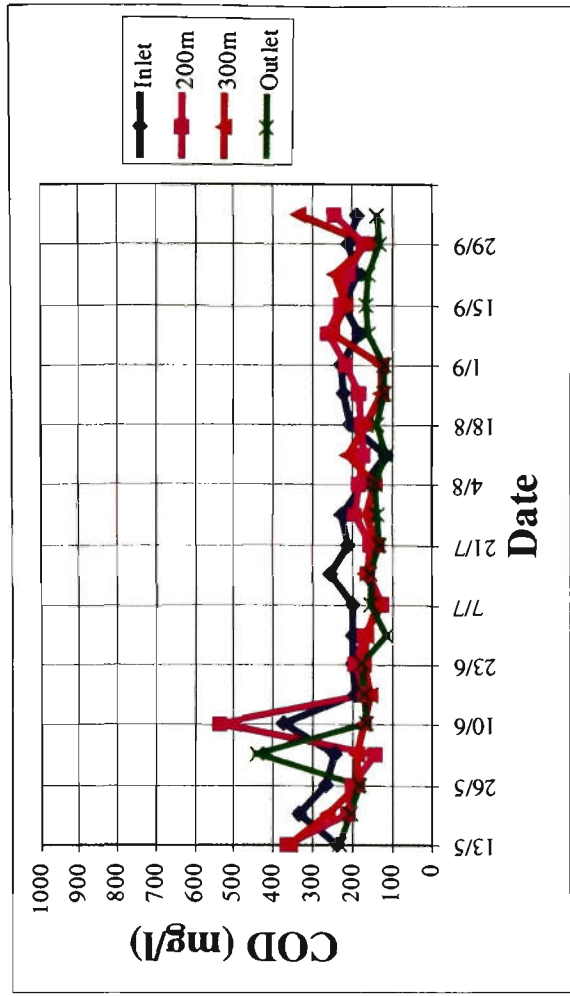


Bay 4

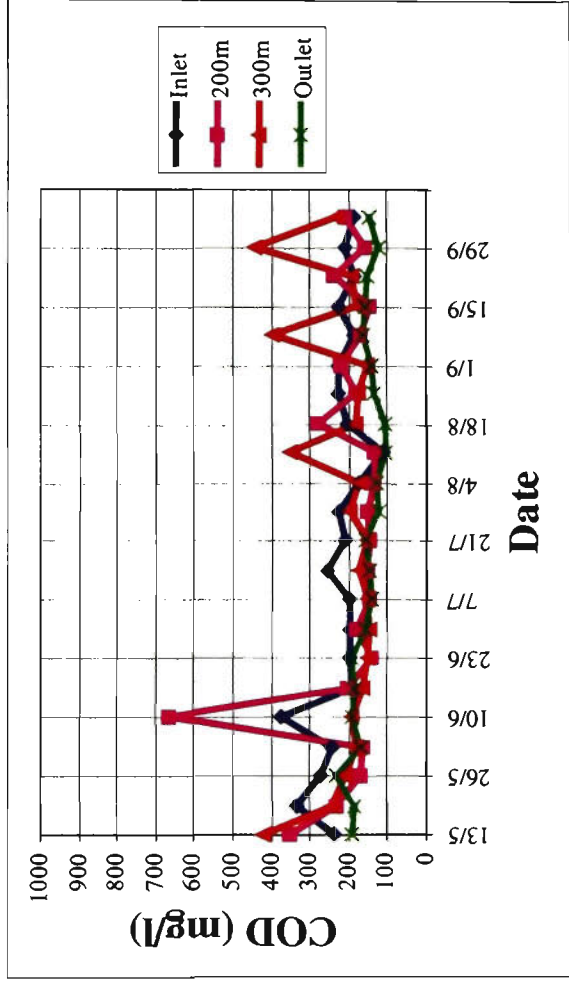
Figure A.10 COD Temporal Plots



Bay 5

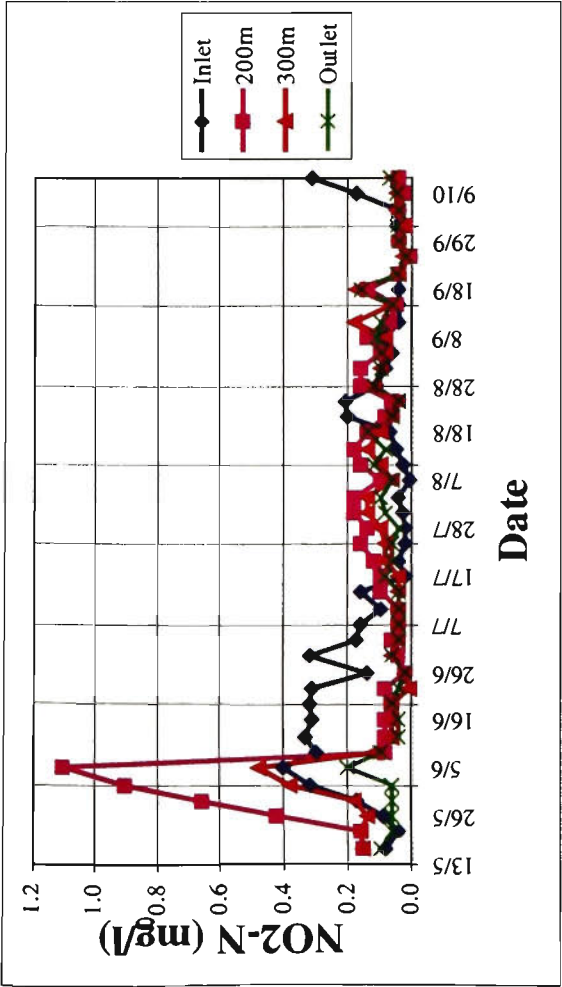


Bay 6

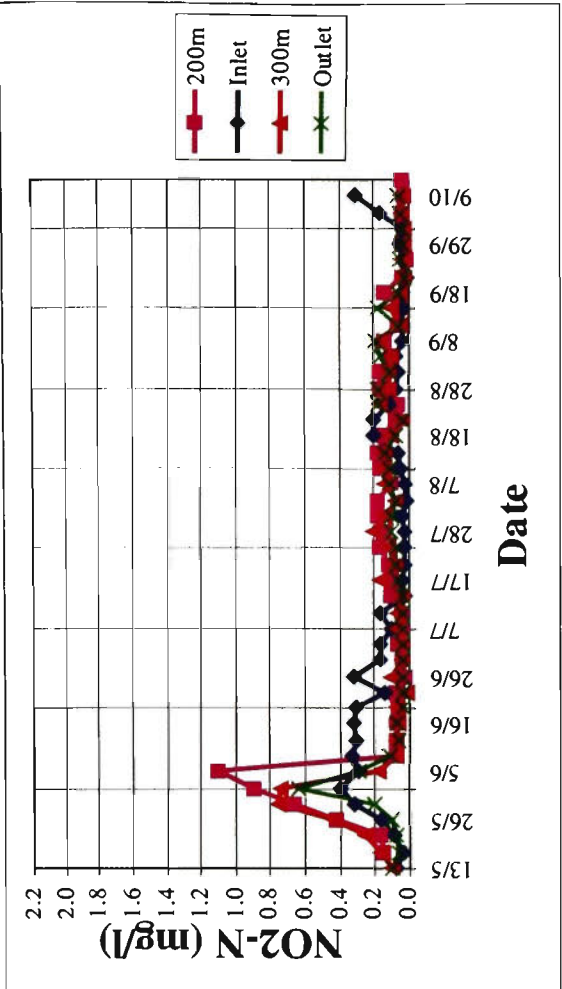


Bay 7

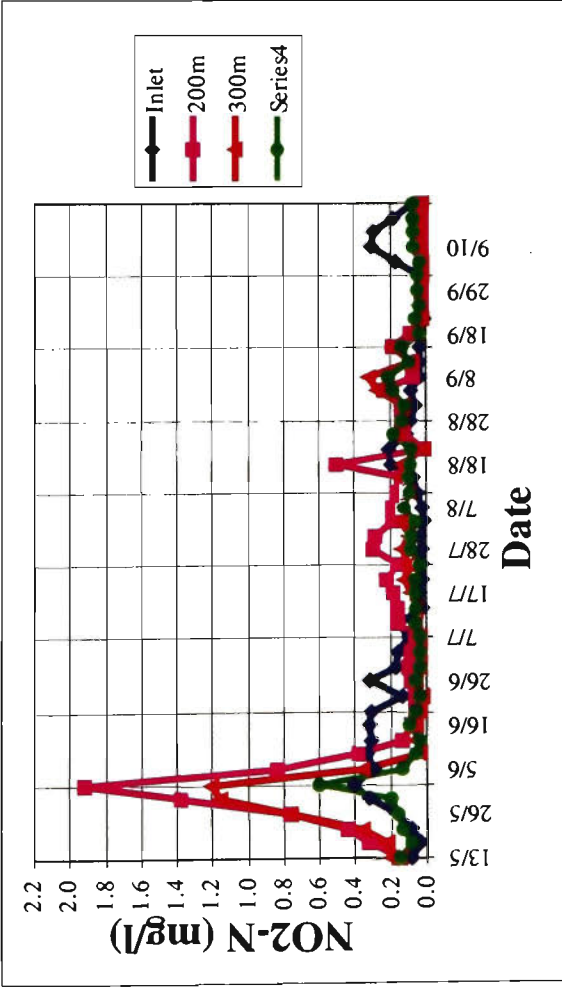
Figure A.10 COD Temporal Plots (Continued)



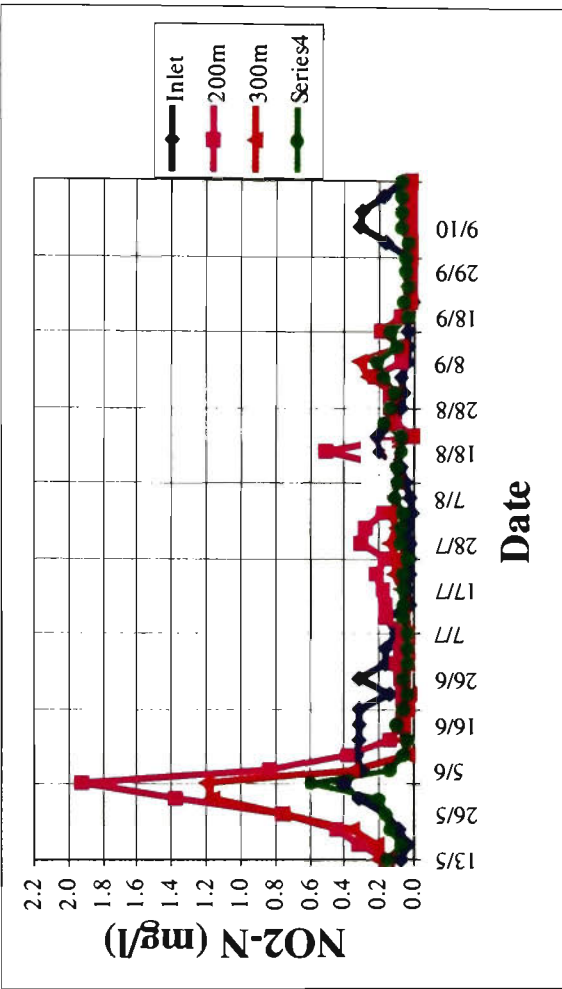
Bay 1



Bay 2

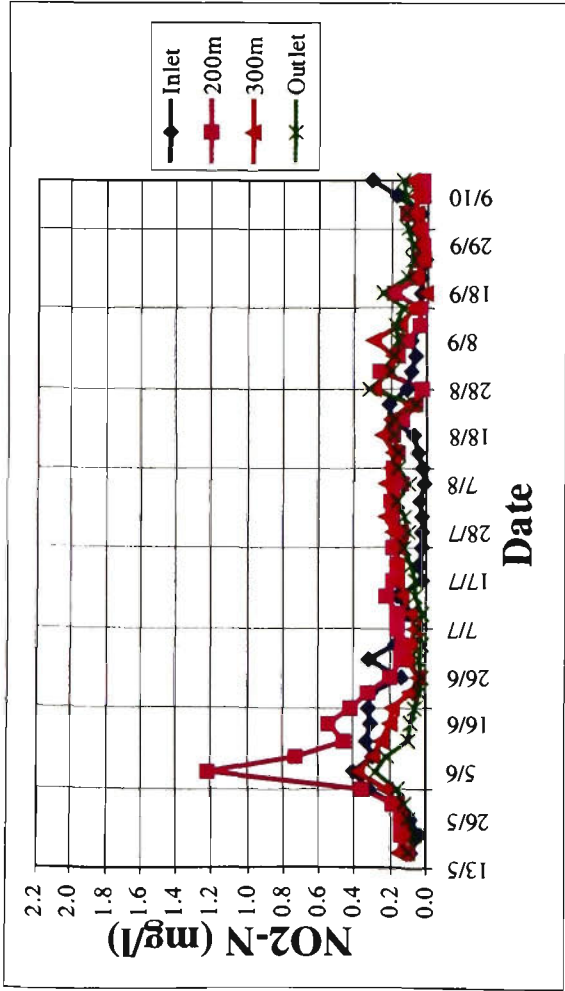


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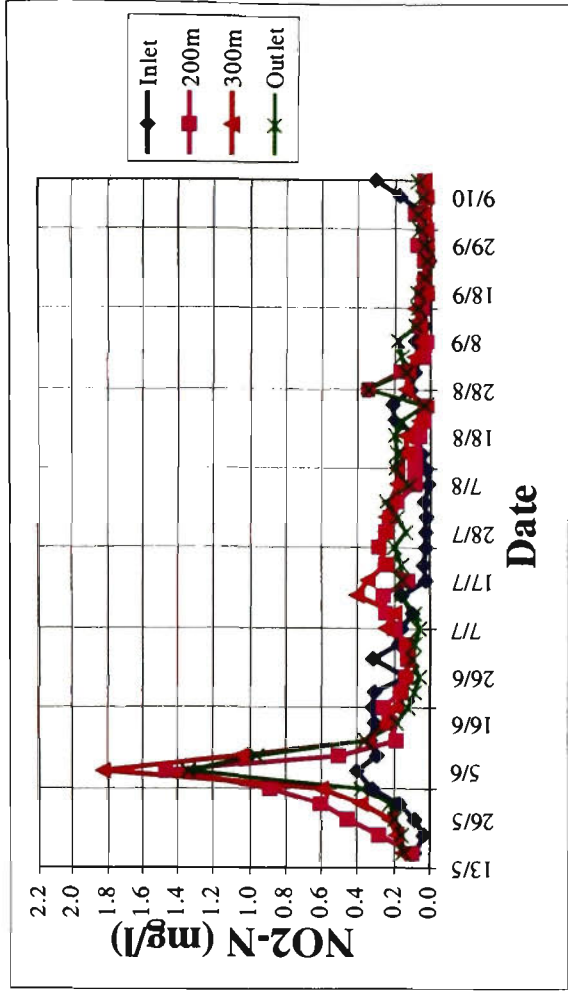


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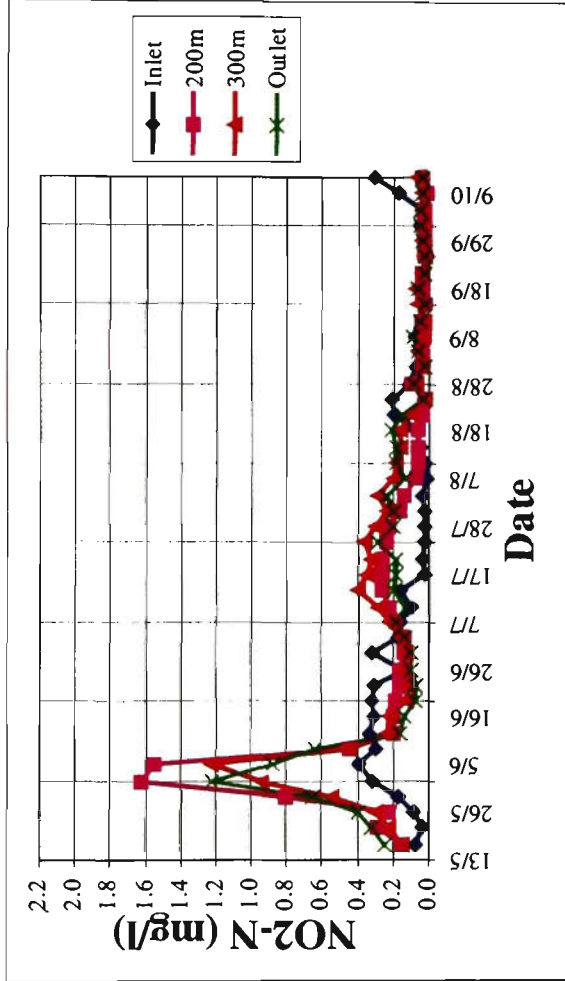
Figure A.11 NO₂-N Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.11 NO₂-N temporal Plots (Continued...)

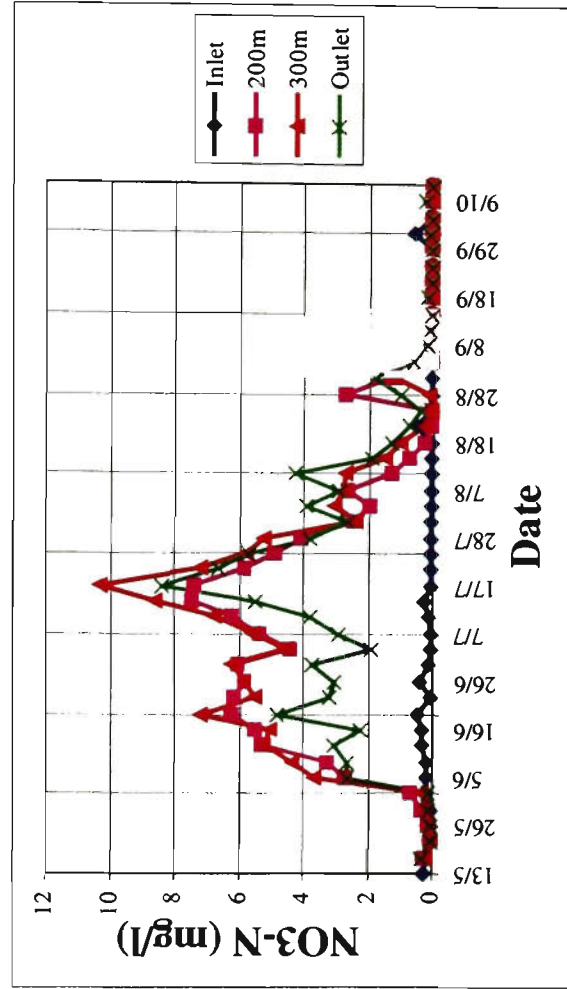
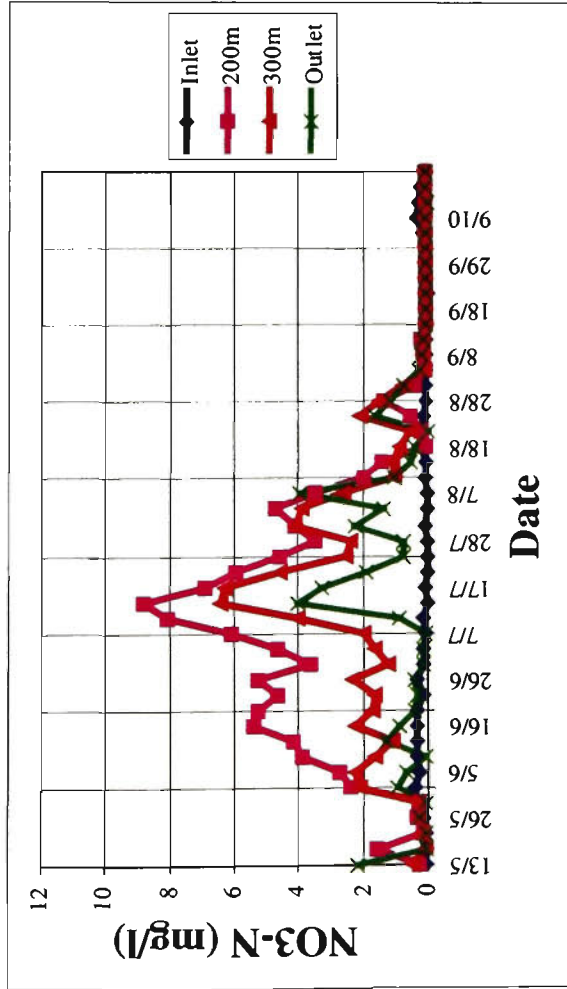
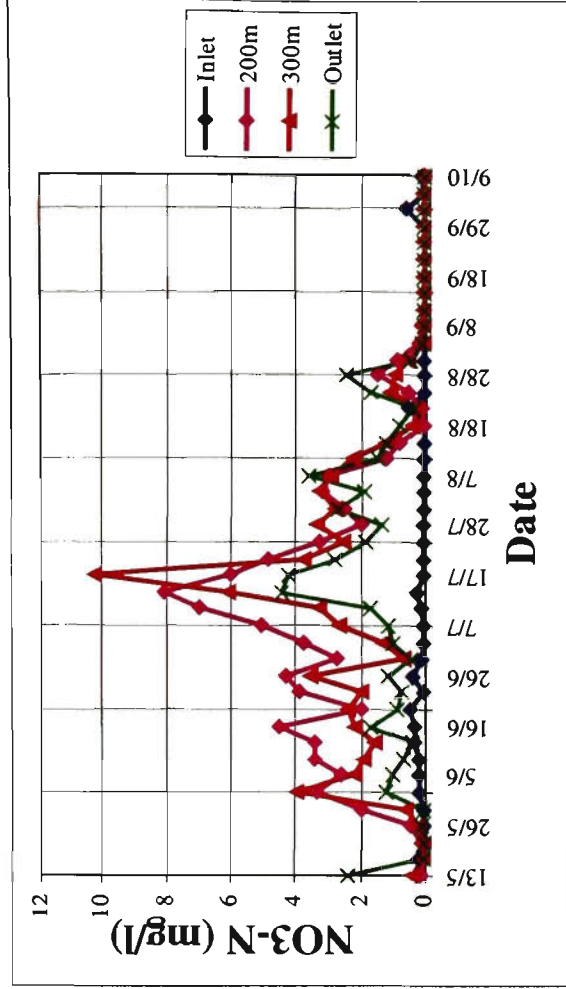
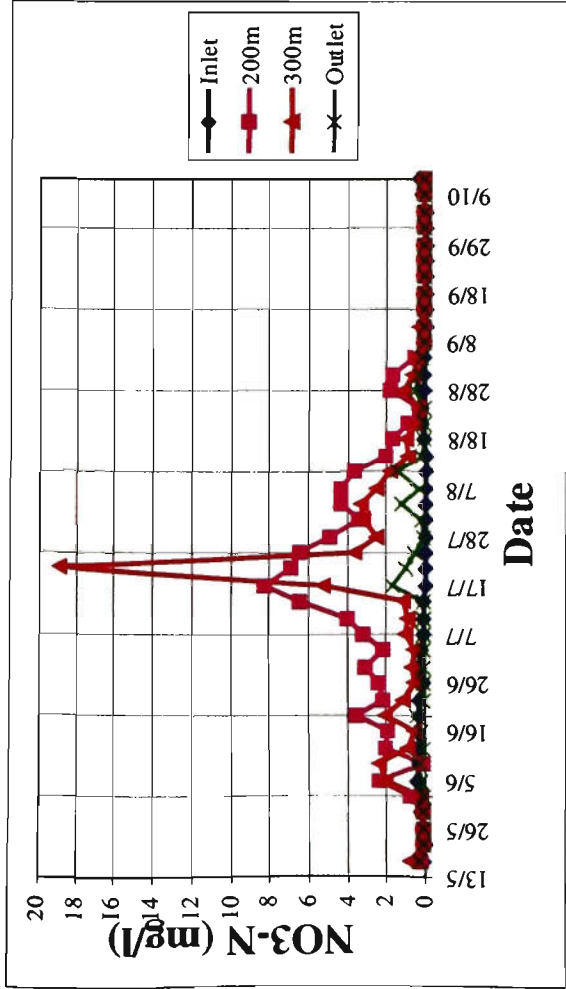
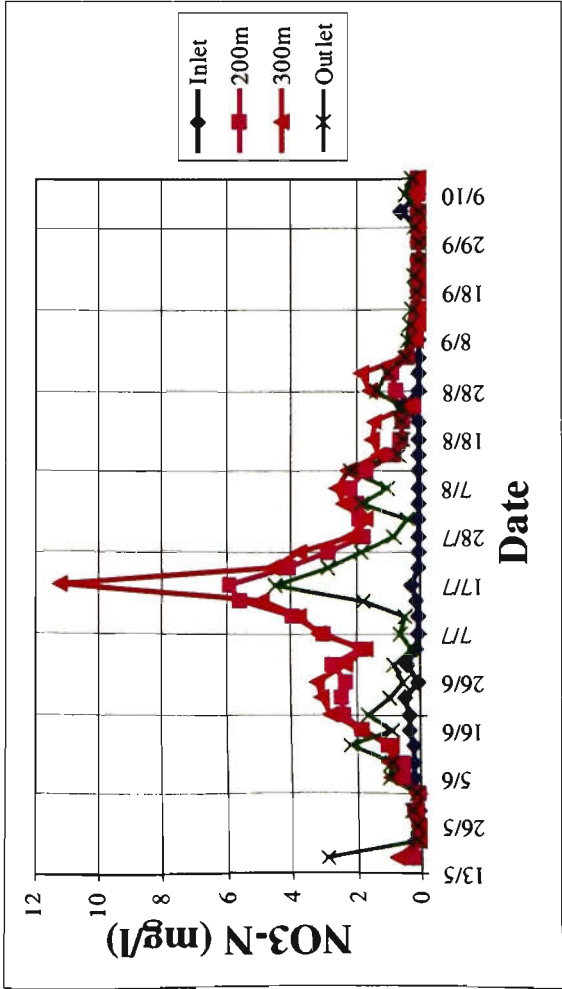
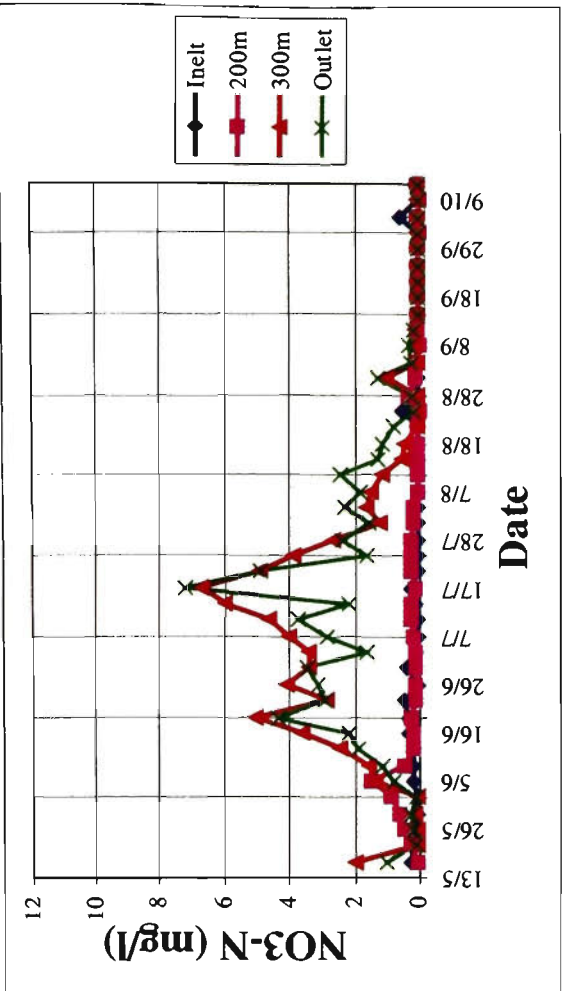


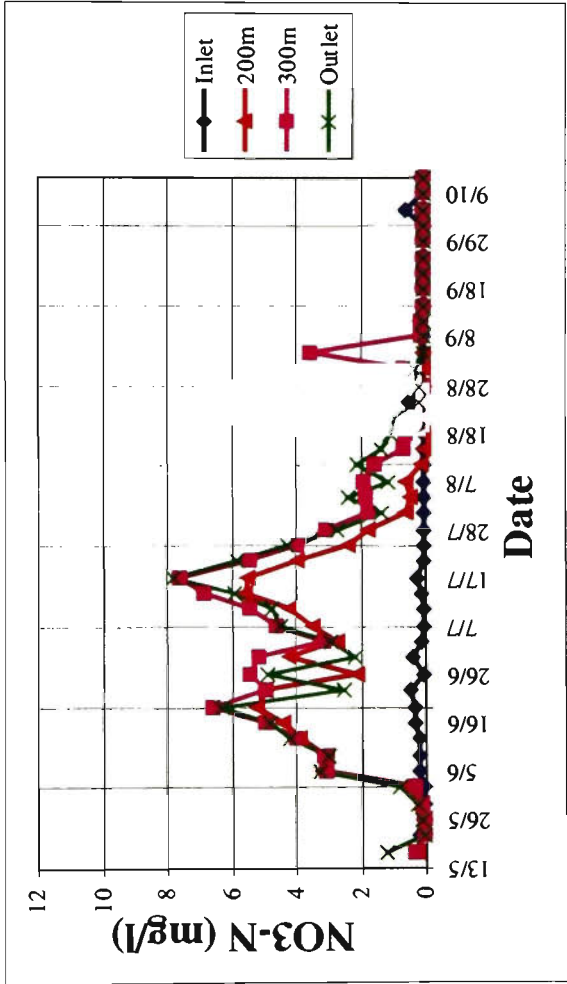
Figure A.12 NO₃-N Temporal Plots



Bay 5

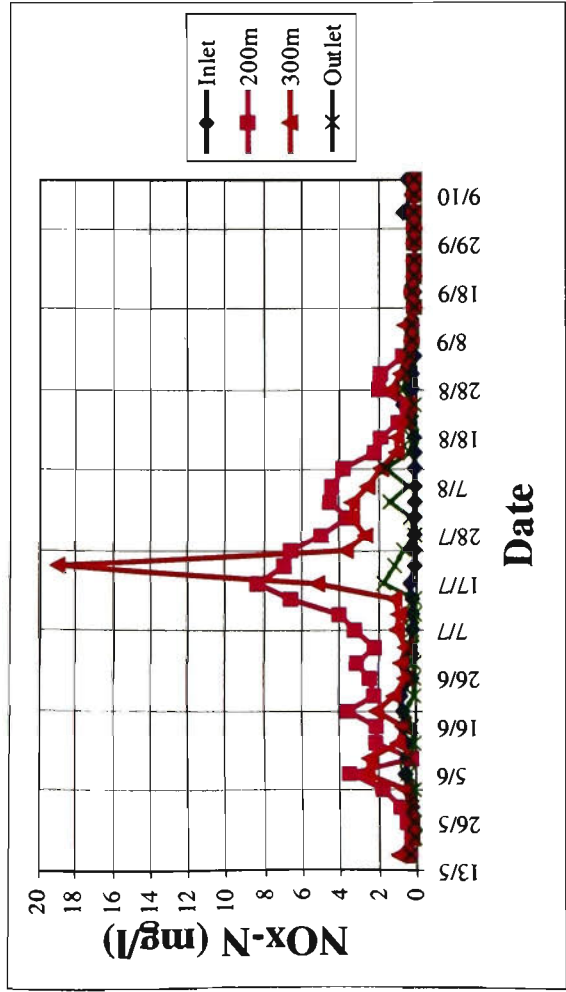


Bay 6

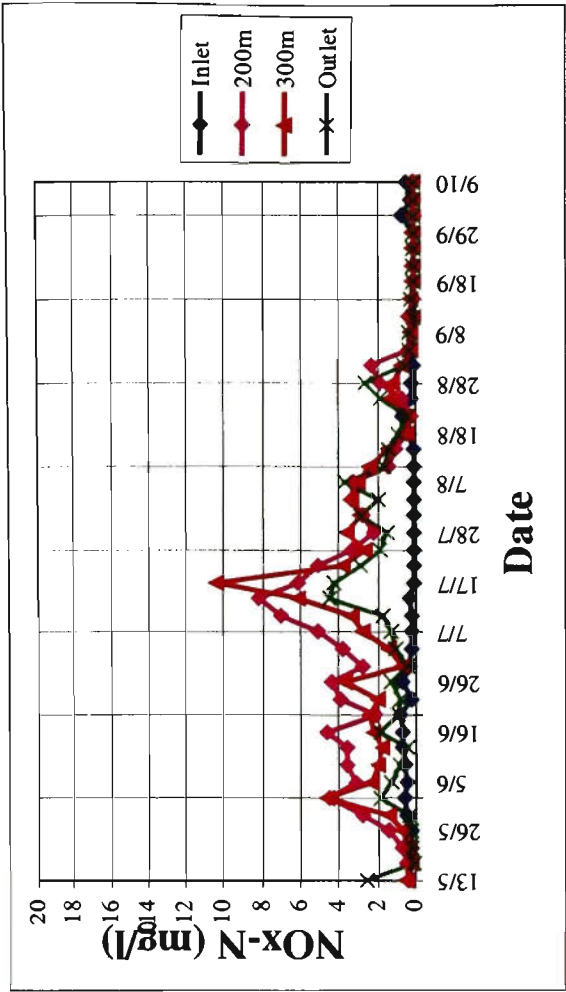


Bay 7

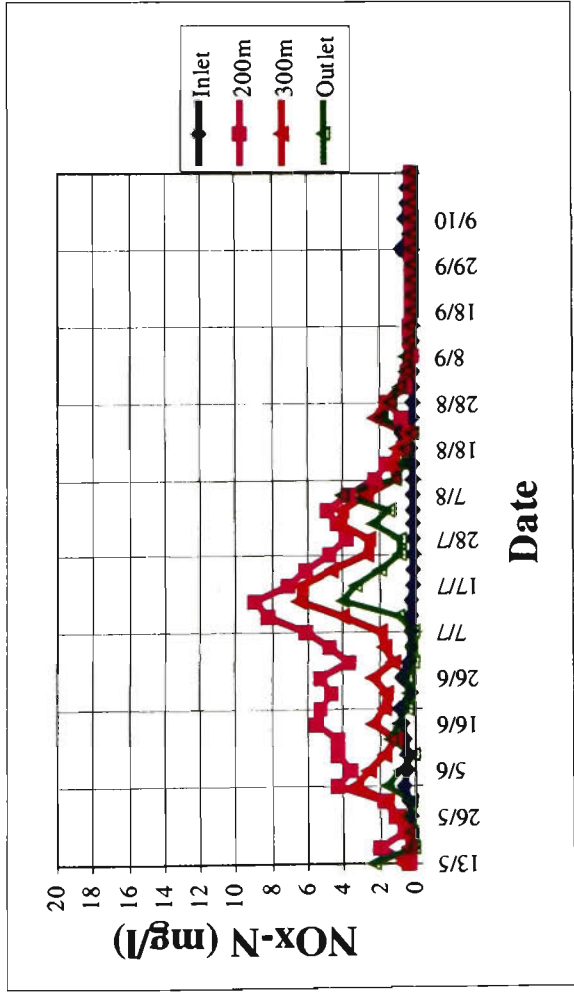
Figure A.12 NO₃-N Temporal Plots (Continued...)



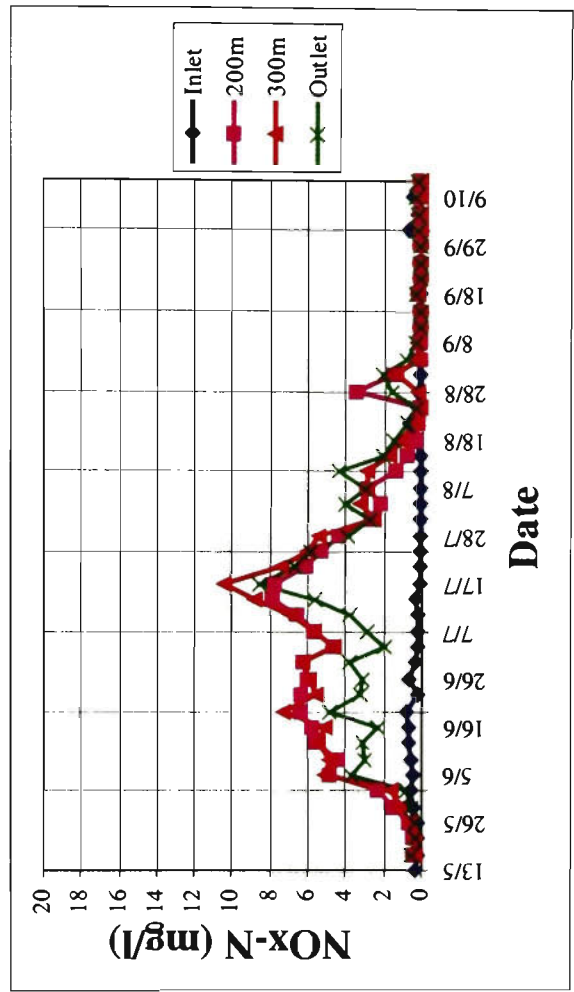
Bay 1



Bay 2

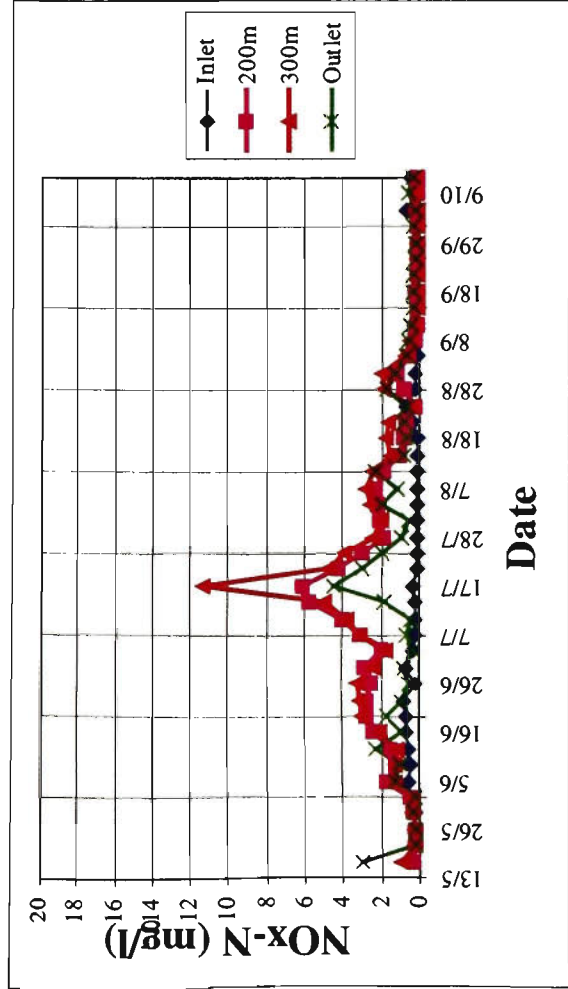


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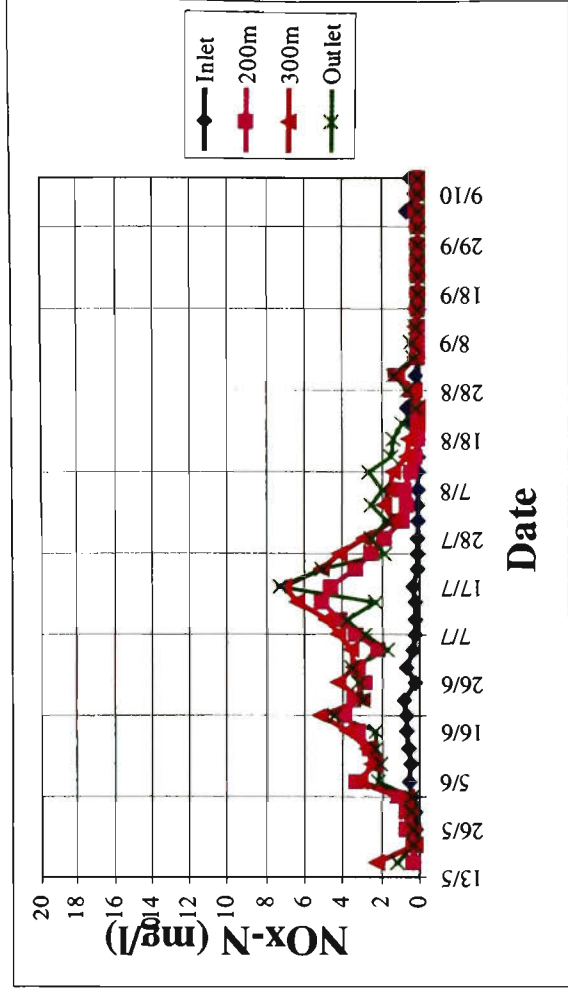


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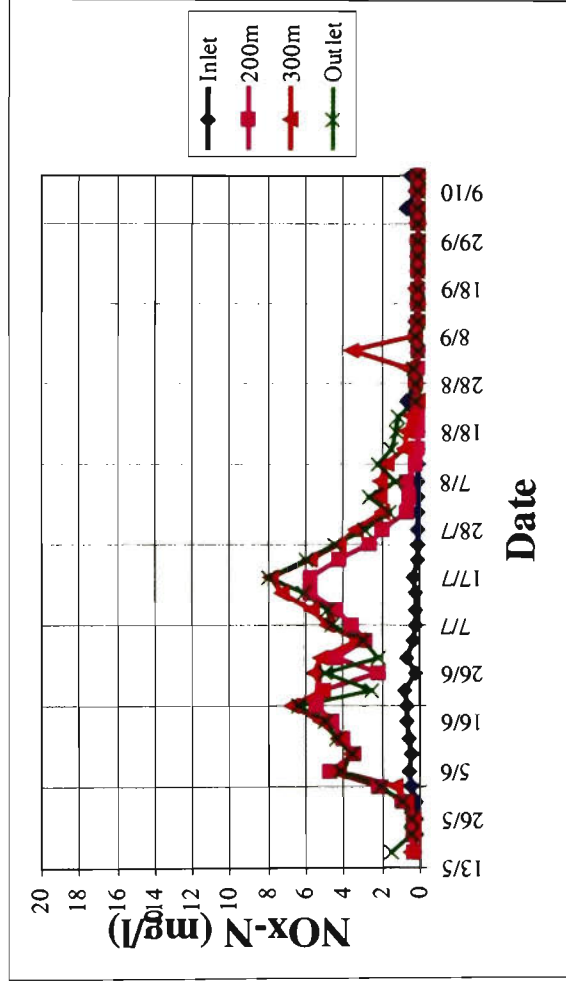
Figure A.13 NO_x-N Temporal Plots



Bay 5

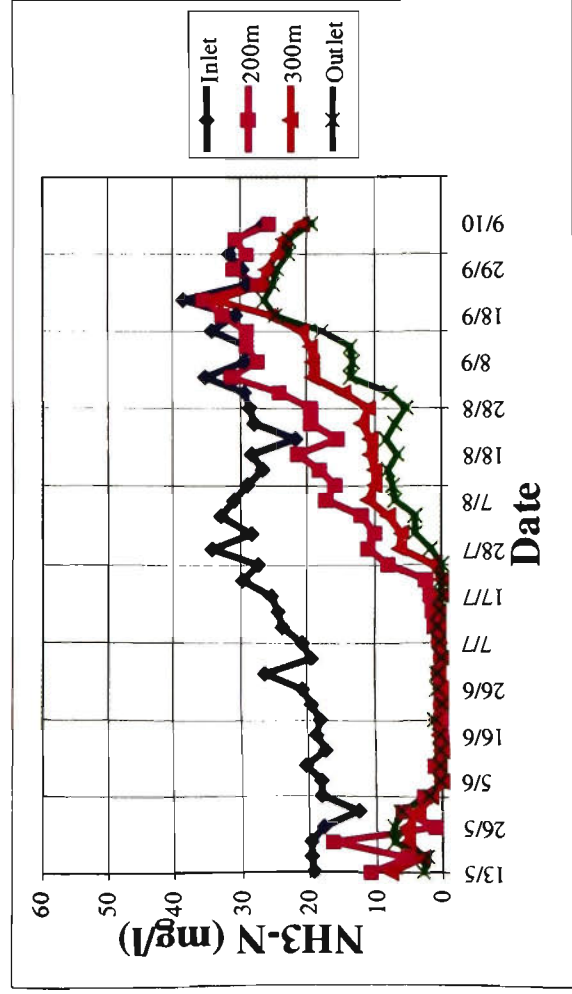


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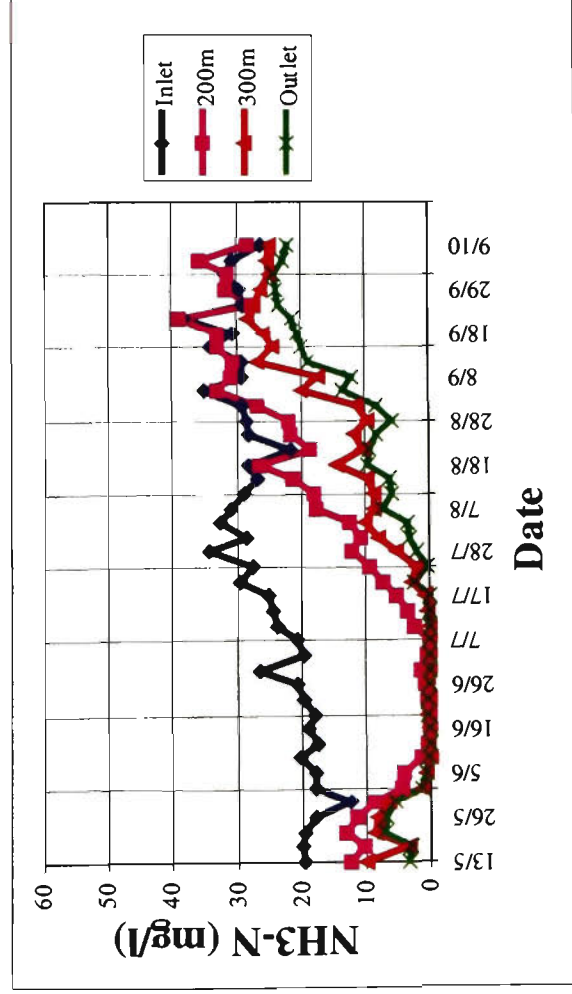


Bay 7

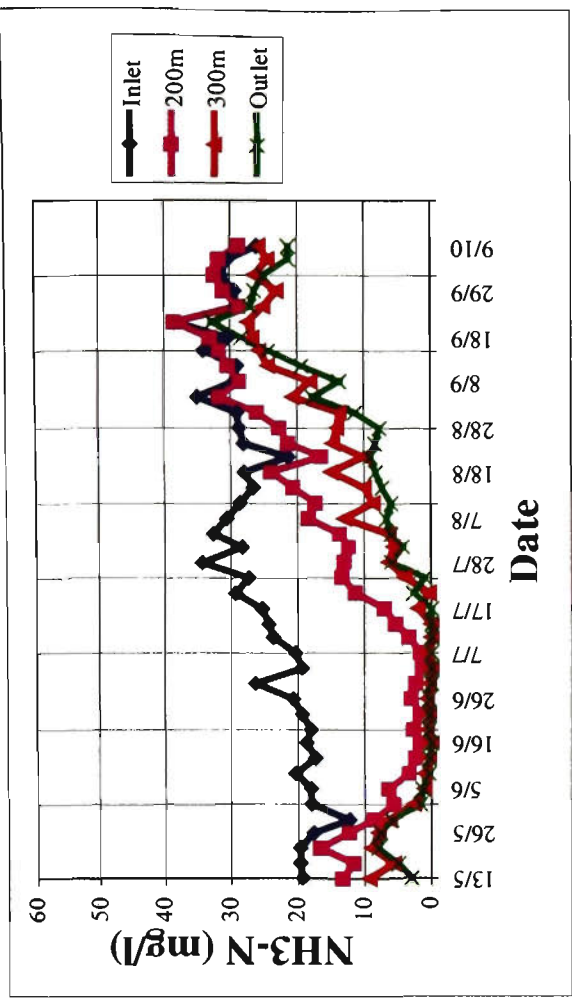
Figure A.13 NO_x-N Temporal Plots (Continued...)



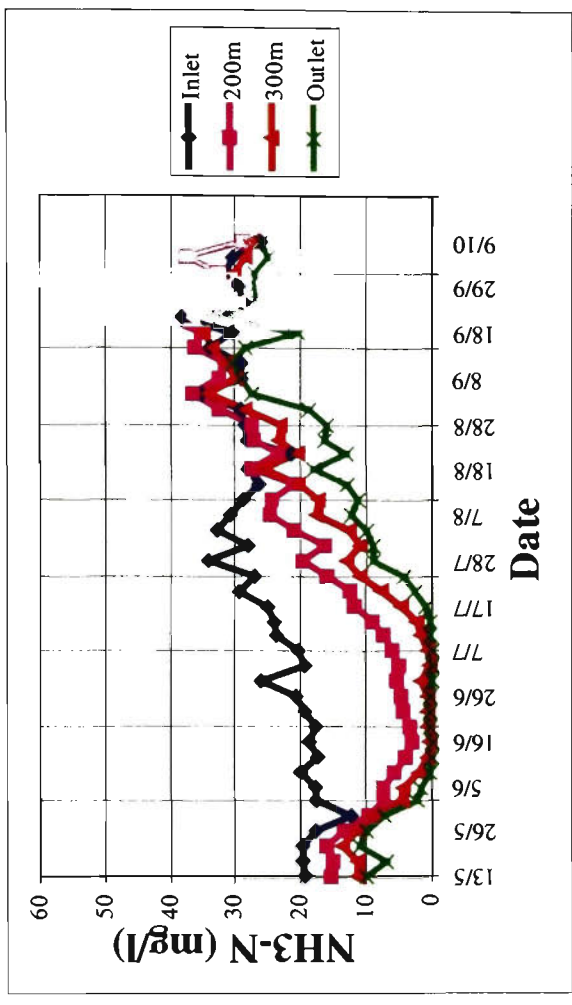
Bay 1



Bay 3

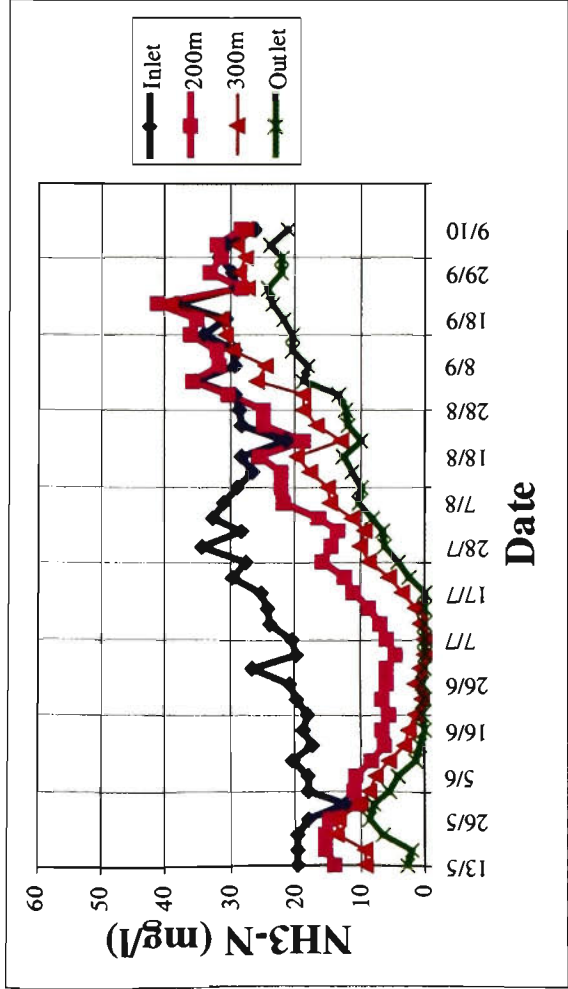


Bay 2

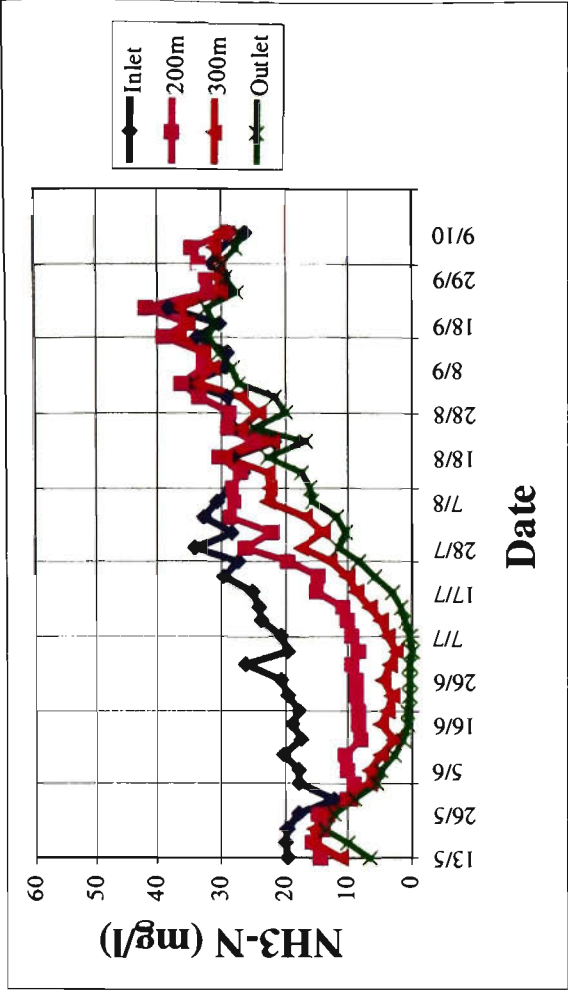


Bay 4

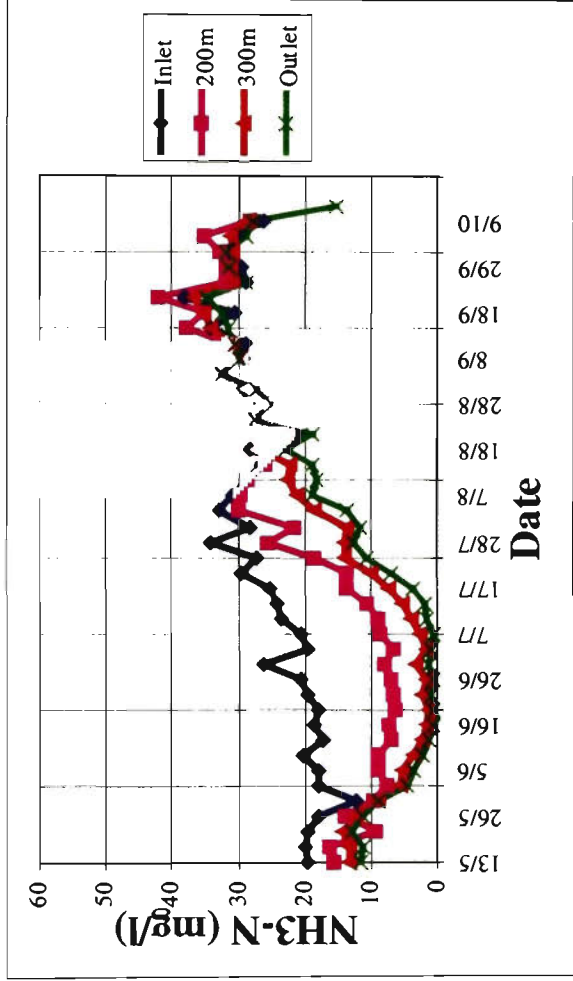
Figure A.14 $\text{NH}_3\text{-N}$ Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.14 $\text{NH}_3\text{-N}$ Temporal Plots (Continued...)

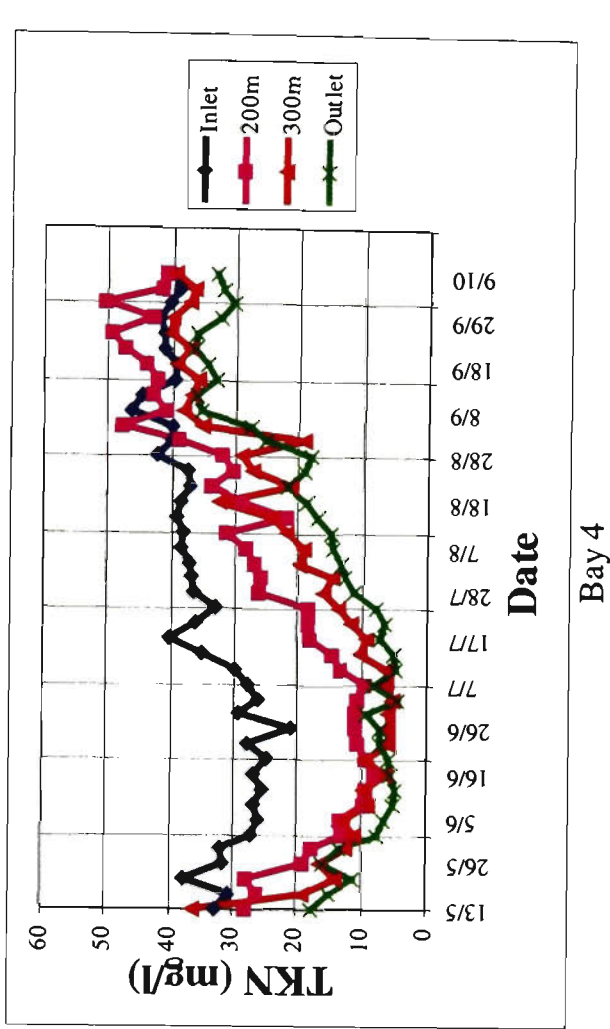
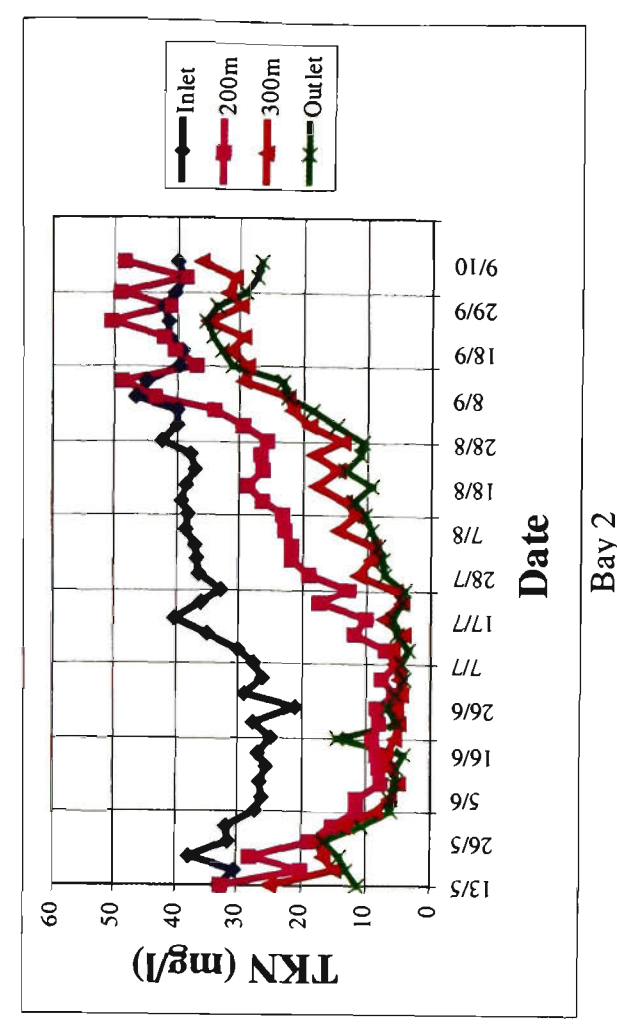
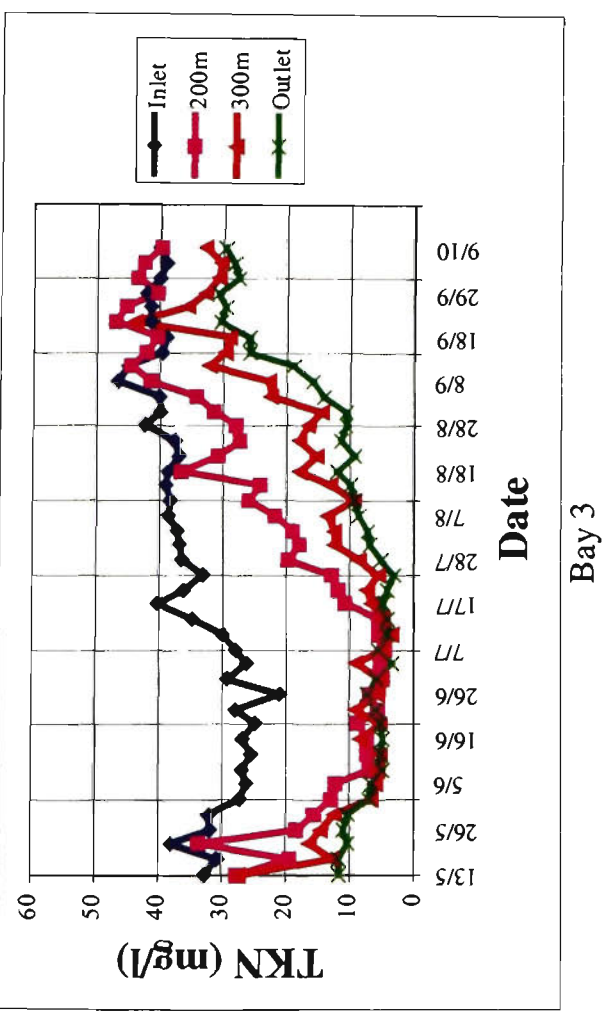
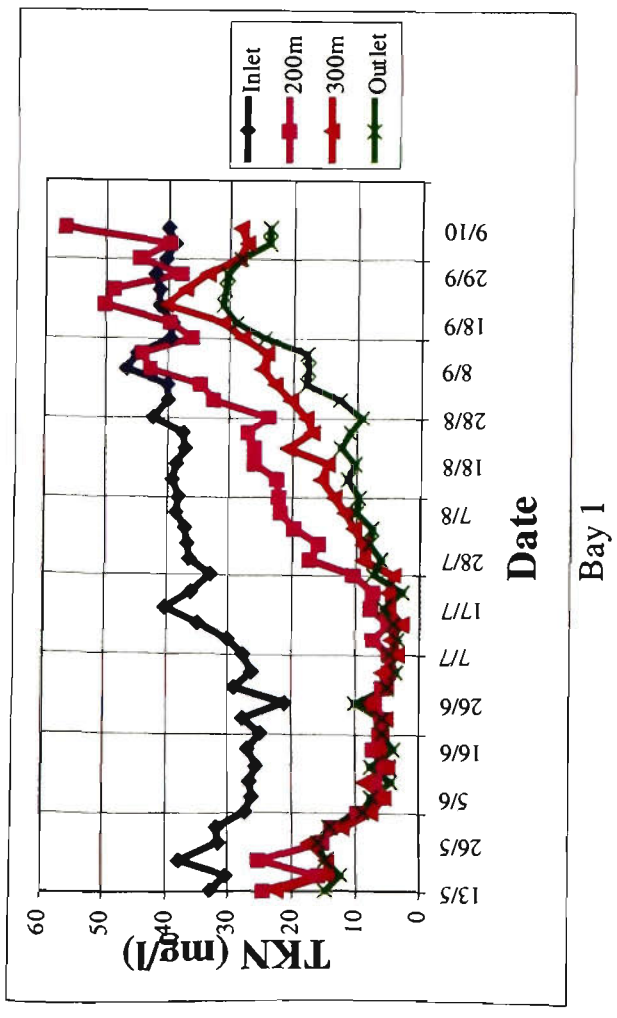


Figure A.15 TKN Temporal Plots

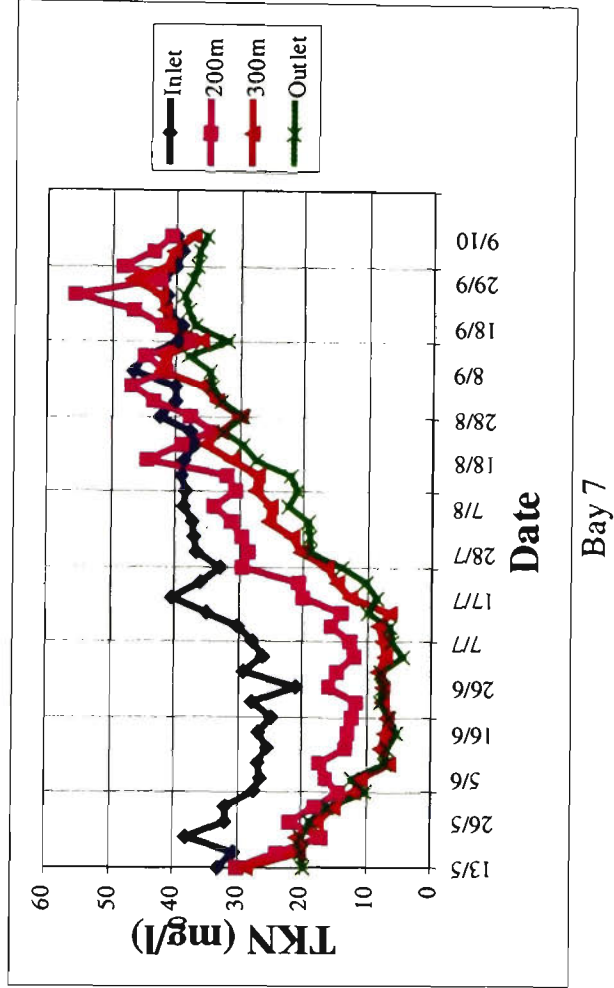
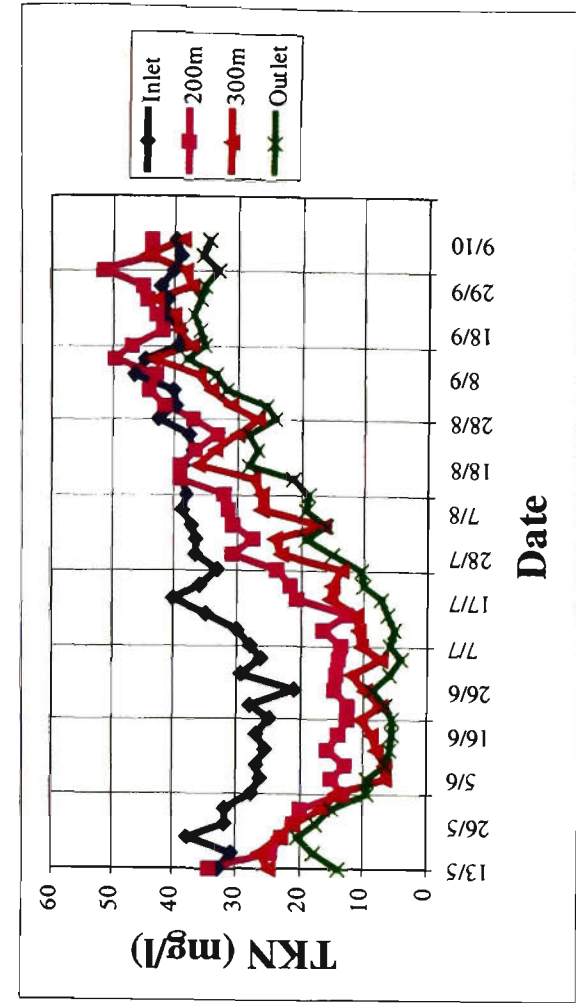
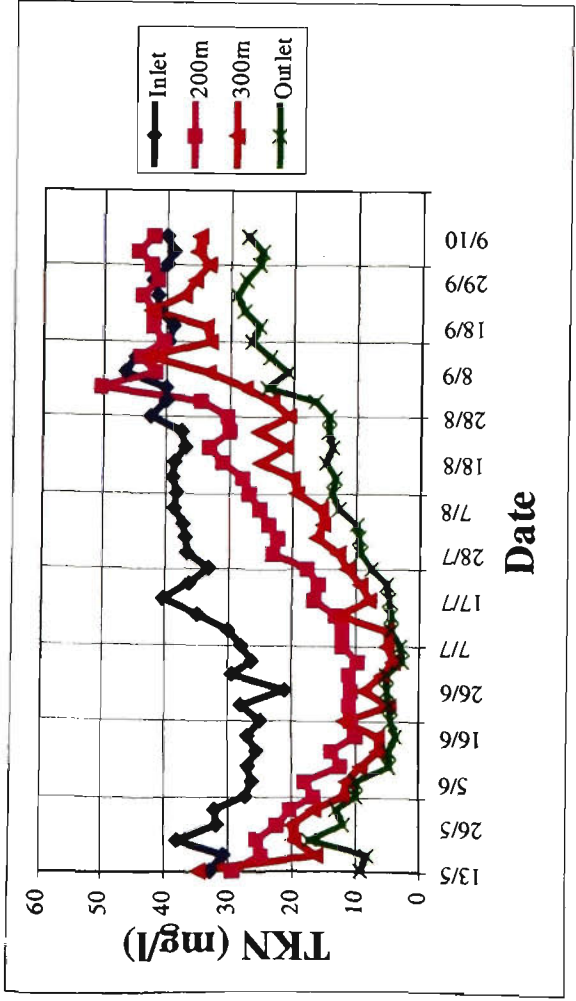
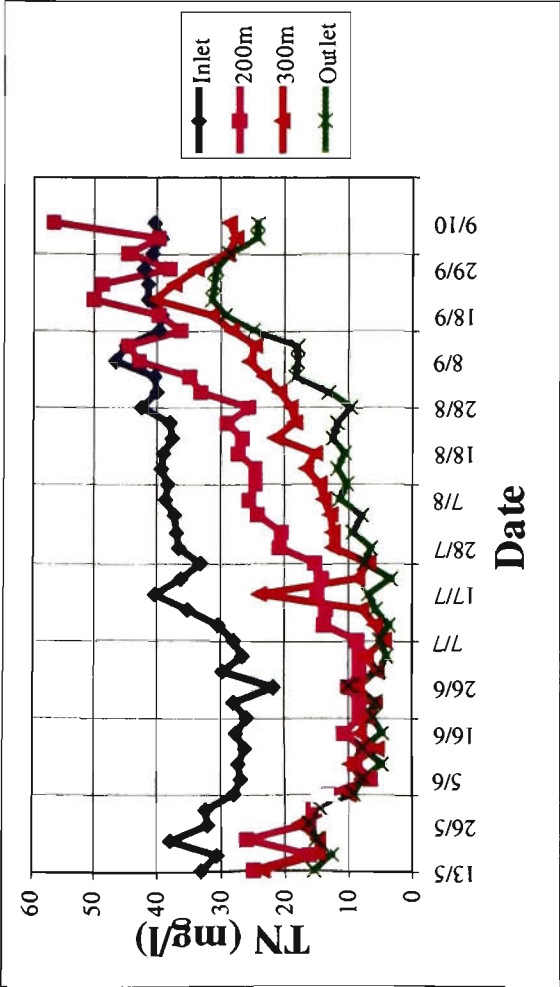
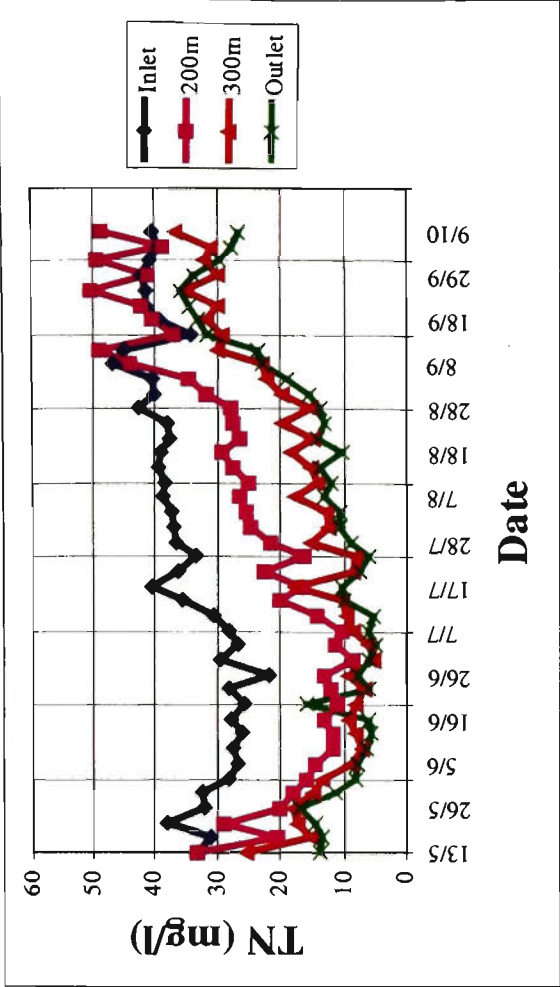


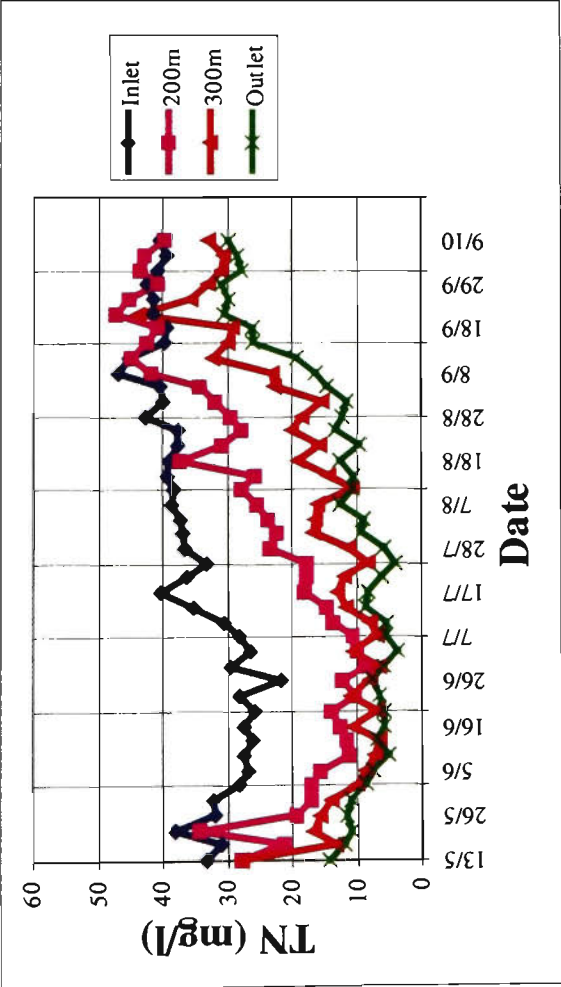
Figure A.15 TKN Temporal Plots (Continued...)



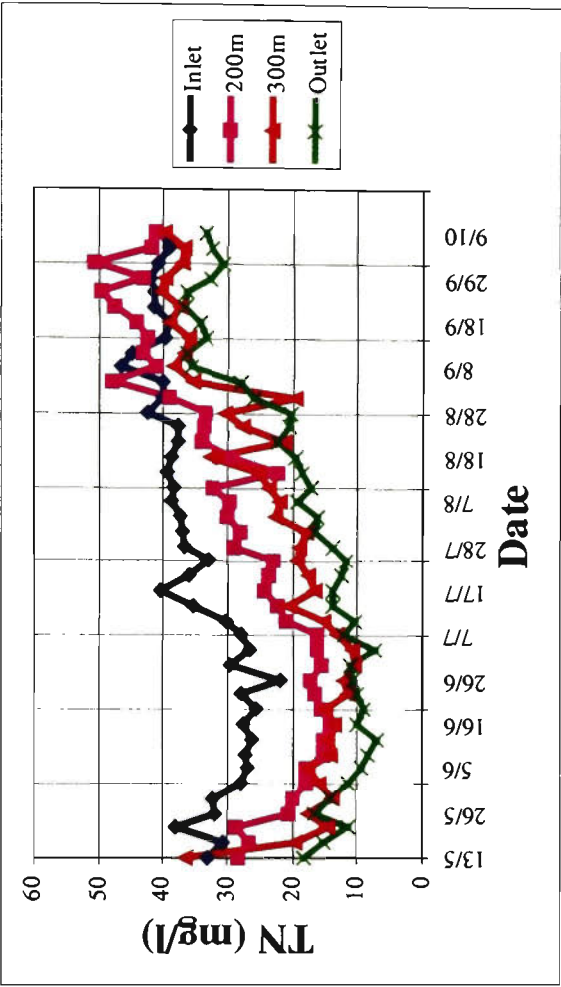
Bay 1



Bay 2

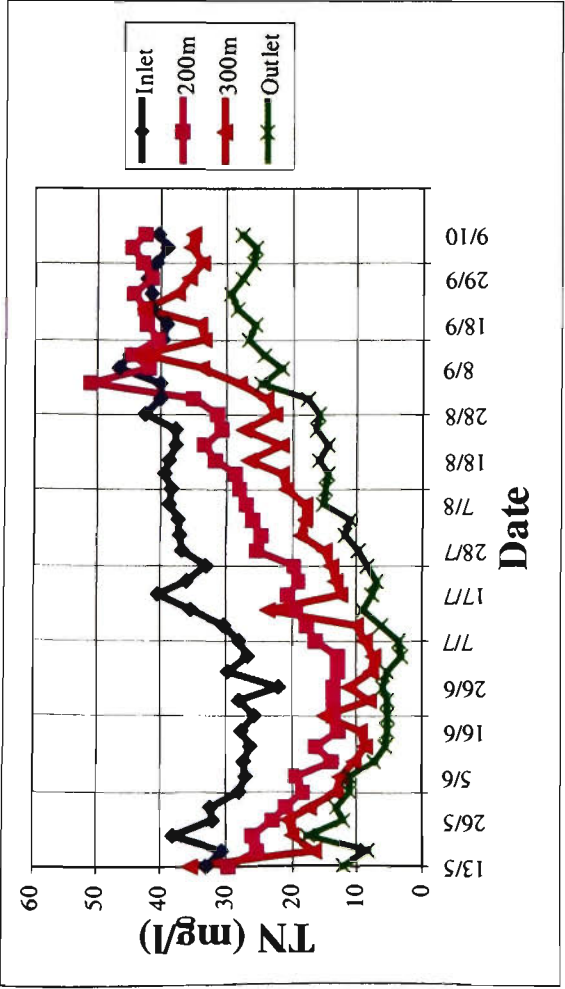


Bay 3

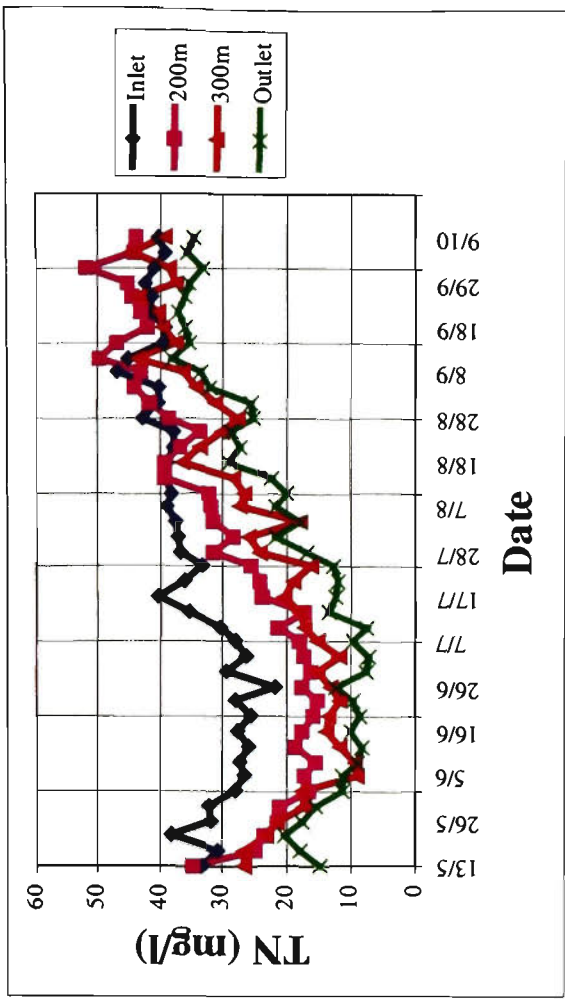


Bay 4

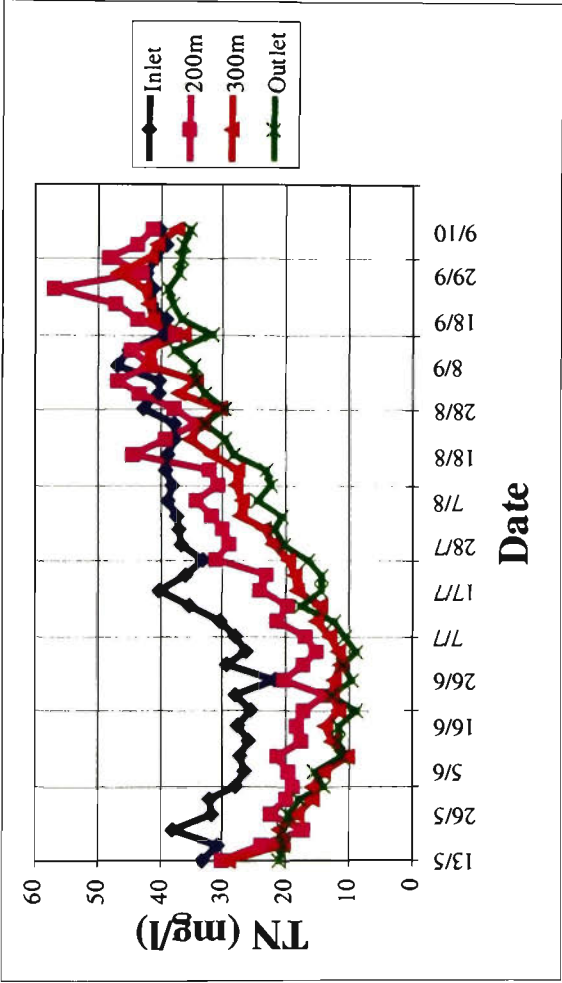
FigureA.16 TN Temporal Plots



Bay 5

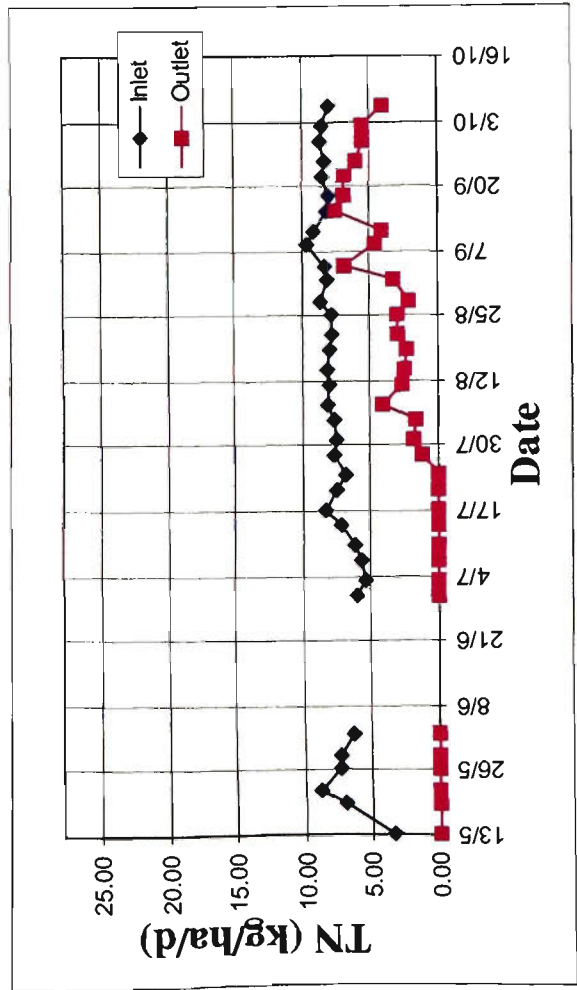


Bay 6

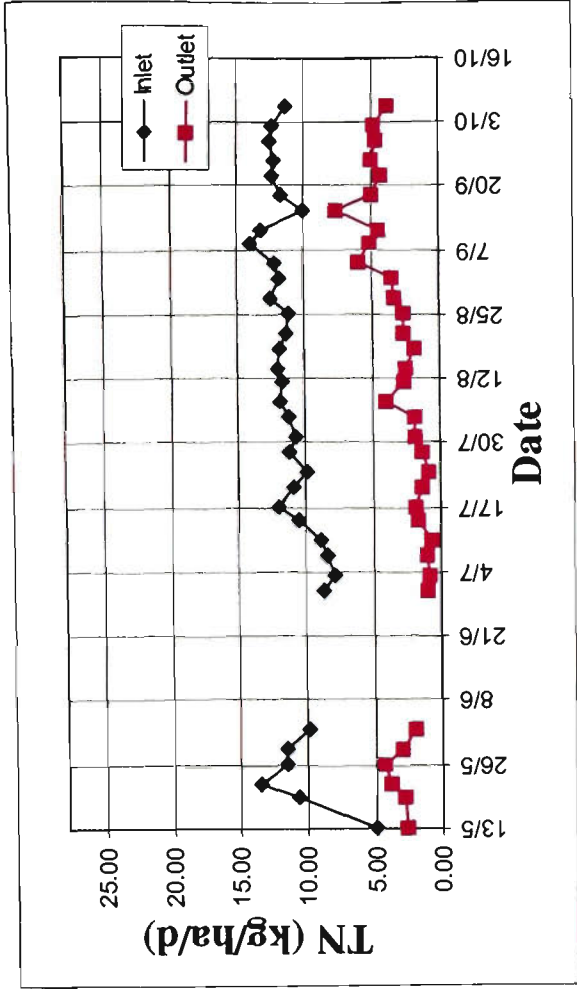


Bay 7

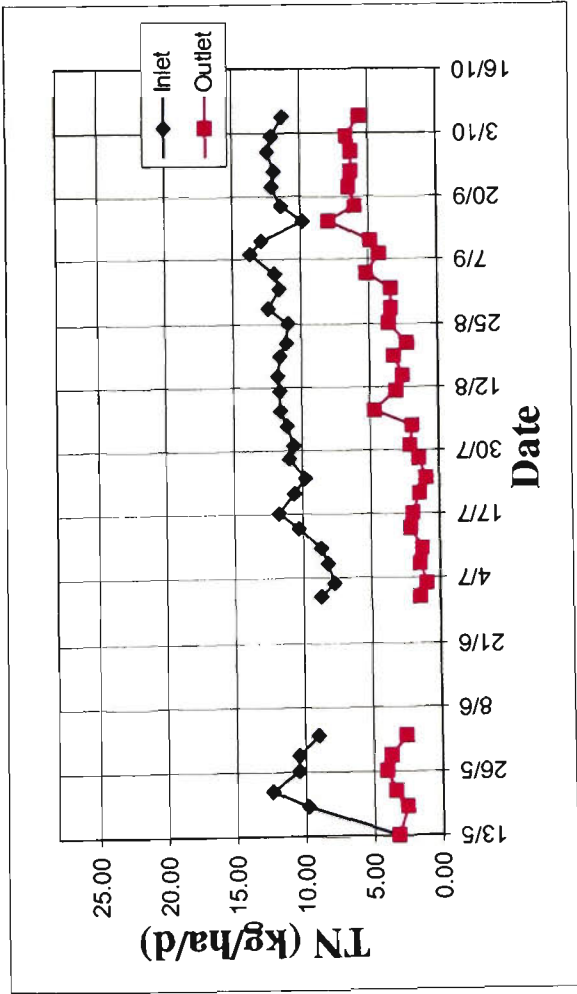
FigureA.16 TN Temporal Plots (Continued...)



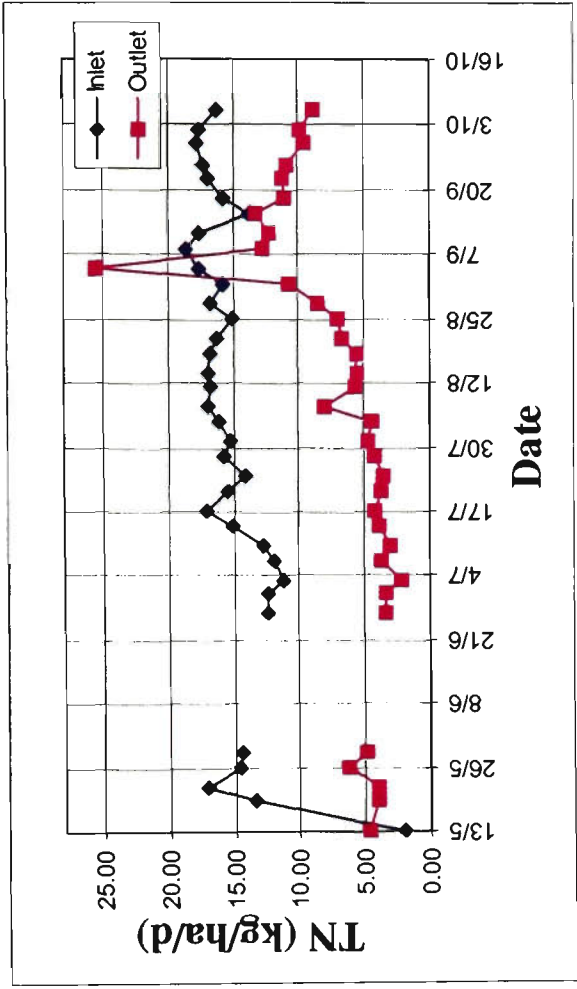
Bay 1



Bay 2

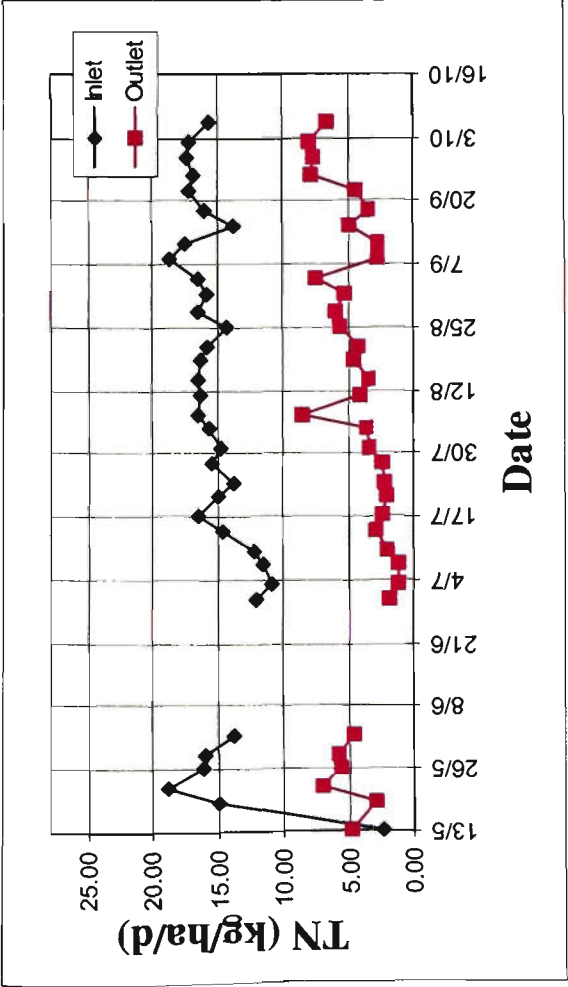


Bay 3

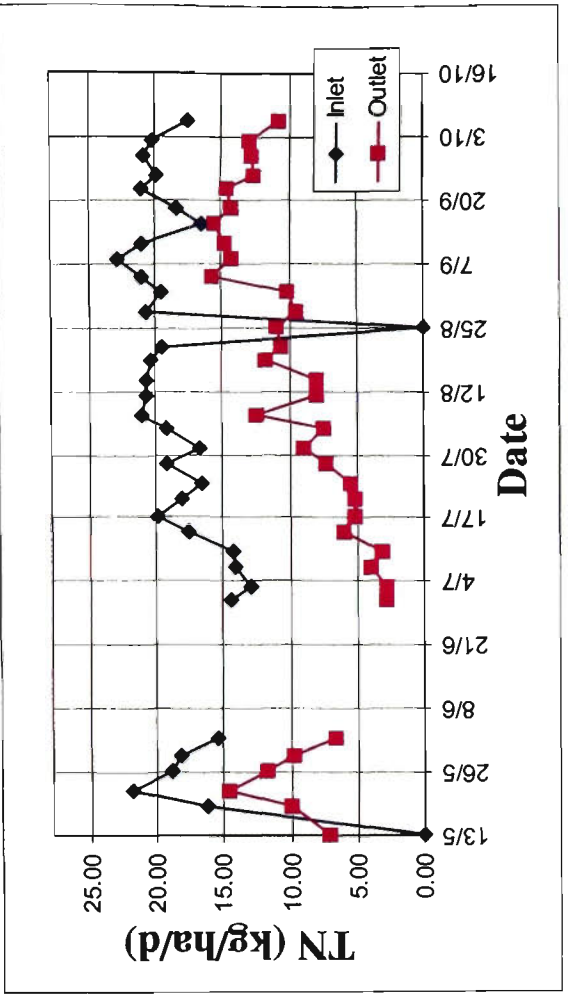


Bay 4

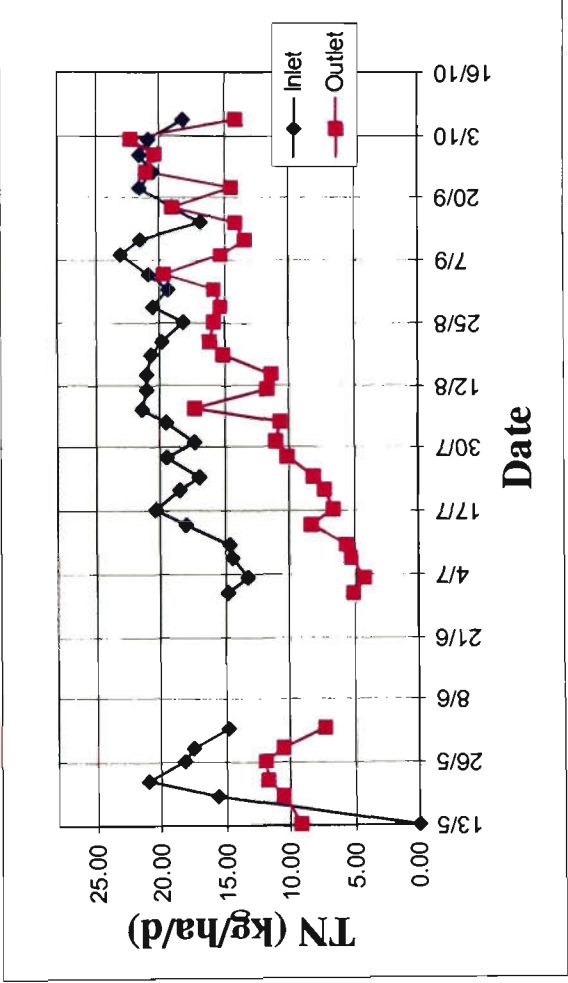
Figure A.17 TN Mass Balance Plots



Bay 5

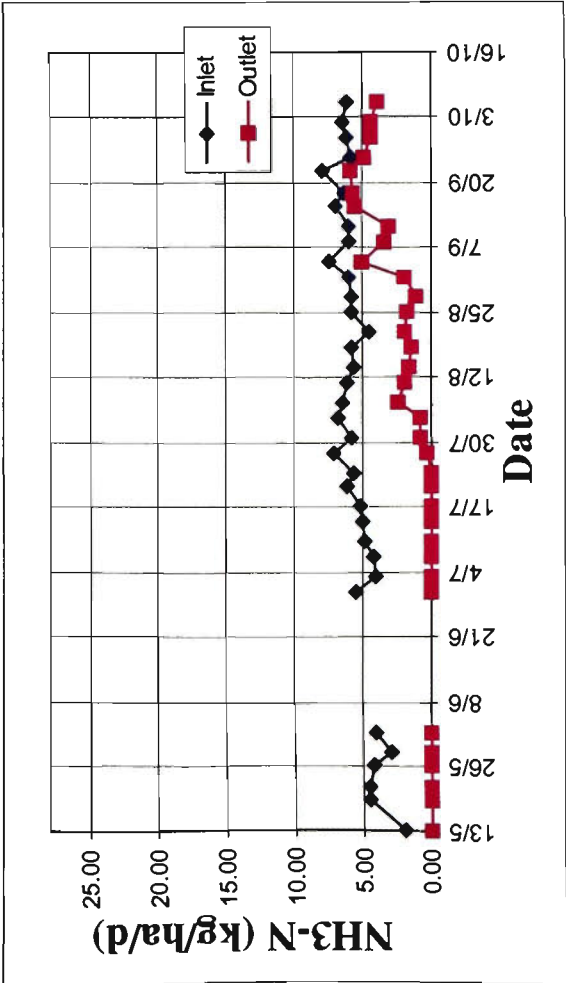


Bay 6

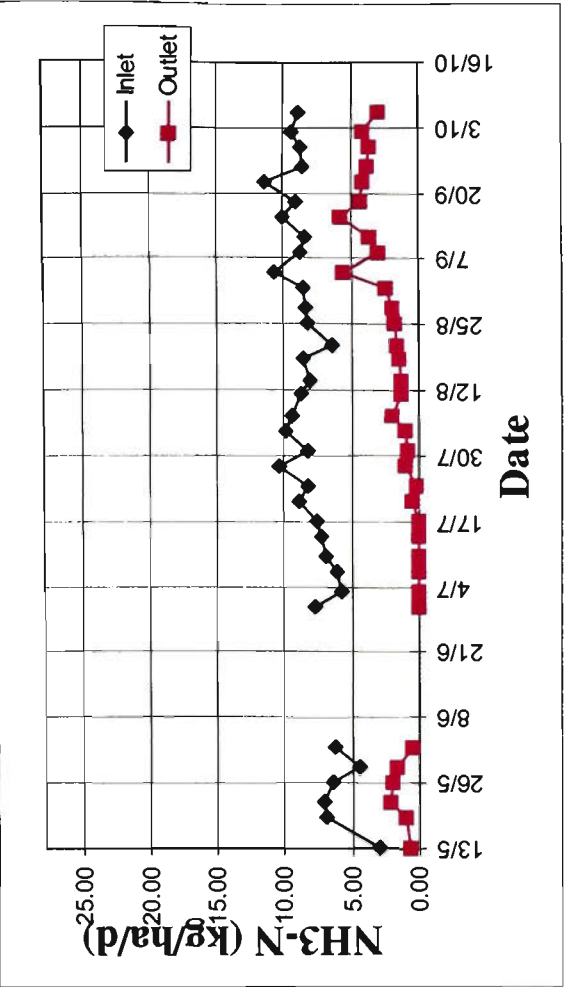


Bay 7

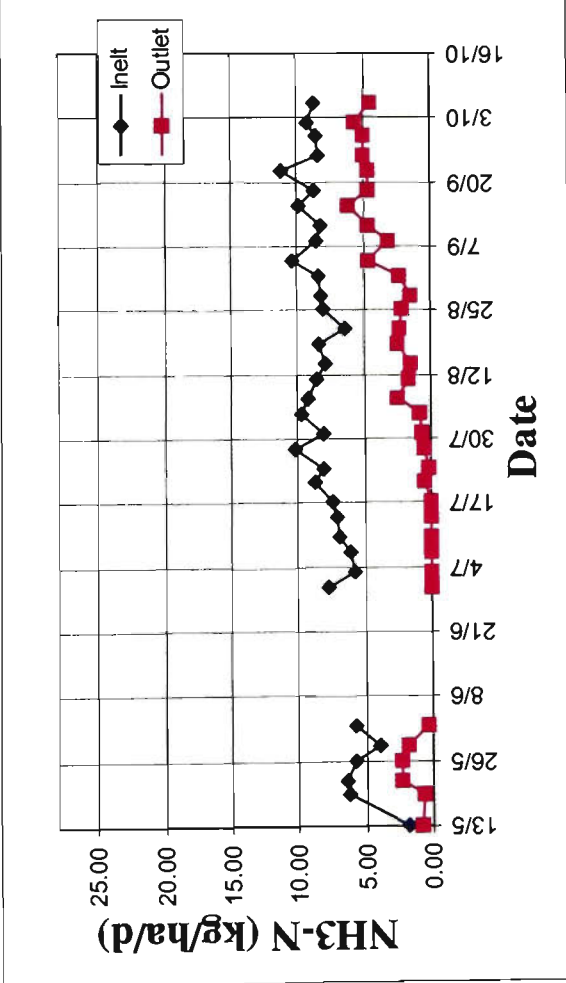
Figure A.17 TN Mass Balance Plots (Continued...)



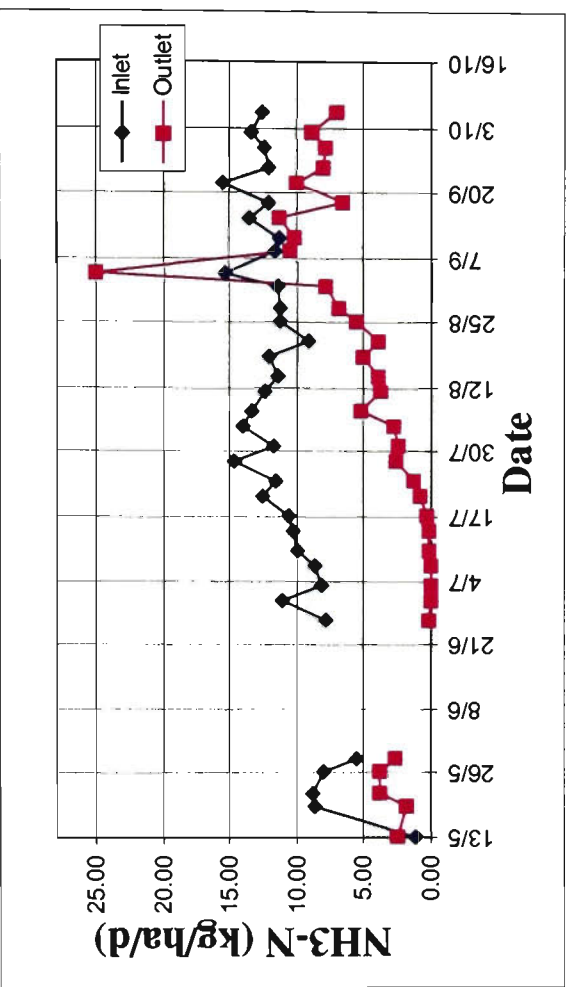
Bay 1



Bay 2

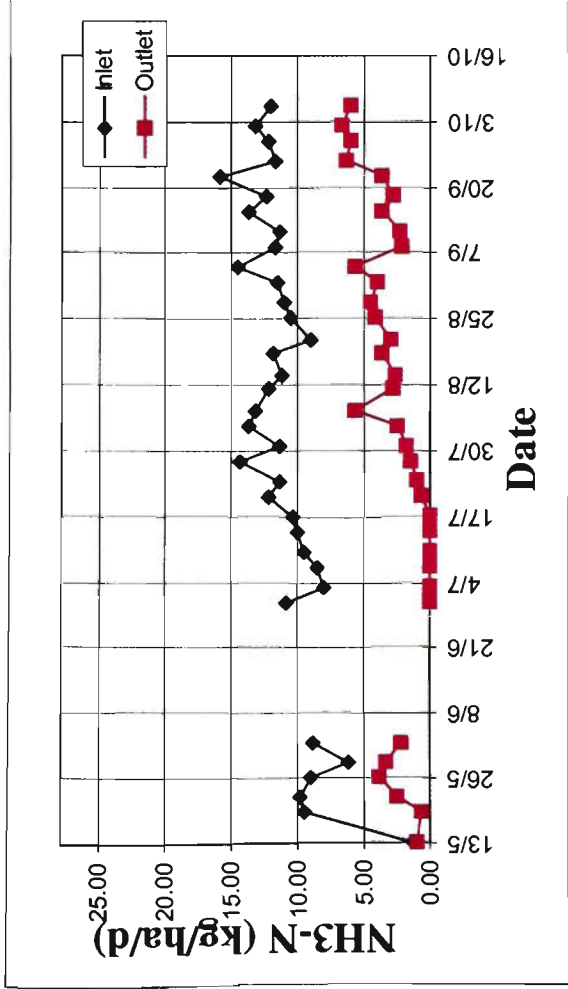


Bay 3

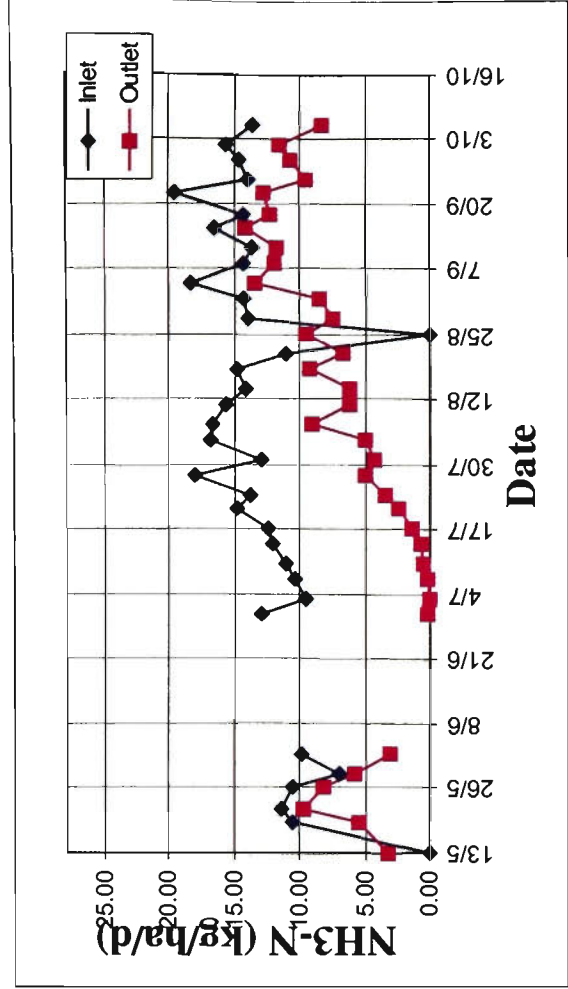


Bay 4

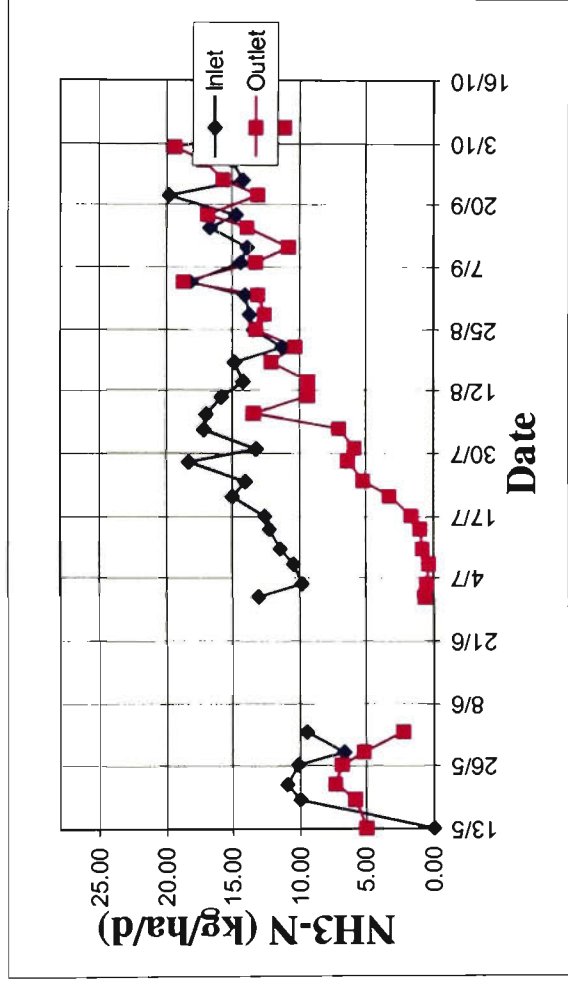
Figure A.18 $\text{NH}_3\text{-N}$ Mass Balance Plots



Bay 5



Bay 6



Bay 7

Figure A.18 NH₃-N Mass Balance Plots (Continued)

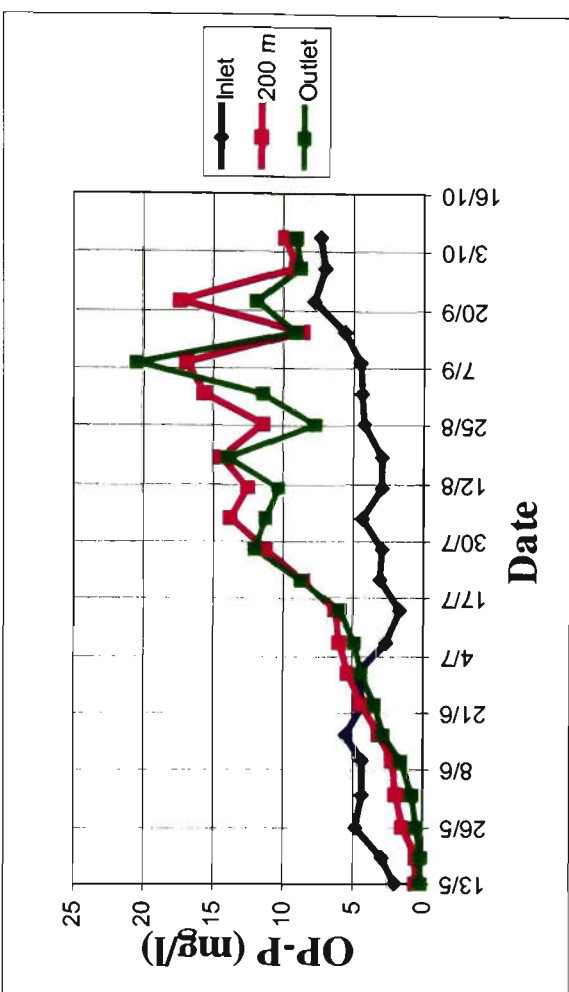
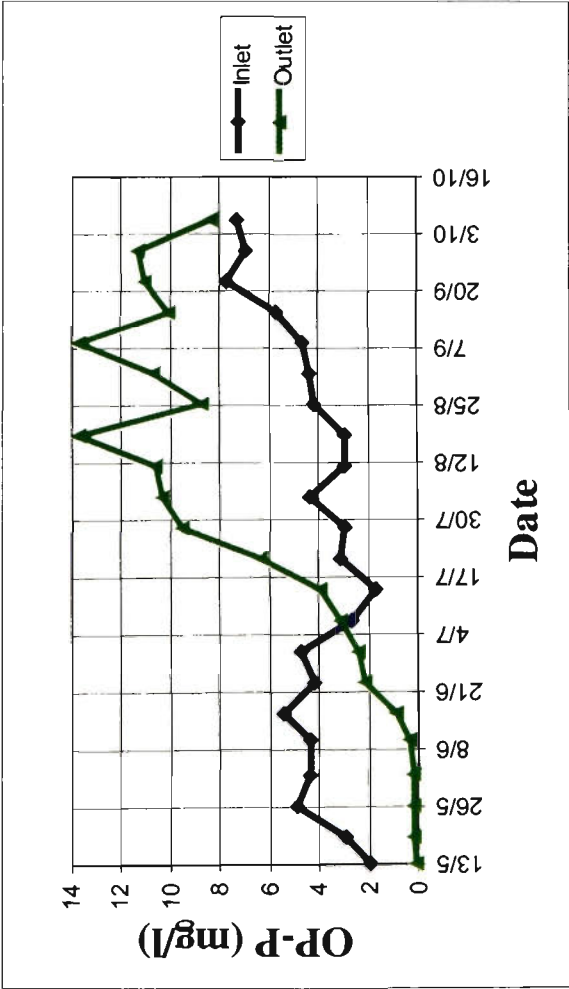
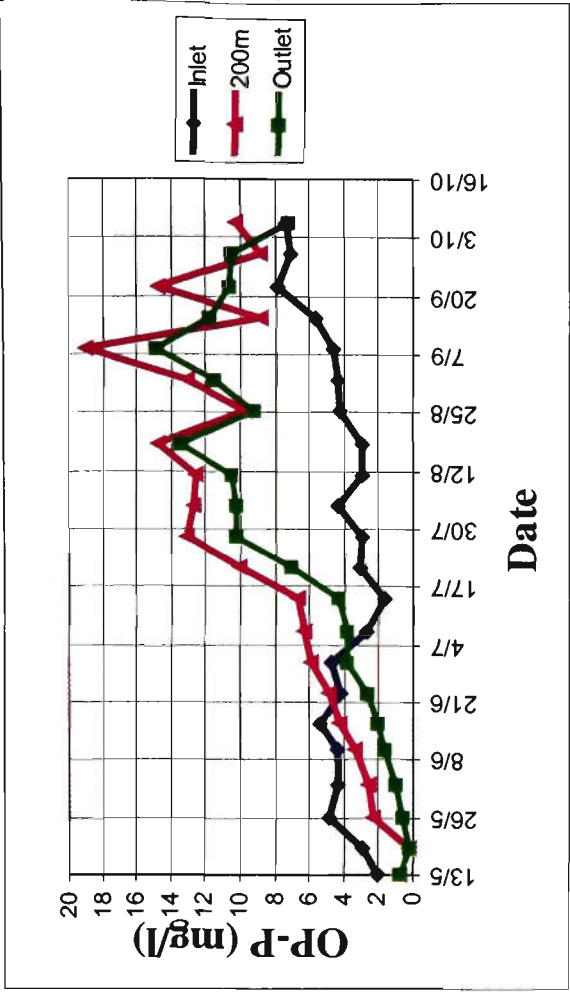
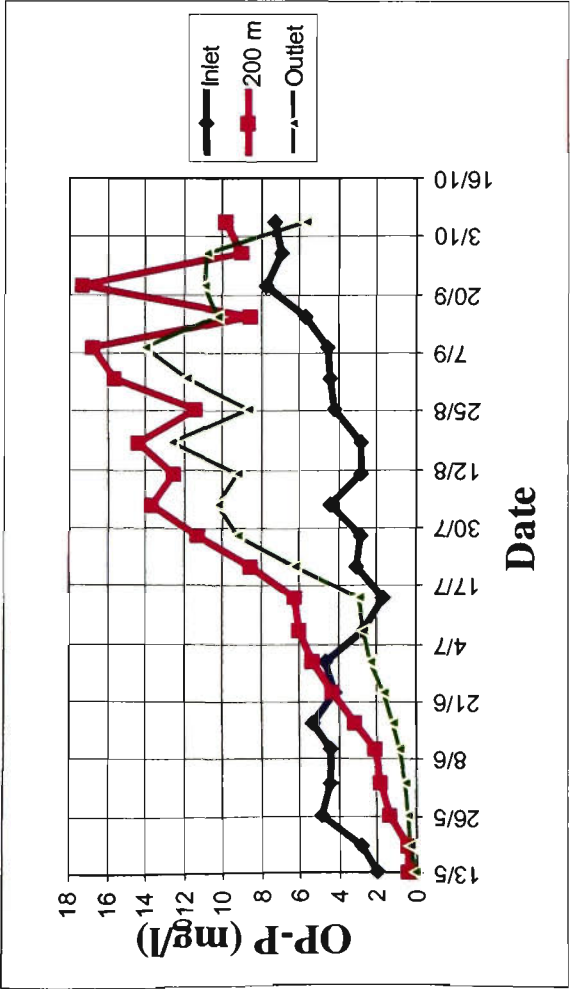
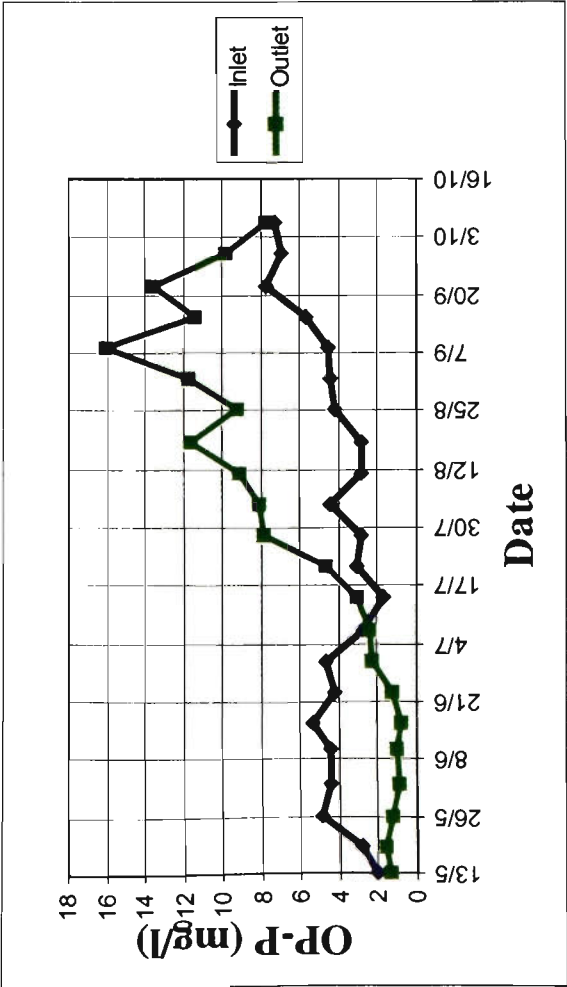
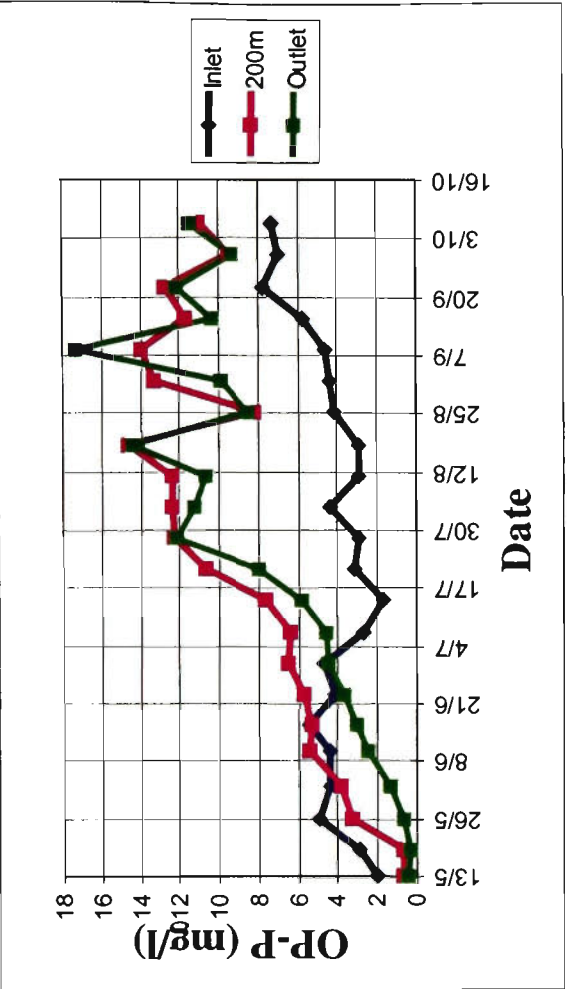


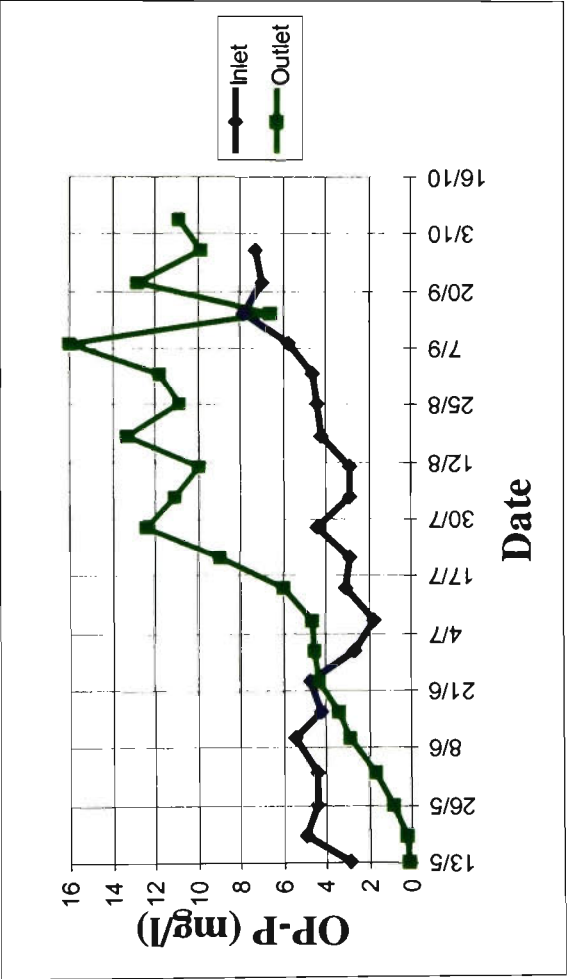
Figure A.19 OP-P Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.19 OP-P Temporal Plots (Continued...)

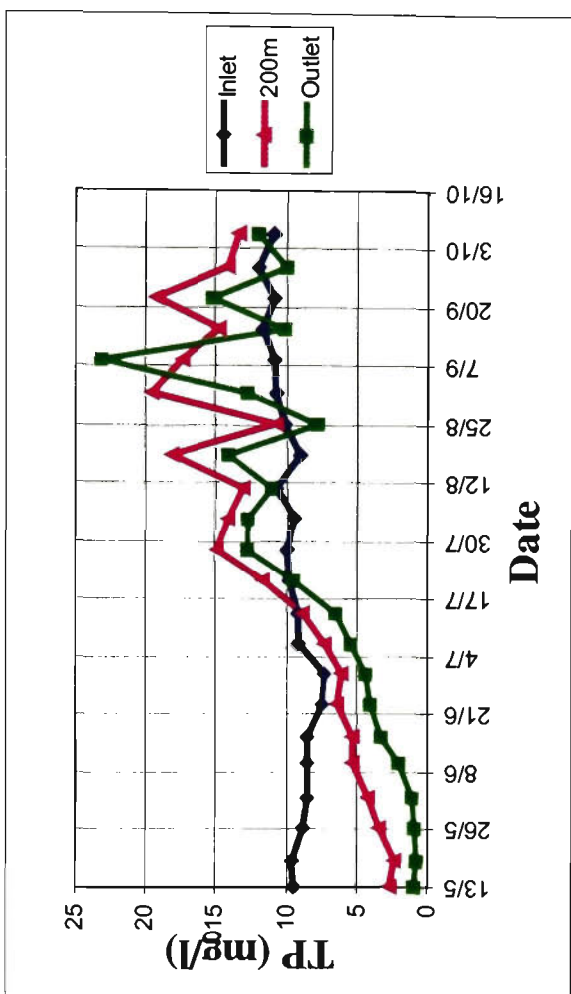
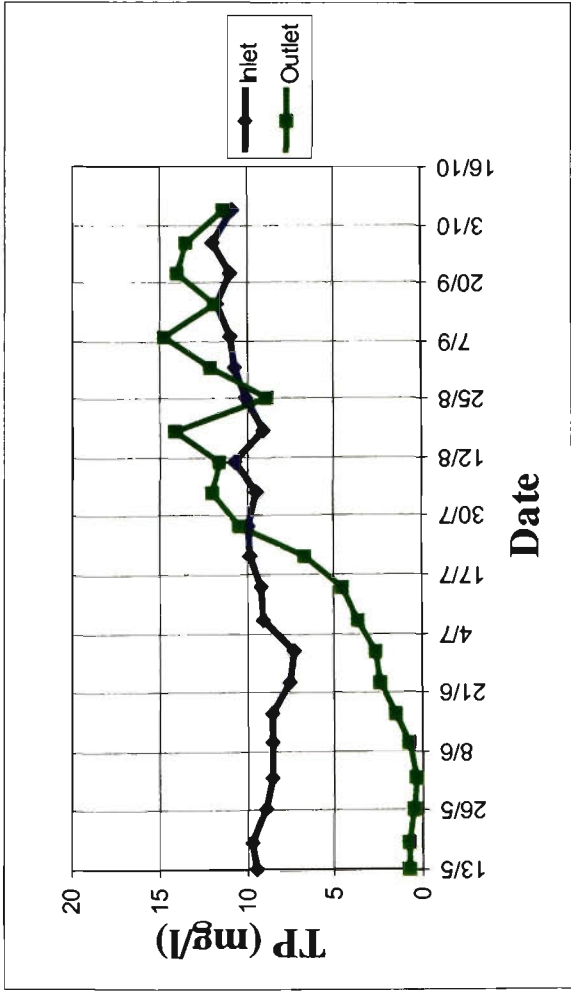
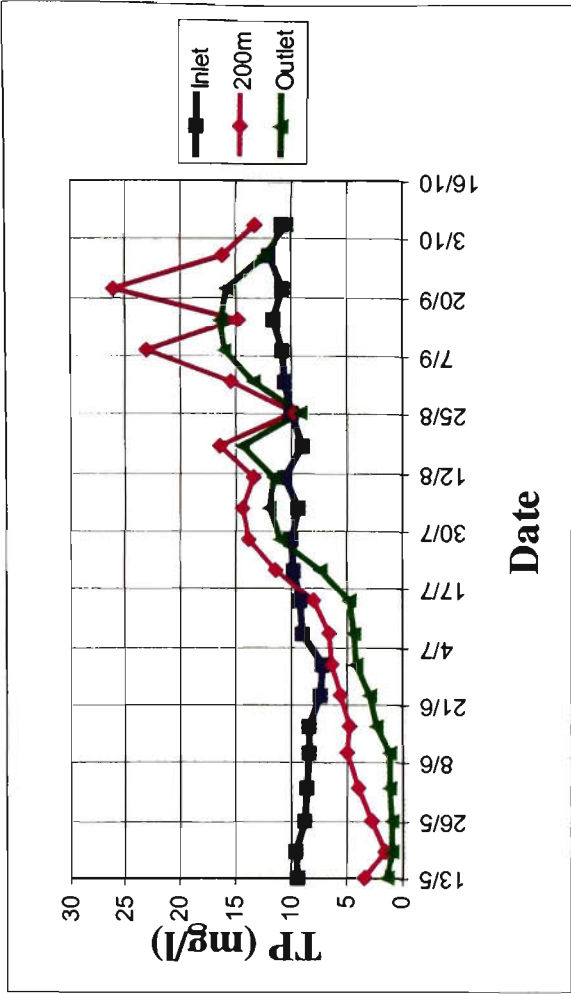
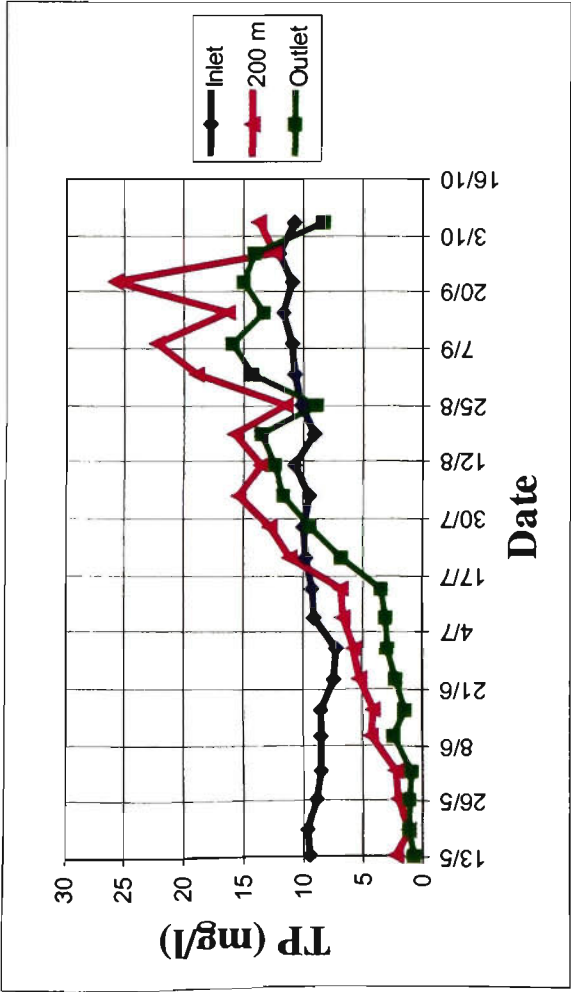
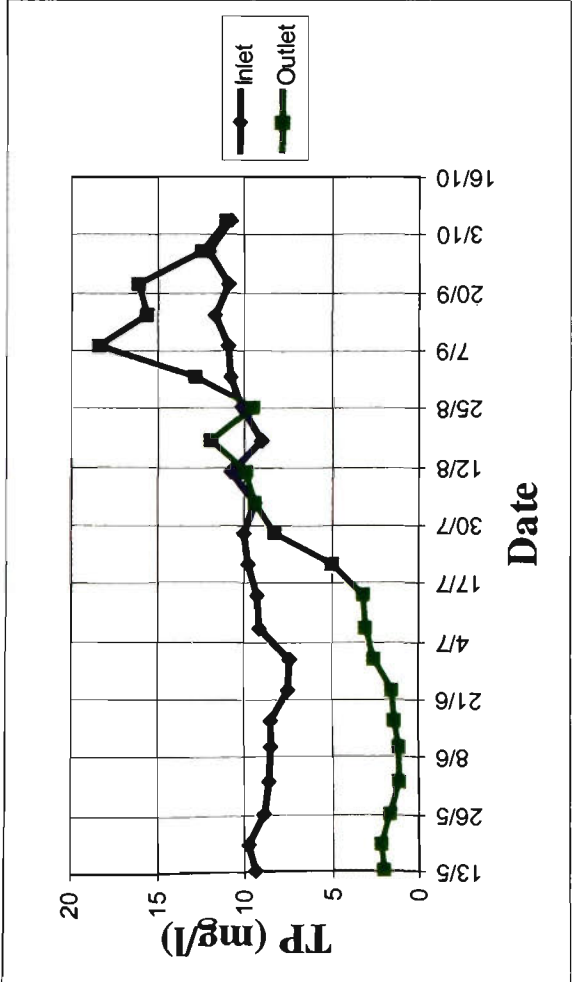
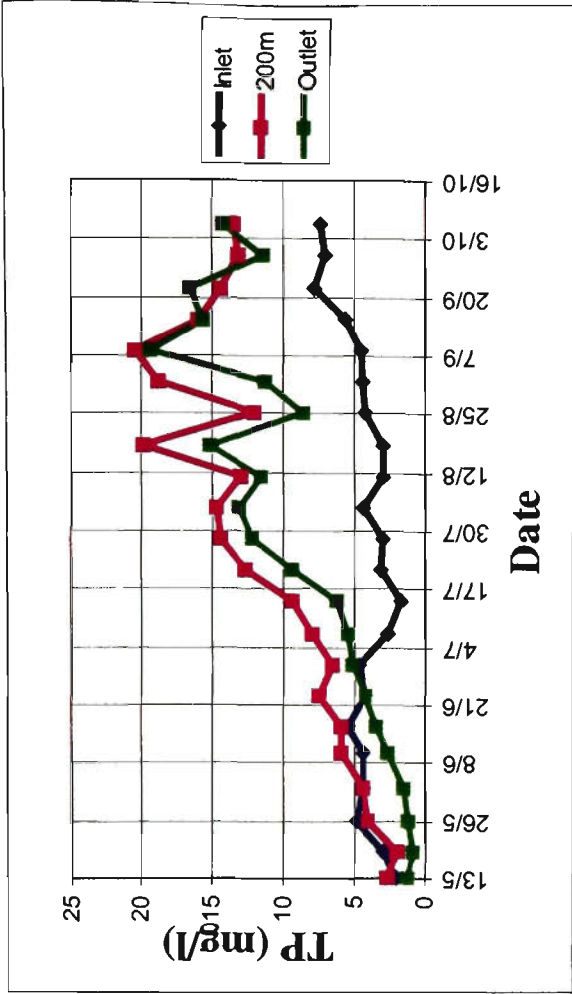


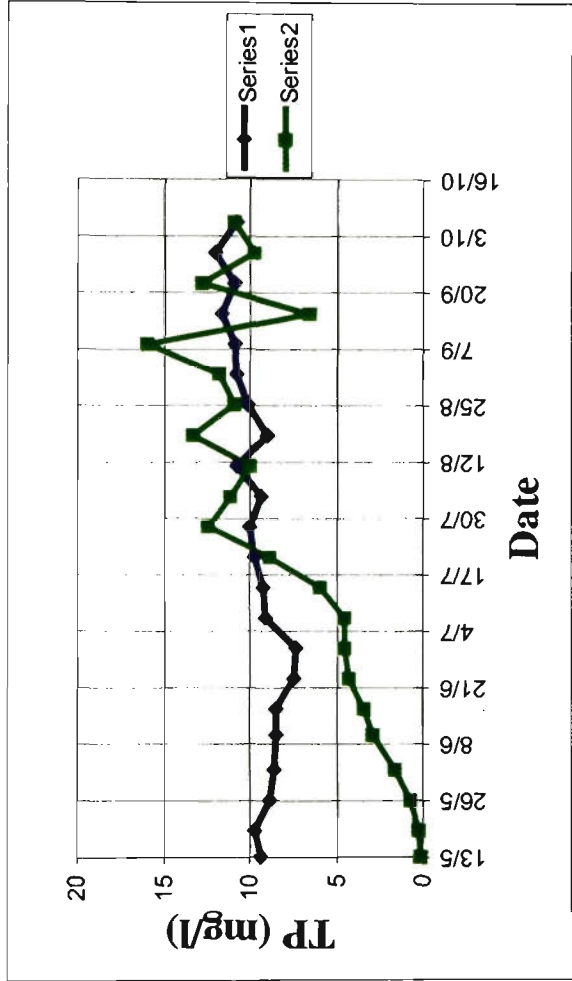
Figure A.20 TP Temporal Plots



Bay 5



Bay 6



Bay 7

Figure A.20 TP Temporal Plots (Continued...)

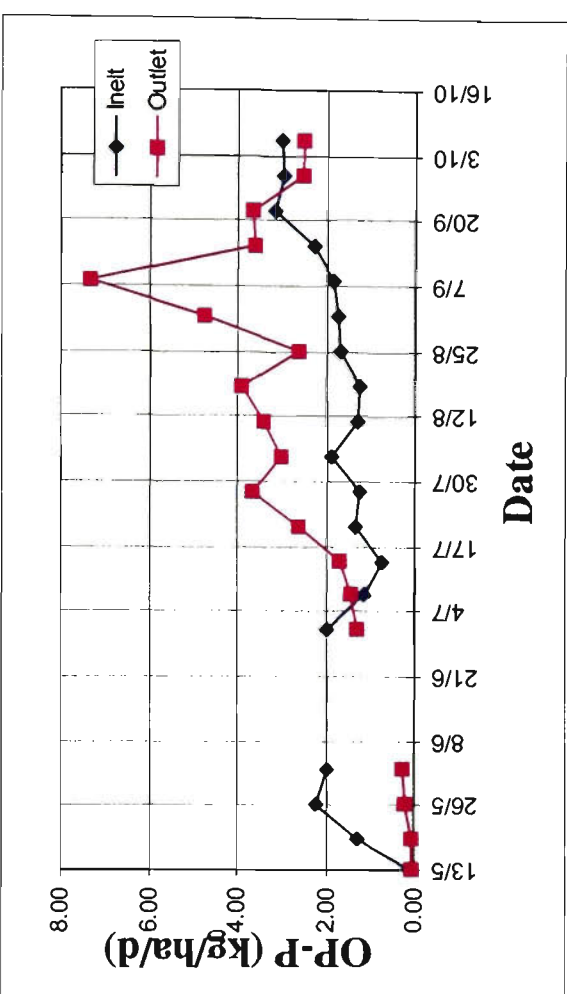
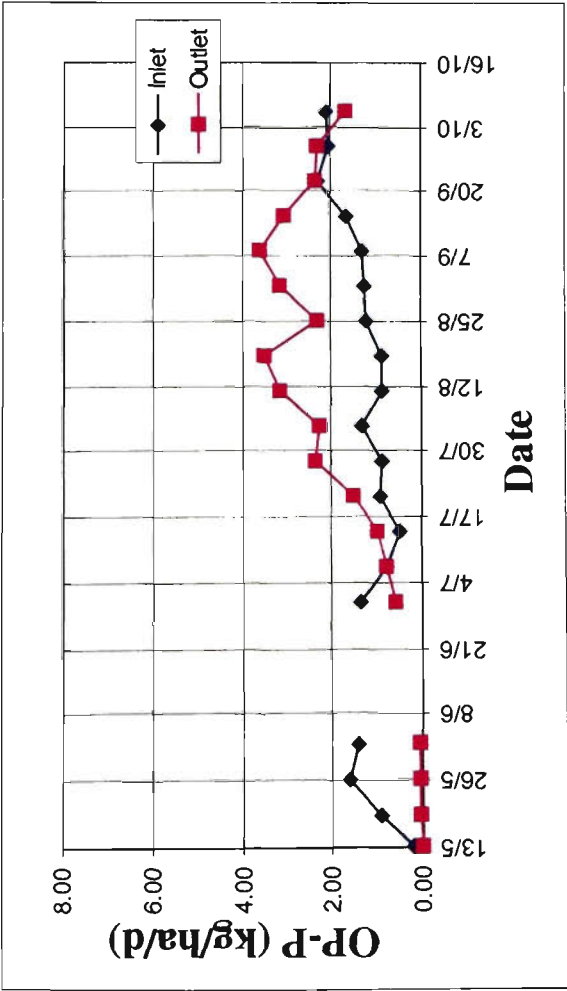
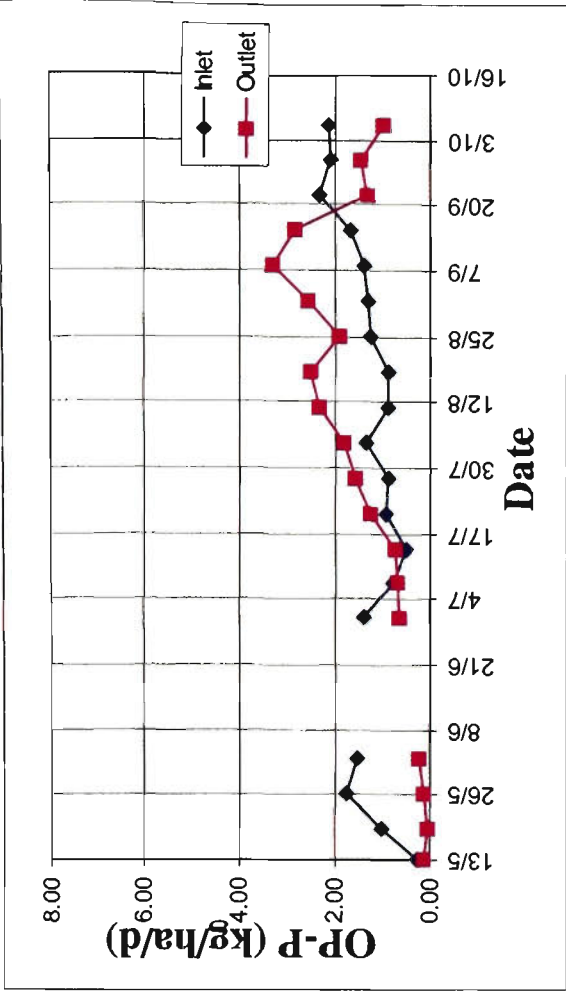
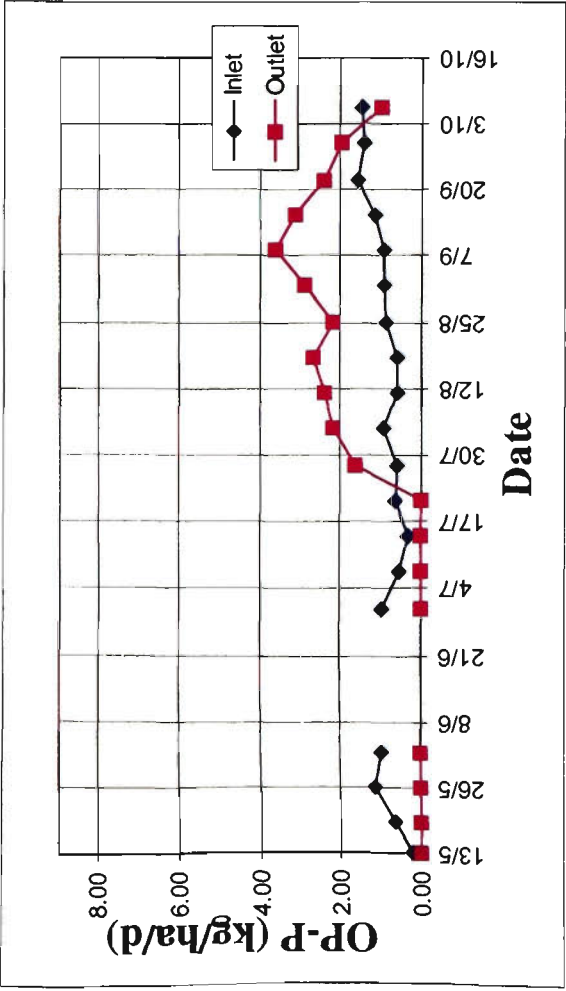


Figure A.21 OP-P Mass Balance Plots

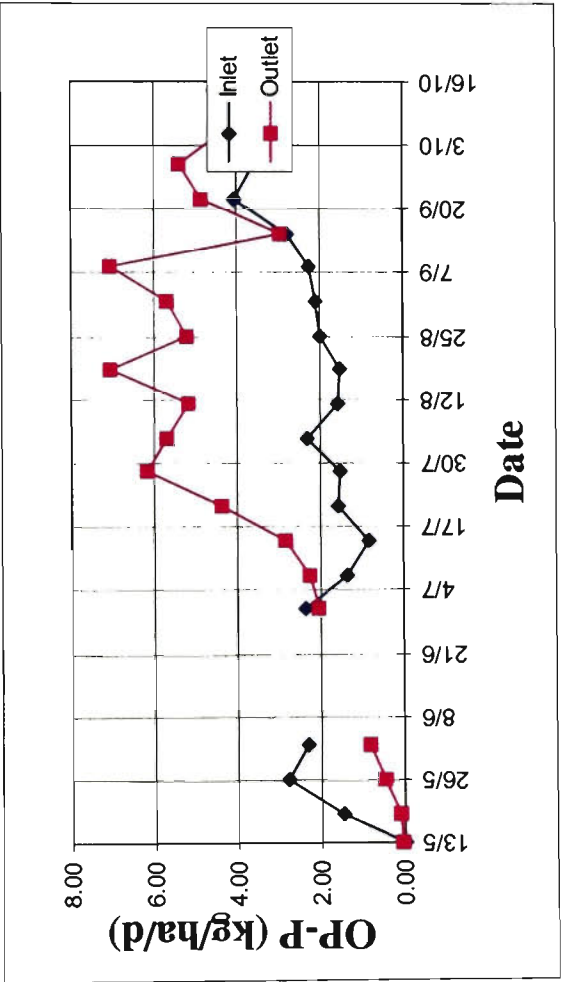
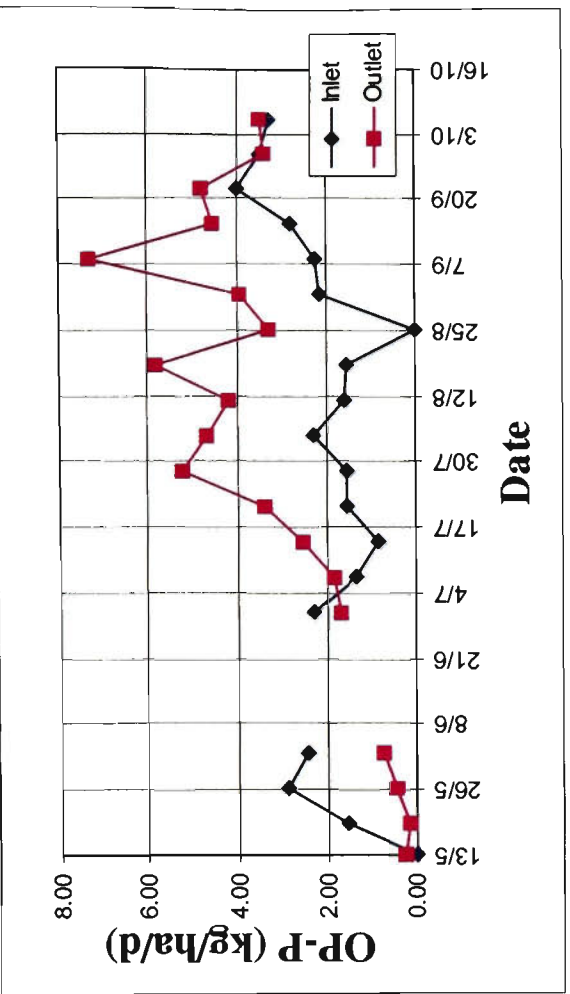
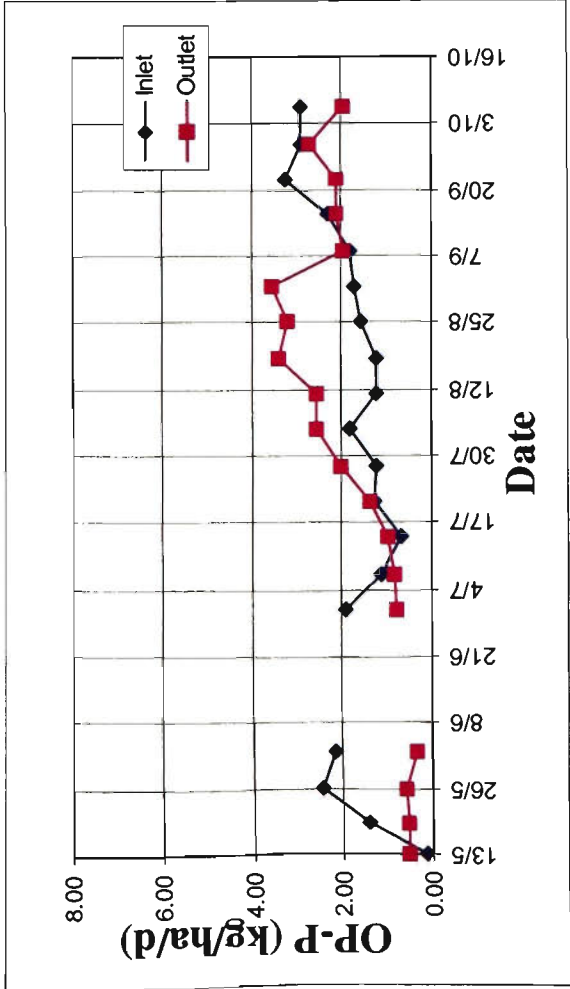


Figure A.21 OP-P Mass Balance Plots (Continued...)

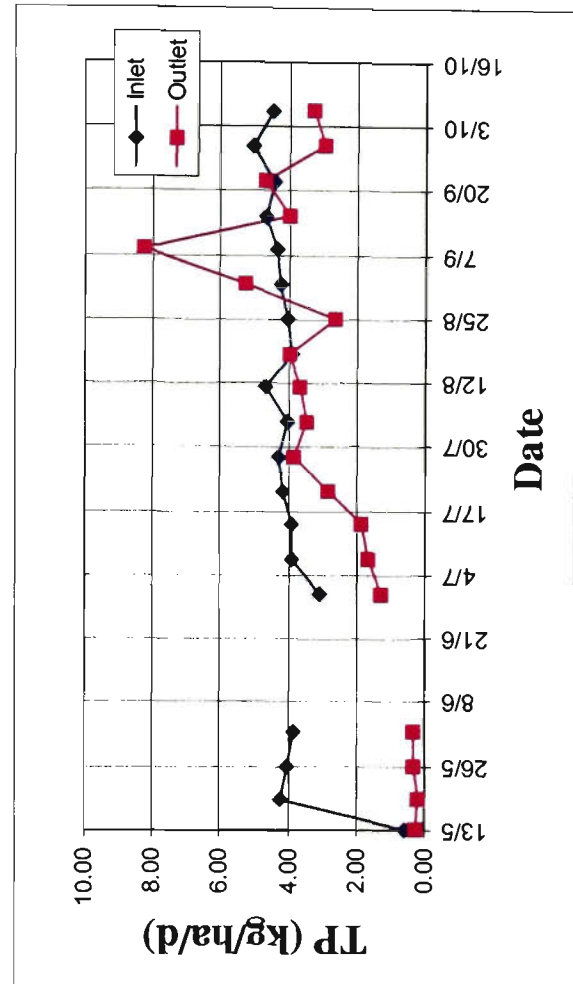
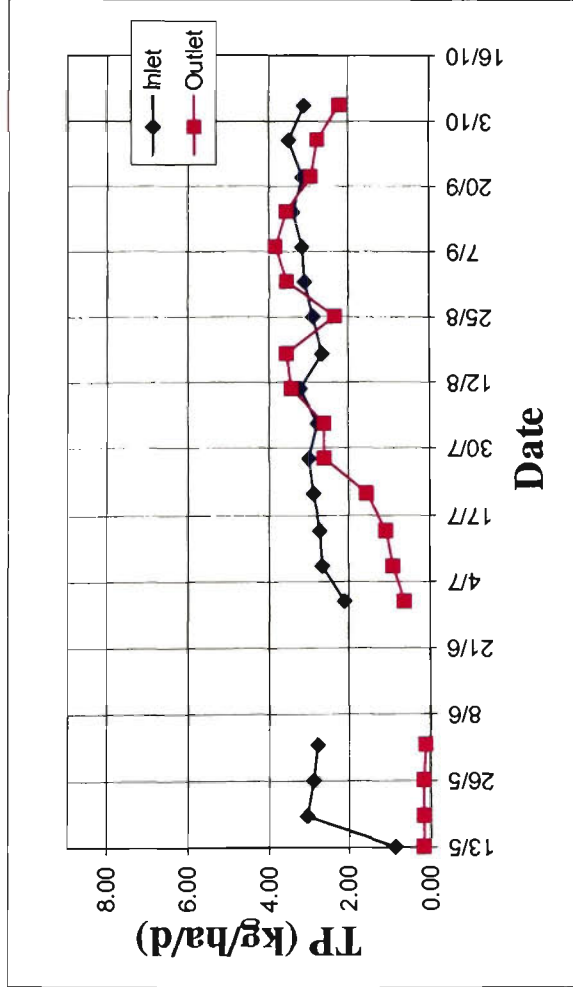
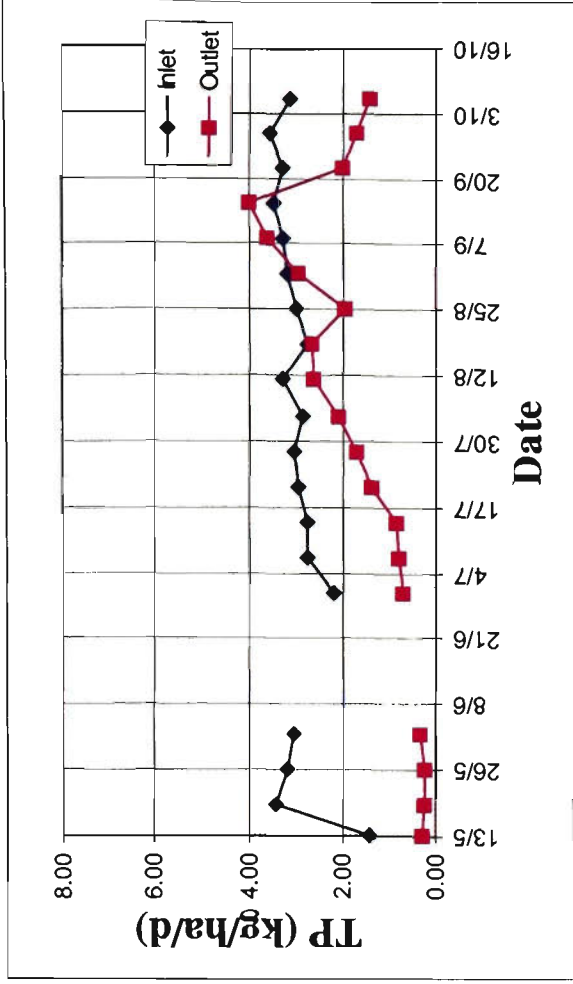
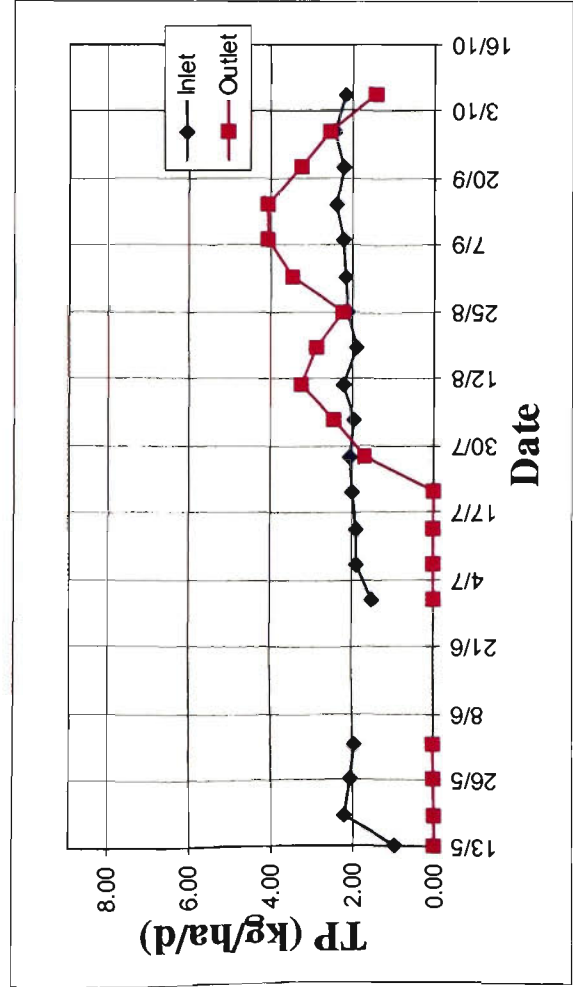


Figure A.22 TP Mass Balance Plots

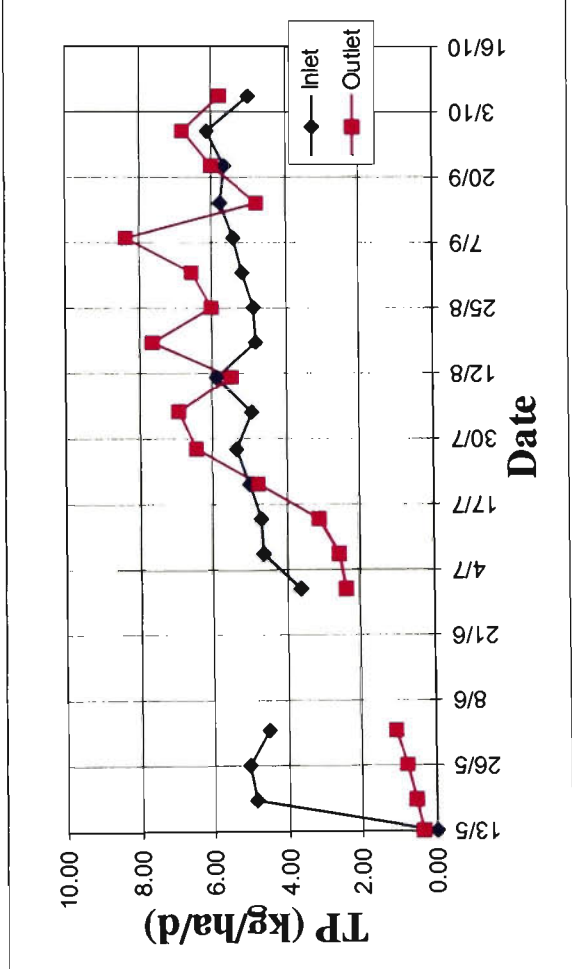
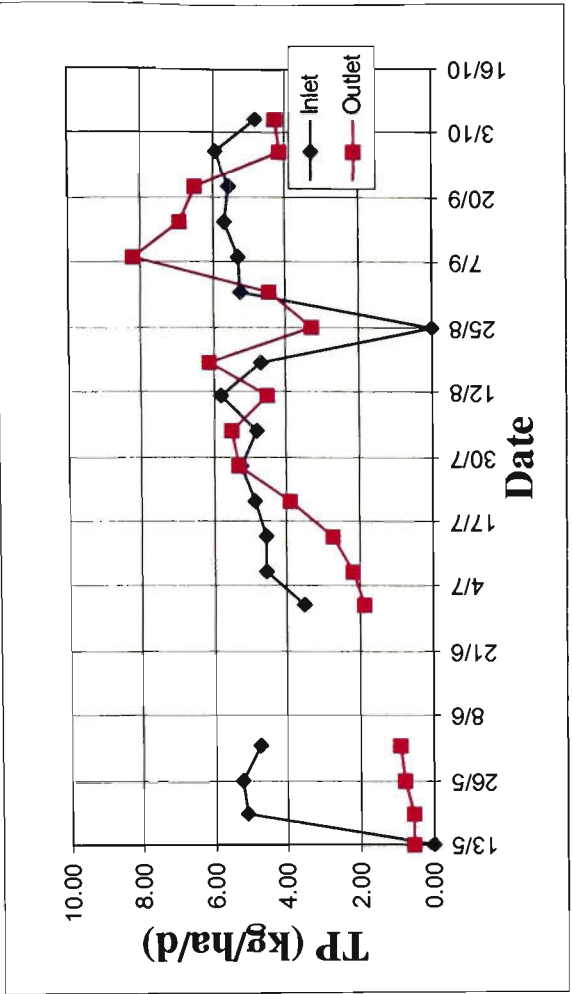
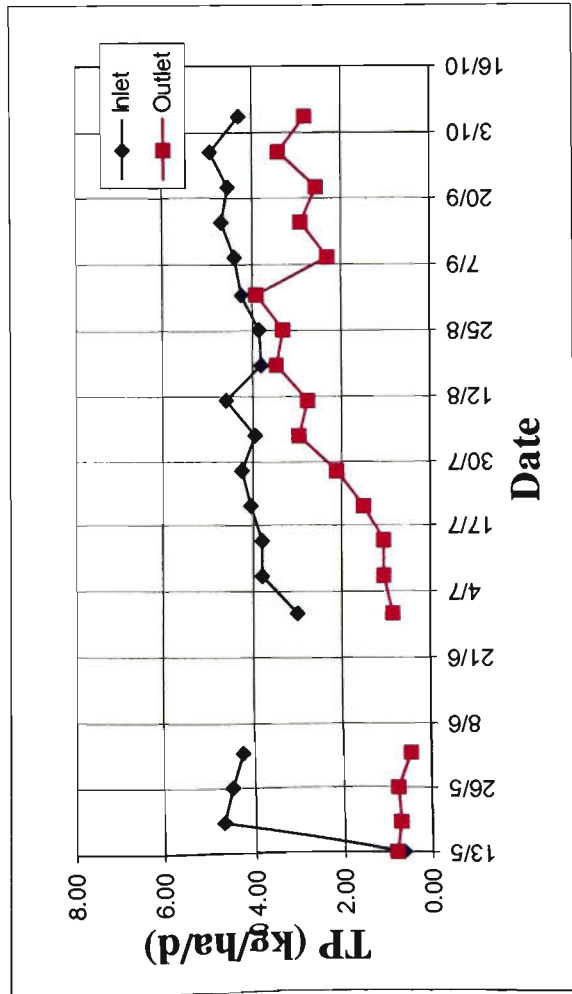


Figure A.22 TP Mass Balance Plots (Continued...)